Environment Canada
Environnement Canada

Atlantic Canada Standards and Guidelines Manual

for the Collection, Treatment, and Disposal of Sanitary Sewage

Prepared by:

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* NOTICE TO THE ENGINEER *

This manual has been prepared for use as a guideline for minimum standards to be met in the collection, treatment, and disposal of sanitary sewage in the Atlantic Provinces. Every effort has been made to ensure that the manual is consistent with current technology and environmental considerations.

THIS MANUAL DOES NOT ELIMINATE THE NECESSITY FOR DETAILED DESIGN. ENGINEERS WHO USE THIS MANUAL IN PREPARING REPORTS, DESIGN DRAWINGS AND SPECIFICATIONS MUST RECOGNIZE THAT HE/SHE RETAINS FULL RESPONSIBILITY FOR THEIR WORK.

PURPOSE OF THE MANUAL

The main body of the manual is intended for use as a guide in the design and preparation of plans and specifications for sewage works and sewage collection and treatment systems; to list and suggest limiting values for items upon which an evaluation of such plans and specifications will be made by the reviewing authority; and to establish as far as practicable, uniformity of practice in Atlantic Canada with practice in other parts of Canada and the United States.

A complete documentation of all parameters related to sewerage works design is beyond the scope of these guidelines, but an attempt has been made to touch upon the parameters of greatest importance from process and reliability standpoints.

By issuing these guidelines, it is not the intention of the regulatory agencies to stifle innovation. Where the designer can show that alternate approaches can produce the desired results, such approaches will be considered for approval.

Wherever possible, designers are encouraged to use actual data obtained from sewage treatment plant flow records, operational studies, etc., rather than use arbitrary design parameters. This is particularly important with sewage treatment plant expansions where the designer may want to use hydraulic and/or organic loading rates in the upper levels of the acceptable loading ranges, or where the designer proposes to deviate from the recommended design parameters.

Where the term "shall" is used, it is intended to mean a mandatory requirement insofar as any confirmatory action by the reviewing authority is concerned. Other terms such as "should", "recommended", "preferred", and "the like" indicate discretionary use on the part of the reviewing authority and deviations are subject to individual consideration.

Designers are advised to familiarize themselves with the requirements of all legislation (as outlined in the policy section) dealing with sewage treatment works, their associated equipment and labour safety requirements.

The manual also contains a number of appendices. These appendices outline typical manpower requirements for various types and sizes of treatment plants, as well as operator training requirements. They also describe treatment plant process control techniques and the recommended format for plant operation and maintenance manuals.

Definition of terms and their use in this document is intended to be in accordance with *Glossary - Water and Sewage Control Engineering*, published by APHA, ASCE, AWWA, and APCF.

The considerable experience of other provincial agencies, authorities and commissions have been freely referred to in the presentation of this manual. Material from the Ministry of the Environment of Ontario, the Water Pollution Control Federation (WPCF), the Water Environment Federation (WEF), the Environmental Protection Agency Office of Technology Transfer, the Association of Boards of Certification Need to Know Criteria, the Nova Scotia Guidelines for the

PURPOSE OF THE MANUAL

Collection and Treatment and Disposal of Municipal Wastewater (1992), the New Brunswick Guidelines for the Collection and Treatment of Wastewater (1987), the Newfoundland guidelines for the Design, Construction, and operation of Water and Sewerage Systems, the Great Lakes - Upper Mississippi River Board of State Sanitary Engineers (199), and the Alberta Standards and Guidelines for Municipal Water Supply, Wastewater, and Storm Drainage Facilities (1988) have all been carefully reviewed and applicable standards have been adopted.

Policies and Criteria contained in this publication will be changed from time to time to conform with advances and improvement in the science, art and practice of Sanitary Engineering. These changes shall be noted in the revision record.

APPENDICES:

1.1 APPLICATION FOR APPROVAL

The regulatory authorities require that application for approval be made in writing by a person responsible for the construction, modification, or operation of sewage works. The application shall be submitted to the appropriate regulatory agency.

An application for the construction or modification of sewage works shall include engineering reports, plans and specifications and all other information which the regulatory agencies may require.

Approval in principle of preliminary reports and plans (concept approval) shall not constitute official approval. No approval for construction or modification can be issued until final detailed plans and specifications have been submitted to the regulatory agency and found to be satisfactory. Such works shall not be undertaken until an official "Certificate of Approval" bearing the necessary signatures has been issued by the Minister of the appropriate regulatory agency.

All final reports, plans and specifications should be submitted at least 90 days prior to the start of the construction or modification. The reports, plans or specifications shall be stamped with the seal and signature of the designing engineer, licensed to practice in the Province of application.

The application shall include sufficient design information and one complete set of plans and specifications submitted directly to the appropriate regulatory agency.

Engineering services are performed in four (4) steps:

a) preliminary evaluation;

b) pre-design report;

c) preparation of construction plans, specifications, contractual documents and design report; and

d) construction compliance, inspection, administration and acceptance, and submission of a post-construction report.

These services are generally performed by engineering firms in private practice but may be executed by municipal or provincial agencies.

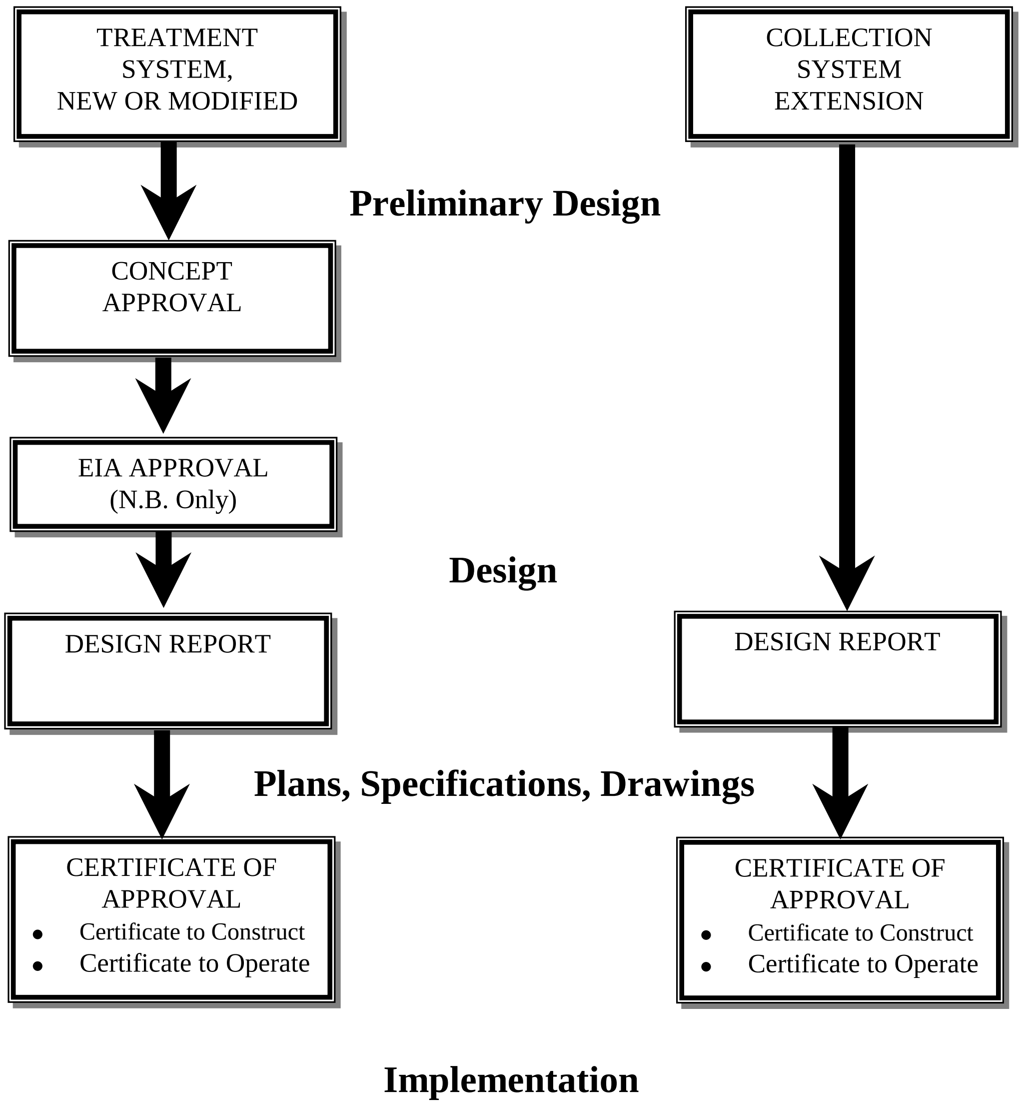
The overall approvals process is outlined in Figure 1.1.

1.2 PRE-DESIGN EVALUATION

A pre-design evaluation shall broadly:

a) describe existing problems;

FIGURE 1.1
APPROVALS PROCESS



Note: New Brunswick issues two separate approvals for construction and operation while Newfoundland, Nova Scotia, and Prince Edward Island include operating approvals within the Certificate of Approval.

b) assess a receiving waters' assimilative capacity;

c) describe design parameters;

d) consider methods for alternate solutions including site and/or route selection;

e) estimate capital and annual operating costs; and

f) outline steps for further project implementation including applications for grants-in-aid and approval by regulatory agencies.

1.2.1 Effluent Discharge Requirements

In the case of wastewater treatment plant effluent discharge requirement will be set by regulatory agencies. These requirements may be as a result of receiving water studies for some cases or they may be governed by a pre-determined discharge policies in others. The discharge policies for Nova Scotia is included in Appendix E. For provinces other than Nova Scotia regulatory agencies should be contacted prior to the start of the pre-design study to determine whether discharge parameters will be set by the regulatory authority or if a receiving water study will be utilized for setting the discharge parameters.

1.2.2 Flow Gauging and Wastewater Characterization Studies

Prior to the preparation of a pre-design report on an existing sewerage system, a comprehensive flow gauging and wastewater characterization study should be conducted. This will aid in gaining a better understanding of important design criteria such as flow rates and variations and wastewater composition.

1.2.3 Infiltration/Inflow Investigations

Prior to the preparation of a pre-design report on a existing sewerage system, a comprehensive infiltration/inflow investigation should be conducted. This will give the designers a better indication of extraneous flow contributions, as well as aid in design solutions, (i.e. the potential for reducing flows at an existing plant).

1.2.4 At-Source Control

Urban wastewater may be composed of many metals/chemicals that are discharged to the sewer system, but may not be treated by conventional treatment, or the level of treatment proposed. This can result in concentrating these constituents into the treatment plant sludge, receiving water, and sediments near the plant outfall.

To protect worker health, collection and treatment infrastructure, and the environment, design of collection and treatment systems must address by-laws and enforcement which keep such materials out of the system (at-source control). Decreased levels of contaminants in wastewater sludge may also result in a saleable commodity that can have economic benefits.

1.3 PRE-DESIGN REPORT

Pre-design reports are necessary in order to obtain a "Concept Approval" from the appropriate regulatory agency. The pre-design report assembles basic information; presents design criteria and assumptions; examines alternate projects with preliminary layouts and cost estimates; describes financing methods giving anticipated charges for users; reviews organizational and staffing requirements; offers a conclusion with a proposed project for client consideration; and outlines official actions and procedures to implement the project.

The concept, factual data and controlling assumptions and considerations for the functional planning of sewage facilities are presented for each process unit and for the whole system. These data form the continuing technical basis for detail design and preparation of construction plans and specifications.

Architectural, structural, mechanical and electrical designs are usually excluded. Sketches may be desirable to aid in presentation of a project. Outline specifications of process units, special equipment, etc., are occasionally included.

1.3.1 Purpose

A pre-design report for a proposed project is used:

a) by the municipality for a description, cost estimates, financing requirements, user commitments, findings, conclusions and recommendations, as a guide to adopt a well-defined project;

b) by the regulatory agency for examination of process operation, control, safety and performance directed to maintenance of water quality when facilities are discharging processed sewage;

c) by investment groups and government funding agencies to evaluate the "quality" of the proposed project with reference to authorization and financing; and

d) by news media for telling a story.

1.3.2 Relation to a Comprehensive Study

The pre-design report for a specific project should be an "outgrowth" of and consistent with an area wide and drainage basin comprehensive study or master plan.

1.3.3 Contents

The pre-design report, to be acceptable for review and approval, must:

a) develop predicted population;

b) establish a specific service area for immediate consideration and indicate possible extensions;

c) present reliable measurements of flow and analyses of wastewater constituents as a basis of process design;

d) estimate costs of immediately proposed facilities;

e) present a reasonable method of financing and show typical financial commitments;

f) suggest an organization and administrative procedure;

g) consider operational requirements with regard to protection of receiving water quality;

h) reflect local bylaws and Federal/Provincial regulations;

i) present summarized findings, conclusions and recommendations for the owner's guidance;

j) include a siting plan. This plan must indicate locations of residences, private and public water supplies, recreational areas, watercourses, zoning, floodplains and other areas of concern when siting sewage collection and treatment facilities;

k) identify existing problems;

l) identify existing and potential receiving water uses; and

m) identify possible treatment plant locations.

1.3.4 Concept and Guidance for Plans and Specifications

The pre-design report should be complete so that plans and specifications may be developed from it without substantial alteration of concept and basis considerations. In short, basic thinking, fundamentals and decisions are spelled out in the pre-design report and carried out in the detailed design plans and specifications.

1.3.5 Format for Content and Presentation

It is urged that the following subsections be utilized as a guideline for content and presentation of the project pre-design report to the Nova Scotia Department of the Environment for review and approval.

1.3.5.1 Title

The Wastewater Facilities Pre-Design Report - collection, conveyance, processing and discharge of wastewater.

1.3.5.2 Letter of Transmittal

A one page letter typed on the firm's letterhead and bound into the report should include:

a) submission of the report to the client;

b) statement of feasibility of the recommended project;

c) acknowledgement to those giving assistance; and

d) reference to the project as outgrowth of approved or "master" plan.

1.3.5.3 Title Page

a) title of project;

b) municipality, county, etc.;

c) names of officials, managers, superintendents;

d) name and address of firm preparing the report; and

e) seal and signature of professional engineer(s) in charge of the project.

1.3.5.4 Table of Contents

a) Section headings, chapter headings and sub-headings;

b) maps;

c) graphs;

d) illustrations, exhibits;

e) diagrams; and

f) appendices.

1.3.5.5 Summary

Highlight, very briefly, what was found from the study.

1.3.5.5.1 Findings

a) population-present, design (when), ultimate;

b) land use and zoning - portion per residential, commercial, industrial, greenbelt, etc.;

c) wastewater characteristics and concentrations - portions of total hydraulic, organic and solid loading attributed to residential commercial and industrial fractions;

d) collection system projects - immediate needs to implement recommended project, deferred needs to complete recommended project and pump stations, force mains, appurtenances, etc.

e) selected process - characteristics of process and characteristics of output.

f) receiving waters - existing water quality and quantity, downstream water uses and impact of project on receiving water;

g) proposed project - total project cost, total annual expense requirement for: debt service; operation, personnel and operation, non-personnel;

h) environmental assessment of selected process;

i) energy requirements - quantities, costs and forms;

j) finances - indicate financing requirements and typical annual charges;

k) organization - administrative control necessary to implement project, carry through to completion and operate and maintain wastewater facility and system; and

l) changes - alert client to situations that could alter recommended project.

1.3.5.5.2 Conclusions

Project, or projects, recommended to client for immediate construction, suggested financing program, etc.

1.3.5.5.3 Recommendations

Summarized, step-by-step actions, for the client to follow in order to implement conclusions:

a) acceptance of report;

b) adoption of recommended project;

c) submission of report to regulatory agencies for review and approval;

d) authorization of engineering services for approved project (construction plans, specifications, contract documents, etc.);

e) legal services

f) enabling ordinances, resolutions, etc., required;

g) adoption of sewer-use ordinance;

h) adoption of operating rules and regulations;

i) financing program requirements;

j) organization and administration (structure, personnel, employment, etc.);

k) time schedules - implementation, construction, completion dates, reflecting applicable hearings, stipulations, abatement orders.

1.3.5.6 Introduction

1.3.5.6.1 Purpose

Reasons for report and circumstances leading up to report.

1.3.5.6.2 Scope

Coordination of recommended project with approved comprehensive master plan and guideline for developing the report.

1.3.5.7 Background

Present only appropriate past history.

1.3.5.7.1 General

a) existing area, expansion, annexation, inter-municipal service, ultimate area;

b) drainage basin, portion covered;

c) population growth, trends, increase during design life of facility (graph);

d) residential, commercial and industrial land use, zoning, population densities, industrial types and concentrations;

e) topography, general geology and effect on project;

f) meteorology, precipitation, runoff, flooding, etc. and effect on project; and

g) total period of time for which project is to be studied.

1.3.5.7.2 Economic

a) assessed valuation, tax structure, tax rates, portions for residential, commercial, industrial property.

b) employment from within and outside service area;

c) transportation systems, effect on commuter influx;

d) exempt property; churches and agricultural exhibition, properties and effect on project; and

e) costs of present water and wastewater services.

1.3.5.7.3 Regulations

a) existing ordinances, rules and regulations including defects and deficiencies, etc.;

b) recommended amendments, revisions or cancellation and replacement;

c) sewer-use ordinance (toxic, aggressive, volatile, etc., substances);

d) surcharge based on volumes and concentration for industrial wastewaters;

e) existing contracts and agreements (inter-municipal, etc.); and

f) enforcement provisions including inspection, sampling detection, penalties, etc.

1.3.5.8 Hydraulic Capacity

The following flows for the design year shall be identified and used as a basis for design for sewers, lift stations, wastewater treatment plants, treatment units, and other wastewater handling facilities. Where any of the terms defined in this Section are used in these design standards, the definition contained in this Section applies.

a. Design Average Flow

The design average flow is the average of the daily volumes to be received for the continuous 12 month period expressed as a volume per unit time. However, the design average flow for facilities having critical seasonal high hydraulic loading periods (e.g., recreational areas, campuses, industrial facilities) shall be based on the daily average flow during the seasonal period.

b. Design Maximum Day Flow

The design maximum day flow is the largest volume of flow to be received during a continuous 24 hour period expressed as a volume per unit time.

c. Design Peak Hourly Flow

The design peak hourly flow is the largest volume of flow to be received during a one hour period expressed as a volume per unit time.

d. Design Peak Instantaneous Flow

The design peak instantaneous flow is the instantaneous maximum flow rate to be received.

e. Design Minimum Day Flow

The design minimum day flow is the smallest volume of flow to be received during a 24 hour period during dry weather when infiltration/inflow are at a minimum, expressed as a volume per unit time.

1.3.5.9 Investigative Considerations - Existing Facilities Evaluation

1.3.5.9.1 Existing Collection System

a) Inventory of existing sewers;

b) isolation from water supply wells;

c) adequacy to meet project needs (structural condition, hydraulic capacity tabulation);

d) gauging and infiltration tests (tabulate);

e) overflows and required maintenance, repairs and improvements;

f) outline repair, replacement and storm water separation requirements;

g) evaluation of costs for treating infiltration/inflow versus costs for rehabilitation of system;

h) establish renovation priorities, if selected;

i) present recommended annual program to renovate sewers; and

j) indicate required annual expenditure.

1.3.5.9.2 Existing Treatment Plant

a) area for expansion;

b) surface condition;

c) subsurface conditions;

d) isolation from habitation;

e) isolation from water supply structures;

f) enclosure of units, winter conditions, odour control, landscaping, etc.; and

g) flooding (predict elevation of 25 and 100 year flood stage).

1.3.5.9.3 Existing Process Facilities

a) capacities and adequacy of units (tabulate);

b) relationship and/or applicability to proposed project;

c) age and condition;

d) adaptability to different usages;

e) structures to be retained, modified or demolished; and

f) outfall.

1.3.5.9.4 Existing Wastewater Characteristics

a) water consumption (from records) total, unit, industrial;

b) wastewater flow pattern, peaks, total design flow;

c) physical, chemical and biological characteristics and concentrations; and

d) residential, commercial, industrial, infiltration fractions, considering organic solids, toxic aggressive, etc., substances; tabulate each fraction separately and summarize.

1.3.5.10 Proposed Project

1.3.5.10.1 Collection System

a) inventory of proposed additions;

b) isolation from water supply well, reservoirs, facilities, etc.;

c) area of services;

d) unusual construction problems;

e) utility interruption and traffic interference;

f) restoration of pavements, lawns, etc.; and

g) basement flooding prevention during power outage.

1.3.5.10.2 *Site Requirements*

Comparative advantages and disadvantages as to cost, hydraulic requirements, flood control, accessibility, enclosure of units, odour control, landscaping, etc., and isolation with respect to potential nuisances and protection of water supply facilities.

1.3.5.10.3 *Wastewater Characteristics*

a) character of wastewater necessary to insure amenability to process selected;

b) need to pretreat industrial wastewater before discharge to sewers;

c) portion of residential, commercial, industrial wastewater fractions to comprise projected growth.

1.3.5.10.4 *Receiving Water Considerations and Assimilative Capacity*

a) wastewater discharges upstream;

b) receiving water base flow (utilize critical flow as specified by approving agency);

c) characteristics (concentrations) of receiving waters;

d) downstream water uses including water supply, recreation, agricultural, industrial, etc.;

e) impact of proposed discharge on receiving waters;

f) tabulate assimilative capacity requirements;

g) listing of effluent characteristics; and

h) tabulation and correlation of plant performance versus receiving water requirements.

1.3.5.11 Alternatives

Alternatives should consider such items as regional solution, optimum operation of existing facilities, flow and waste reduction, location of facilities,phased construction, necessary flexibility and reliability, sludge disposal, alternative treatment sites, alternative processes and institutional arrangements.

1.3.5.11.1 Alternate Process and Site

a) describe and delineate (line diagrams);

b) preliminary design for cost estimates;

c) estimates of project cost (total) dated, keyed to construction cost index, escalated, etc.;

d) advantages and disadvantages of each;

e) individual differences, requirements, limitations;

f) characteristics of process output;

g) comparison of process performances;

h) operation and maintenance expenses;

i) annual expense requirements (tabulation of annual operation, maintenance, personnel, debt obligation for each alternate), and

j) environmental assessment of each.

1.3.5.12 Selected Process and Site

a) identify and justify process and site selected;

b) adaptability to future needs;

c) environmental assessment;

d) outfall location; and

e) describe immediate and deferred construction.

1.3.5.13 Project Financing

a) review applicable financing methods;

b) effect of Provincial and Federal funding;

c) assessment by front meter, area unit or other benefit;

d) charges by connection, occupancy, readiness-to-serve, water consumption, industrial wastewater discharge, etc.;

e) existing debt service requirements;

f) annual financing and bond retirement schedule;

g) tabulate annual operating expenses;

h) show anticipated typical annual charge to user and non-user; and

i) show how representative properties and users are to be affected.

1.3.5.14 Legal and Other Considerations

a) needed enabling legislation, ordinances, rules and regulations;

b) contractual considerations for inter-municipal cooperation;

c) public information and education; and

d) statutory requirements and limitations.

1.3.5.15 Appendices: Technical Information and Design Criteria

1.3.5.15.1 Collection System

a) design tabulations - flow, size, velocities, etc.;

b) regulator or overflow design;

c) pump station calculations, including energy requirements;

d) special appurtenances;

e) stream crossings; and

f) system map (report size).

1.3.5.15.2 Process Facilities

a) criteria selection and basis;

b) hydraulic and organic loadings - minimum, average, maximum and effect;

c) unit dimensions;

d) rates and velocities;

e) detentions;

f) concentrations;

g) recycle;

h) chemical additive control;

i) physical control;

j) removals, effluent concentrations, etc. Include a separate tabulation for each unit to handle solid and liquid fractions;

k) energy requirement; and

l) flexibility.

1.3.5.15.3 Process Diagrams

a) process configuration, interconnecting piping, processing, flexibility, etc.;

b) hydraulic profile;

c) organic loading profile;

d) solids control system;

d) solids profile; and

e) flow diagram with capacities, etc.

1.3.5.15.4 Space for Personnel, Laboratories and Records

1.3.5.15.5 Chemical Control

a) processes needing chemical addition;

b) chemicals and feed equipment; and

c) tabulation of amounts and unit and total costs.

1.3.5.15.6 Support Data

a) outline unusual specifications, construction materials and construction methods;

b) maps, photographs, diagrams (report size);

c) other.

1.4 DETAILED DESIGN DOCUMENTATION

1.4.1 General

Upon obtaining a "Concept Approval," the owner or his/her representative must prepare and submit detailed design documentation. This includes a Design Report, plans, specifications and contractual documents, and any applications for approval required by the regulatory agency with jurisdiction over the proposed project.

1.4.2 Design Report

The Design Report shall contain detailed design calculations for each unit or process of the wastewater treatment or collection facility. The design report shall also address operational and maintenance issues for that particular facility.

1.4.2.1 Format for Content and Presentation

It is urged that the following subsection be utilized as a guideline for content and presentation of the project Design Report to the appropriate regulatory agency for review and approval.

1.4.2.1.1 Title

The Wastewater Facilities Design Report - collection, conveyance, processing and discharge of wastewater.

1.4.2.1.2 Letter of Transmittal

A one page letter typed on the firm's letterhead and bound into the report should include:

a) submission of the report to the client;

b) acknowledgement to those giving assistance; and

c) reference to the project as outgrowth of approved or "master" plan.

1.4.2.1.3 Title Page

a) title of project;

b) municipality, county, etc.;

c) names of officials, managers, superintendents;

d) name and address of firm preparing the report; and

e) seal and signature of professional engineer(s) in charge of the project.

1.4.2.1.4 Table of Contents

a) section headings, chapter headings and sub-headings

b) maps;

c) graphs;

d) illustrations, exhibits;

e) diagrams; and

f) appendices.

1.4.2.1.5 Collection System

a) detailed design tabulations - flow, size, velocities, etc;

b) regulator or overflow design calculations;

c) detailed pump station calculations, including energy requirements;

d) special appurtenances;

e) stream crossings; and

f) system map (report size).

1.4.2.1.6 Process Facilities

a) hydraulic and organic loadings - minimum, average, maximum and effect;

b) detailed calculations used to determine:

- unit dimensions;
- rates and velocities;
- detentions;
- concentrations;
- recycle;
- removals, effluent concentrations, etc. Include a separate tabulation for each unit to handle solid and liquid fractions;
- energy requirement;
- flexibility; and

c) chemical requirements and control.

1.4.2.1.7 Process Diagrams

a) process configuration, interconnecting piping, processing, flexibility, etc;

b) hydraulic profile;

c) organic loading profile;

d) solids control system;

e) solids profile; and

f) flow diagram with capacities, etc.

1.4.2.1.8 Laboratory

a) physical and chemical tests and frequency to control process;

b) time for testing;

c) space and equipment requirements; and

d) personnel requirements - number, type, qualifications, salaries, benefits (tabulate).

1.4.2.1.9 Operation and Maintenance

a) routine and special maintenance duties;

b) time requirements;

c) tools, equipment, vehicles, safety, etc.;

d) personnel requirements - number, type, qualifications, salaries, benefits, (tabulate); and

e) maintenance work space and storage.

1.4.2.1.10 Office Space for Administrative Personnel and Records

1.4.2.1.11 Personnel Service - Locker Room and Lunch Room

1.4.2.1.12 Chemical Control

a) process needing chemical addition;

b) chemicals and feed equipment; and

c) tabulation of amounts and unit and total costs.

1.4.2.1.13 Collection System Control

a) cleaning and maintenance;

b) regulator and overflow inspection and repair;

c) flow gauging;

d) industrial sampling and surveillance;

e) regulation enforcement;

f) equipment requirements;

g) trouble-call investigation; and

h) personnel requirements - number, type, qualifications, salaries, benefits (tabulate).

1.4.2.1.14 Control Summary

a) personnel;

b) equipment;

c) chemicals;

d) utilities - list power requirements of major units; and

e) summation.

1.4.2.1.15 Support Data

a) outline unusual specifications, construction materials and construction methods;

b) maps, photographs, diagrams (report size); and

c) other.

1.4.2.1.16 Appendices

Related data not necessary to an immediate understanding of the design report should be placed in the appendices.

1.4.3 Plans

1.4.3.1 General

All plans for sewage work shall bear a suitable title showing the name of the municipality, sewer district, or institution; and shall show the scale in appropriate units, the north point, date and the name of the engineer, his signature on an imprint of his registration seal.

The plans shall be clear and legible. They shall be drawn to scale which will permit all necessary information to be plainly shown. The size of the plans should be 570 x 817 mm (size A1 (21 x 33 in (size D)). Datum used should be indicated. Locations and logs of test borings, when made, shall be shown on the plans.

Detail plans shall consist of plan views, elevations, sections and supplementary views which, together with the specifications and general layouts, provide the working information for the contract and construction of the works. Include dimensions and geodetic elevations of structures, the location and outline form of equipment, location and size of piping, water levels and ground elevations.

1.4.3.2 Plans of Sewers

1.4.3.2.1 General Plans

A comprehensive plan of the existing and proposed sewers shall be submitted for projects involving new sewer systems or substantial additions to existing systems. This plan shall show the following:

a) Geographical Features

i) topography and elevations - existing or proposed streets and all streams or water surfaces shall be clearly shown. Contour lines at suitable intervals should be included;

ii) streams -the direction of flow in all streams and high and low water elevations of all water surfaces at sewer outlets and overflows shall be shown.

iii) boundaries - the boundary lines of the municipality, the sewer district or area to be sewered shall be shown.

b) Sewers

The plan shall show the location, size and direction of flow of all existing and proposed sanitary and combined sewers draining to the treatment works concerned.

1.4.3.2.2 Detail Plans

Detail plans shall be submitted. Profiles should have a horizontal scale of not more than 1:500 and a vertical scale of not more than 1:50. Plans and profiles shall show:

a) location of streets and sewers;

b) line of ground surface, size, material and type of pipe, length between manholes, invert and surface elevation at each manhole and grade of sewer between each two adjacent manholes. All manholes shall be numbered on the plan and correspondingly numbered on the profile.

Where there is any question of the sewer being sufficiently deep to serve any residence, the elevation and location of the basement floor shall be plotted on the profile of the sewer which is to serve the house in question. The engineer shall state that all sewers are sufficiently deep to serve adjacent basements except where otherwise noted on the plans.

c) locations of all special features such as inverted siphons, concrete encasement, elevated sewers, etc.;

d) all known existing structures both above and below ground which might interfere with the proposed construction, particularly water mains, gas mains, storm drains, etc.;

e) special detail drawings, made to a scale to clearly show the nature of the design, shall be furnished to show the following particulars:

(i) all stream crossings and sewer outlets, with elevations of the stream bed and of normal and extreme high and low water levels;

(ii) details of all special sewer joints and cross-sections; and

(iii) details of all sewer appurtenances such as manholes, lamp holes, inspection chambers, inverted siphons, regulators, tide gates and elevated sewers.

1.4.3.3 Plans of Sewage Pumping Stations

1.4.3.3.1 Location Plan

A plan shall be submitted for projects involving construction or revision of pumping stations. This plan shall show the following:

a) the location and extent of the tributary area;

b) any municipal boundaries with the tributary area; and

c) the location of the pumping station and force main and pertinent elevations.

1.4.3.3.2 Detail Plans

Detail plans shall be submitted showing the following, where applicable:

a) topography of the site;

b) existing pumping station;

c) proposed pumping station, including provisions for installation of future pumps;

d) elevation of high water at the site and maximum elevation of sewage in the collection system upon occasion of power failure;

e) maximum hydraulic gradient in downstream gravity sewers when all installed pumps are in operation; and

f) test borings and groundwater elevations.

1.4.3.4 Plans of Sewage Treatment Plant

1.4.3.4.1 Location Plans

A plan shall be submitted, showing the sewage treatment plant in relation to the remainder of the system.

Sufficient topographic features shall be included to indicate its location with relation to streams and the point of discharge of treated effluent.

1.4.3.4.2 General Layout

Layouts of the proposed sewage treatment plant shall be submitted, showing:

a) topography of the site;

b) size and location of plant structures;

c) schematic flow diagram showing the flow through various plant units;

d) piping, including any arrangements for by-passing individual units. Materials handled and direction of flow through pipes shall be shown;

e) hydraulic profiles showing the flow of sewage, supernatant, mixed liquor and sludge; and

e) test borings and ground water elevations.

1.4.3.4.3 Detail Plans

a) Location, dimensions and elevations of all existing and proposed plant facilities;

b) elevations of high and low water level of the body of water to which the plant effluent is to be discharged;

c) type, size, pertinent features and manufacturer's rated capacity of all pumps, blowers, motors and other mechanical devices;

d) minimum, average and maximum hydraulic flow in profile; and

e) adequate description of any features not otherwise covered by specifications or engineer's report.

1.4.4 Specifications

Complete technical specifications for the construction of sewers, sewage pumping stations, sewage treatment plants and all appurtenances, shall accompany the plans.

The specifications accompanying construction drawings shall include, but not be limited to, all construction information not shown on the drawings which is necessary to inform the builder in detail of the design requirements as to the quality of materials and workmanship and fabrication of the project and the type, size, strength, operating characteristics and rating of equipment; allowable infiltration; the complete requirements for all mechanical and electrical equipment, including machinery, valves, piping and jointing of pipe; electrical apparatus, wiring and meters; laboratory fixtures and equipment; operating tools; construction materials; special filter materials such as stone, sand, gravel or slag; miscellaneous appurtenances, chemicals when used; instructions for testing materials and equipment as necessary to meet design standards; and operating tests for the completed works and component units. It is suggested that these performance tests be conducted at design load conditions wherever practical.

1.5 REVISIONS TO APPROVED PLANS

Any deviations from approved plans or specifications affecting capacity, flow or operation of units shall be approved in writing before such changes are made. Plans or specifications so revised should, therefore, be submitted well in advance of any construction work which will be affected by such changes, to permit sufficient time for review and approval. Structural revisions or other minor changes not affecting capacities, flows, or operation will be permitted during construction without approval. "As-built" plans clearly showing such alterations shall be submitted to the reviewing agency at the completion of the work.

1.6 CERTIFICATE OF APPROVAL

A Certificate of Approval shall be issued prior to construction by the appropriate regulatory agency to the owner/operator only upon final approval of the Design report, plans, specifications and contract documents. The permit shall provide the owner/operator with the authority to proceed with the construction of that particular project. The Certificate of Approval only provides the owner/operator with the authority to operate the newly constructed system if it is clearly identified in the approval. Depending upon the province of jurisdiction a separate operating approval may be issued following construction.

1.7 OPERATION DURING CONSTRUCTION

Specifications shall contain a program for keeping existing treatment plant units in operation during construction of plant additions. Should it be necessary to take plant units out of operation, a shut-down schedule which will minimize pollution effects on the receiving stream, shall be reviewed and approved in advance by the appropriate reviewing agency and shall be adhered to.

1.8 OPERATING REQUIREMENTS

1.8.1 General

Any newly constructed sewerage system or treatment plant shall be put into operation only if it meets each of the following criteria:

a) in the case of Nova Scotia and New Brunswick a "Construction Report", and in the case of Prince Edward Island and Newfoundland a "Certificate of Compliance" has been submitted by the engineer to the regulatory agency and has subsequently been reviewed and approved; and

b) in the case of New Brunswick an "Approval to Operate" has been issued by the regulatory agency to the owner/operator of the system or plant. In the case of Nova Scotia, Prince Edward Island, and Newfoundland the "Certificate of Approval" issued before construction will address operating issues.

1.8.2 Post-Construction Report / Certificate of Compliance

The "Post-Construction Report / Certificate of Compliance" shall contain all information regarding major changes from the approved plans or specification made during construction. These major changes include any deviations which affect capacity, flow or operation of units. The "Post-Construction Report / Certificate of Compliance" shall also include all commission or start-up of equipment tests and any other tests results produced during construction. The "Post-Construction Report / Certificate of Compliance" must also guarantee that all as-built drawings, operation and maintenance manuals, and any other relevant documentation have been turned over to the owner/operator by the engineer.

1.8.3 Approval To Operate

In the province of New Brunswick, after the completion of construction and submission of the construction report, the owner/operator or his authorized agent shall apply to the appropriate regulatory agency for an "Approval to Operate".

The purpose of the permit is to clearly outline the operating and reporting requirements for the wastewater treatment facility. The permit shall outline the plant's effluent limitations. The owner or his authorized agent should consult with the regulatory agency to develop the terms and conditions of the licence, followed by the submission of an application for the licence itself. The application should include a letter with enclosed completed application forms and a copy of the operations manual for the facility.

The terms and conditions of any "Permit to Operate" shall only remain in effect for a specified period. Following this period, the owner/operator shall apply for a renewal of the permit.

In Newfoundland, Nova Scotia, and Prince Edward Island operating issues will be covered in the "Certificate of Approval" issued prior to construction.

1.9 MONITORING REQUIREMENTS

A monitoring program, including regular sampling of sewage treatment systems effluent and recording of flows, shall be undertaken by the systems operating authority/owner. This monitoring program should be carried out in compliance with sampling and analysis requirements set by the appropriate regulatory agency.

A typical sampling frequency for compliance is presented in the following table:

Table 1.1 SAMPLING REQUIREMENTS

Plant Class	Sampling Frequency
I	6/year
II	2/month
III	2/month
IV	1/week

Samples should be 24-hour composite samples, except for those collected at lagoons, which may be grab samples. Samples shall be analyzed for BOD_5, suspended solids and fecal coliforms. Additional parameters requiring sampling shall be listed in the "Approval to Operate" or " Certificate of Approval".

1.9.1 Owner/Operator Responsibility

The owner/operator of any wastewater treatment or collection facility shall be responsible for conducting all compliance sampling. The owner/operator shall ensure that all compliance sampling is conducted in accordance with Section 1.9 and in the stipulations of the "Approval to Operate" or "Certificate of Approval".

1.9.2 Regulatory Agencies' Responsibility

The regulatory agency shall be responsible for enforcing compliance requirements, as described in the Certificate of Approval/Approval to Operate issued to any wastewater treatment or collection facility.

1.10 COMPLIANCE REQUIREMENTS

A sewage treatment plant shall be considered in compliance with the effluent limitations if 80 percent of the sample test results, the frequency and number of which are specified by the regulatory authority, meet the required standard for that plant. No single result can be greater than two times the required standard for the plant.

1.11 REPORTING REQUIREMENTS

The operator/authority/owner shall ensure that all monitoring results are submitted to the appropriate regulatory agency.

2.1 TYPE OF SEWERAGE SYSTEM

In general and except for special reasons, the Minister will approve plans for new systems or extensions only when designed upon a separate sewer basis, in which rain water from roofs, streets and other areas and groundwater from foundation drains are excluded. Overflows from intercepting sewers should not be permitted at points where they will adversely affect a watercourse or the use of water therefrom. Otherwise provision shall be made for treating the overflow.

2.2 DESIGN CAPACITY CONSIDERATIONS

In general, sewer systems should be designed for the estimated ultimate tributary population, except in considering parts of the systems that can be readily increased in capacity. Similarly, consideration should be given to the maximum anticipated capacity of institutions, industrial parks, etc.

In determining the required capacities of sanitary sewers the following factors should be considered:

a. maximum hourly domestic sewage flow;

b. additional maximum sewage or waste from industrial plants;

c. inflow and groundwater infiltration;

d. topography of area;

e. location of waste treatment plant;

f. depth of excavation; and

g. pumping requirements.

The basis of design for all sewer projects shall accompany the documents.

2.3 HYDRAULIC DESIGN

2.3.1 Sewage Flows

Sewage flows are made up of waste discharges from residential, commercial, institutional and industrial establishments, as well as extraneous non-waste flow contributions such as groundwater and surface runoff.

2.3.2 Extraneous Sewage Flows

2.3.2.1 Inflow

When designing sanitary sewer systems, allowances must be made for the leakage of groundwater into the sewers and building sewer connections (infiltration) and for other extraneous water entering the sewers from such sources as leakage through manhole covers, foundation drains, roof down spouts, etc.

Due to the extremely high peak flows that can result from roof down spouts, they should not, in any circumstances, be connected directly, or indirectly via foundation drains, to sanitary sewers. The Departments of Municipal Affairs, The Environment and Health also discourage the connection of foundation drains to sanitary sewers. Studies have shown that flows from this source can result in gross overloading of sewers, pumping stations and sewage treatment plants for extended periods of time. The Departments recommend that foundation drainage be directed either to the surface of the ground or into a storm sewer system, if one exists.

2.3.2.2 Infiltration

The amount of groundwater leakage directly into the sewer system (infiltration) will vary with the quality of construction, type of joints, ground conditions, level of groundwater in relation to pipe, etc. Although such infiltration can be reduced by proper design and construction, it cannot be completely eliminated and an allowance must be made in the design sewage flows to cover these flow contributors. Despite the fact that these allowances are generally referred to as infiltration allowances, they are intended to cover the peak extraneous flows from all sources likely to contribute non-waste flows to the sewer system. The infiltration allowances used for sewer design should not be confused with leakage limits used for acceptance testing following construction. The latter allowance are significantly lower and apply to a sewer system when the system is new and generally without the private property portions of the building sewers constructed.

2.3.3 Domestic Sewage Flows

Unless actual flow measurement has been conducted, the following criteria should be used in determining peak sewage flows from residential areas, including single and multiple housing, mobile home parks, etc.:

a. design population derived from drainage area and expected maximum population over the design period;

b. average daily domestic flow (exclusive of extraneous flows) of 340 L/cap·d;

c. peak extraneous flow (including peak infiltration and peak inflow) of from 0.14 to greater than 0.28 L/s per gross hectare; and

d. peak domestic sewage flows to be calculated by the following equation:

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

where:

Q(d) = peak domestic sewage flow (including extraneous flow) in L/s.

P = design population, in thousands

q = average daily per capita domestic flow in L/cap.d. (exclusive of extraneous flows)

M = peaking factor (as derived from

Harman Formula **Babbit Formula**

$$M = 1 + \frac{14}{4 + P^{0.5}} \quad \text{or} \quad M = \frac{5}{P^{0.2}} \quad \text{or as}$$

determined from flow studies for similar developments in the same municipality). The minimum permissible peaking factor shall be 2.0.

I = unit of peak extraneous flow, in L/s per hectare.

A = tributary area in gross hectares.

2.3.4 Commercial and Institutional Sewage Flows

2.3.4.1 Flow Variation

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

2.3.4.2 Flow Equivalent

In general, the method of estimating sewage flows for large commercial areas is to estimate a population equivalent for the area covered by the development and then calculate the sewage flows on the same basis in the previous section. A population equivalent of 85 persons per hectare is often used. It is also necessary to calculate an appropriate peaking factor and select a representative unit of peak extraneous flow.

2.3.4.3 Individual Flow Rate

For individual commercial and institutional users the following sewage flow rates are commonly used for design. Where a range is stated the lower figure is the minimum requirement.

Sewage Flows (Average Daily)

Shopping Centres	- 2500-5000 L/1000 m-day (based on total floor area)
School (basic)	- 40 L/student-day
cafeteria	- 60 L/student-day
shower	- 80 L/student-day
Nursing Home	- 375 L/bed-day
Senior Citizen Home	
	- 650 L/apartment-day
Hospital	- 950 L/bed-day
Restaurant	- 225 L/seat-day + 100 L/employee-day
Motel	- 300 L/room-day (add for restaurant)
Campgrounds	- 500 L/campsite-day

2.3.4.4 Peak Factor

When using the above unit demands, maximum day and peak rate factors must be developed. For establishments in operation for only a portion of the day, such as schools, shopping plazas, etc., the water usage should also be factored accordingly. For instance, with schools operating for 8 hours per day, the water usage rate will be at an average rate of say 70 L/student-day x 24/8 or 210 L/student day over the 8-hour period of operation. The water usage will drop to residual usage rates during the remainder of the day. Schools generally do not exhibit large maximum day to average day ratios and a factor 1.5 will generally cover this variation. For estimation of peak demand rates, an assessment of the water using fixtures is generally necessary and a fixture-unit approach is often used.

The peak water usage rates in campgrounds will vary with the type of facilities provided (showers, flush toilets, clothes washers, etc.) and the ratio of these facilities to the number of campsites. A peak rate factor of 4 will generally be adequate, however, and this factor should be applied to the average expected water usage at full occupancy of the campsite.

2.3.5 Industrial Sewage Flows

2.3.5.1 Flow Variation

Peak sewage flow rates from industrial areas vary greatly depending on such factors as the extent of the area, the types of industries present, the provision of in-plant treatment or regulation of flows, and the presence of cooling waters in the sanitary sewer system.

2.3.5.2 Flow Rate

The calculation of design sewer flow rates for industrial areas is, therefore, difficult. Careful control over the type of industry permitted in new areas is perhaps the most acceptable way to approach the problem. In this way, a reasonable allowance can be made for peak industrial sewage flow for an area and then the industries permitted to locate in the area can be carefully monitored to ensure that all the overall allowances are not exceeded. Industries with the potential to discharge sewage at higher than the accepted rate could either be barred from the area, or be required to provide flow equalization and/or off-peak discharge facilities, or be restricted by a sewer-use by-law.

2.3.5.3 Flow Allowances

Some typical sewage flow allowances for industrial areas are 35 m^3/hectare-day for light industry and 55 m^3/hectare-day for heavy industry.

2.3.6 Combined Sewer Interceptors

In addition to the above requirements, interceptors for combined sewers shall have capacity to receive sufficient quantity of combined wastewater for transport to treatment works to insure attainment of the appropriate provincial and federal water quality standards

2.3.6.1 Combined Sewer Overflows

The design requirements for sanitary sewers outlined in this manual specify that all new sewer systems be designed as separate sewers. There will, however, still remain many existing combined sewer systems. This will result in the continued existence of combined sewer overflows (CSO's). This being the case, all receiving water quality studies and waste load allocation models must take into account the effect of CSO's. It is the objective of the regulatory agencies to reduce, where possible and practical, the frequency and duration of CSO's so as to minimize their associated impacts on a receiving water.

2.4 DETAILS OF DESIGN AND CONSTRUCTION

2.4.1 Sewer Capacity

Sewers shall be designed to handle the peak anticipated sewage flow when flowing full.

2.4.2 Pressure Pipes

Sanitary sewers may be designed as pressure pipes provided that the hydraulic gradient for maximum flow is below basement elevations.

2.4.3 Minimum Pipe Size

No public sewer shall be less than 200 mm in diameter.

2.4.4 Depth

In general, sewers shall be deep enough to prevent freezing and to receive sewage from most basements.

Insulation shall be provided for sewers that cannot be placed at a depth sufficient to prevent freezing.

2.4.5 Slope

Sewers shall be laid with a uniform slope between manholes.

2.4.5.1 Minimum Slopes

All sewers shall normally be designed and constructed to give mean velocities, when flowing full, of not less than 0.6 meters per second or greater than 4.5 meters per second based on Kutter's or Manning's formula using "n" value of 0.013. Use of other practical "n" values may be permitted by the reviewing agency if deemed justifiable. Velocities above 4.5 m/s may be permitted with high velocity protection.

The following are the minimum slopes which will provide a velocity of 0.6 m/s when sewers are flowing full:

TABLE 2.1 - MINIMUM SLOPES FOR FULL-PIPE VELOCITY OF 0.6 M/S

Sewer Size	Minimum Slope in Metres per 100 Metres
200 mm	0.40
250 mm	0.28
300 mm	0.22
350 mm	0.17
375 mm	0.15
400 mm	0.14
450 mm	0.12
525 mm	0.10
600 mm	0.08
675 mm	0.067
750 mm	0.058
900 mm	0.046

It is recommended that the actual pipe slopes should not be less than 0.5 percent (0.5 m/100 m).

2.4.5.2 *Increased Slopes*

To achieve 0.6 m/s flow velocities in sewers which will flow less than 1/3 full, steeper slopes than given above must be used where conditions permit. For instance, the minimum slopes mentioned above would have to be doubled when depth of flow is only 1/5 full and quadrupled when depth of flow is only 1/10 full to achieve 0.6 m/s flow velocity.

2.4.5.3 *Reduced Slopes*

Under special conditions, if full and justifiable reasons are given, slopes slightly less than those required for the 0.6 meter per second 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 design average flow. Whenever such decreased slopes are selected, the design engineer must furnish with his report his computations of the anticipated flow velocities of average and daily or weekly peak flow rates. The pipe diameter and slopes shall be selected to obtain the greatest practical velocities to minimize settling problems. The operating authority of the sewer system will give written assurance to the appropriate reviewing agency that any additional sewer maintenance required by reduced slopes will be provided.

2.4.5.4 *High Velocity Protection*

Where velocities greater than 4.5 meters per second are unavoidable, special provisions shall be made to protect against displacement by erosion and shock.

2.4.5.5 Steep Slope Protection

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

a. not over 11 meters center to center on grades 20 percent and up to 35 percent.

b. not over 7.3 meters center to center on grades 35 percent and up to 50 percent.

c. not over 5 meters center to center on grades 50 percent and over.

2.4.6 Alignment

Sewers 600 mm or less in diameter shall be laid with a straight alignment between manholes.

2.4.7 Curvilinear Sewers

Curvi-linear sewers may be considered for pipe sizes in excess of 600 mm with the following restrictions applicable:

1. The sewer shall be laid as a simple curve of a radius equal to or greater than 60 m.

2. Manholes shall be located at the ends of the curve and at intervals not greater than 90 m along the curve.

3. The curve shall run parallel to the curb or street center line.

4. The minimum grade on curves shall be fifty percent greater than the minimum grade required for straight runs of sewers.

5. Length of pipe shall be such that deflections at each joint shall be less than the allowable maximum recommended by the manufacturer.

6. In general, curved sewers should be used only where savings in costs or the difficulty of avoiding other utilities necessitates their use.

7. A free ball or tethered ball test should be used, such a ball being 10 mm to 25 mm less in diameter than the sewer.

2.4.8 Changes in Pipe Size

When a sewer joins a larger one at a manhole, the invert of the larger sewer should be lowered sufficiently to maintain the same energy gradient. An approximate method of securing these results is to place the 0.8 depth point of both sewers at the same elevations. Changes in size of sewers less than or equal to 600 mm shall be at manholes only.

2.4.9 Allowance for Hydraulic Losses at Sewer Manholes

Differences in elevation across manholes should be provided to account for hydraulic losses. The elevation drop may be calculated using the head loss formula:

Head loss Across Manholes

$H = k\,(V_2^2 - V_1^2)/2g$

where:

H	=	Head loss	m
k	=	coefficient	dimensionless
V_1	=	entrance velocity	m/s
V_2	=	exit velocity	m/s
g	=	acceleration due to gravity	m/s^2

Where sewer velocities are less than 2.5 m/s and the velocity change across the manhole is less than 0.6 m/s the invert drop may be determined using the following table.

Table 2.2 – Recommended Invert Drop

Invert Drop	
a) straight run	15 mm
b) 45 degree turn	30 mm
c) 90 degree turn	60 mm

2.4.10 Sewer Services

Sewer services shall be consistent with the Local Municipality Authority or Provincial Plumbing and Drainage Regulations. It is required that unless Tees or "Wyes" have been installed, that saddles be used in connecting the service to the sewer. Generally these are placed at an angle of 45 degrees above horizontal. Connections shall be made by authorized personnel only.

Pipes with watertight and rootproof joints should be used for house connections. Minimum pipe size should be 150 mm diameter for double connections and 100 mm diameter for single connections.

2.4.11 Sulphide Generation

Where sulphide generation is a possibility, the problem shall be minimized by designing sewers to maintain flows at a minimum cleansing velocity of 1.0 m/s.

Where corrosion is anticipated because of either sulphate attack or sulphides, consideration shall be given to the provision of corrosion resistant pipe material or effective protective linings.

2.4.12 Materials

Any generally accepted material for sewers will be given consideration, but the material selected should be adaptable to local conditions, such as character of industrial wastes, possibility of septicity, soil characteristics, exceptionally heavy external loading, abrasion and similar problems.

All sewers shall be designed to prevent damage from super-imposed loads. Proper allowance for loads on the sewer shall be made because of the width and depth of trench. When standard strength sewer pipe is not sufficient, the additional strength needed may be obtained by using extra strength pipe or by special construction.

2.4.13 Metering and Sampling

Where no other measuring devices are provided, one manhole on the outfall line shall be constructed with a suitable removable weir for flow measurements. Easy access for flow measurement and sampling shall be provided. Similar manholes should be constructed on sewer lines from industries to facilitate checking the volume and composition of the waste.

2.4.14 Sewer Extensions

In general, sewer extensions shall be allowed only if the receiving sewage treatment plant is either:

a. Capable of adequately processing the added hydraulic and organic load or

b. Provision of adequate treatment facilities on a time schedule acceptable to the approving agencies is assured.

2.4.15 Installation

2.4.15.1 Standards

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

2.4.15.2 Trenching

a. The width of the trench shall be ample to allow the pipe to be laid and jointed properly and to allow the backfill to be placed and compacted as needed. The trench sides shall be kept as nearly vertical as possible. When wider trenches are dug, appropriate bedding class and pipe strength shall be used.

b. Ledge rock, boulders and large stones shall be removed to provide a minimum clearance of 150 mm below and on each side of all pipe(s).

2.4.15.3 *Foundation*

The foundation provides the base for the sewer pipe soil system. The project engineer should be concerned primarily with the presence of unsuitable soils, such as peat or other highly organic or compressible soils, and with maintaining a stable trench bottom.

2.4.15.4 *Bedding*

The sewer pipe should be bedded on carefully compacted granular material. The granular material shall have a minimum thickness of 150 mm and cover the full width of the trench.

In general, a well-graded crushed stone is a more suitable material for sewer pipe bedding than a uniformly graded pea gravel. For small sewer pipes, the maximum size should be limited to about 10% of the pipe diameter. Crushed stone or gravel meeting the requirement of ASTM Designation C33, Gradation 67 (19-9.8 mm) will provide the most satisfactory sewer pipe bedding. However, the recommendation of the manufacturer should also be taken into consideration when specifying a particular bedding material. Material removed from the trench shall not be used as bedding material.

2.4.15.5 *Haunching*

The material placed at the sides of a pipe from the bedding up to the spring line is the haunching.

Material used for sewer pipe haunching should be shovel sliced or otherwise placed to provide uniform support for the pipe barrel and to fill completely all voids under the pipe. Haunching material is to be compacted manually. The material used may be similar to the material used for bedding. Material removed from the trench shall not be used as haunching material.

2.4.15.6 *Initial Backfil*

Initial backfill is the material which covers the sewer pipe and extends from the haunching to a minimum of 300 mm above the top of the pipe. Its function is to anchor the sewer pipe, protect the pipe from damage by subsequent backfill and insure the uniform distribution of load over the top of the pipe. It should be placed in layers. The material used for initial backfill may be similar to the material used for bedding and haunching, however, it shall be of a material which will develop a uniform and relatively high density with little compactive effort. Material removed from the trench shall not be used as initial backfill.

2.4.15.7 *Final Backfill*

Final backfill is the material which extends from the top of the initial backfill to the top of the trench. It should be placed in 300 mm layers.

The material consists of the excavated material containing no organic matter or rocks having any dimension greater than 200 mm. In most cases, final backfill does not affect the pipe design. Compaction of the final backfill is usually controlled by the location as follows: traffic areas; 95% of modified Proctor density required; general urban areas; 90% of modified Proctor density may be adequate; undeveloped areas; 85% of modified Proctor density may be required. Trench backfilling should be done in such a way as to prevent dropping of material directly on the top of pipe through any great vertical distance.

2.4.15.8 Borrow Materials

Because the material removed from the trench is not to be used as part of the bedding, haunching, nor initial backfill, material must be imported from another source. Borrow material must meet the specifications for final backfill.

Either cohesive or noncohesive material may be used; however, the project engineer should assess the possible change in groundwater movement if cohesive material is used in rock or if noncohesive material is used in impermeable soil.

2.4.15.9 Deflection Test

a. Deflection testing will not be necessary unless one or more of the following conditions are known to exist:

- Improper construction practices are evident
- Questionable embedment materials have been used
- Severe trench construction conditions were encountered
- Other inspection or testing methods have indicated unacceptable installation conditions.

b. Measure pipe deflection by pulling a mandrel gauge through each pipe from manhole to manhole after backfilling. Within thirty days after installation, pull a mandrel gauge measuring 5% deflection through the installed section of pipeline. If this test fails, proceed with 7½% deflection test.

c. 7½% deflection test: Thirty days prior to completion of the Period of Maintenance, pull a mandrel gauge measuring 7½% deflection through the installed section of pipeline. If the 7½% deflection test fails, the defects must be located and repaired, and the pipe subsequently retested.

d. If the deflection test is to be run using a rigid ball or mandrel, it shall have a diameter equal to 92.5% of the inside diameter of the pipe. The test shall be performed without mechanical pulling devices.

2.4.16 Joints

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

2.5 MANHOLES

2.5.1 Location

Manholes shall be located at all junctions, changes in grade, size or alignment (except with curvilinear sewers) and termination points of sewers.

2.5.2 Spacing

2.5.2.1 Normal Spacing

The maximum acceptable spacing for manholes is 90 to 120 m for sewers 200 to 450 mm in diameter. Spacings of up to 150 m may be used for sewers 450 mm to 750 mm in diameter. Larger sewers may use greater manhole spacing.

Cleanouts may be used only with approval of the regulatory agencies and shall not be substituted for manholes nor installed at the end of laterals greater than 45 m in length.

2.5.2.2 Extended Spacing

For any particular municipality, the acceptable manhole spacing will vary depending upon the sewer cleaning equipment available. Some municipalities may allow longer spacing intervals. The above limits may, therefore, be exceeded provided the applicant can demonstrate the suitability of equipment available to handle such spacing.

2.5.3 Minimum Diameter

The minimum diameter of a sanitary manhole shall be 1050 mm.

2.5.4 Drop Manholes

A drop pipe should be provided for a sewer entering a manhole at an elevation of 600 mm or more above the manhole invert. Where the difference in elevation between the incoming sewer and the manhole invert is less than 600 mm the invert should be filleted to prevent solids deposition.

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.

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.

2.5.5 Manhole Bases

Precast bases may be used for manholes up to 9 m deep.

2.5.6 Pipe Connections

A flexible watertight joint shall be provided on all pipes, within 300 mm of the outside wall of the manhole.

2.5.7 Frost Lugs

Where required, frost lugs shall be provided to hold precast manhole sections together.

2.5.8 Frame and Cover

The manhole frame and cover shall be made of cast iron and designed to meet the following conditions:

a. adequate strength to support superimposed loads;

b. provision of a good fit between cover and frame to eliminate movement in traffic; and

c. a reasonably tight closure.

2.5.9 Watertightness

Manholes shall be of the pre-cast or poured-in-place concrete type, or of another type approved by the regulatory agencies. All manhole joints must be watertight and the manhole shall be waterproofed on the exterior, if required.

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

2.5.10 Flow Channel and Benching

The channel should be, as far as possible, a smooth continuation of the pipe. The completed channel should be U-shaped.

2.5.10.1 Small Pipe Channel

For sewer sizes less than 375 mm, the channel height should be at least one half the pipe diameter.

2.5.10.2 Large Pipe Channel

For sewer sizes 375 mm and larger, the channel height should not be less than three-fourths of the pipe diameter.

2.5.10.3 Bench Area

The bench should provide good footing for a workman and a place for tools and equipment.

2.5.10.4 Bench Slope

Benching should be at a slope of at least 1:12 (vertical:horizontal) and not greater than 1:8. Benching should have a wood float finish.

2.5.11 Corrosion Protection

Where corrosion is anticipated because of either sulphate attack or sulphides, consideration shall be given to the provision of corrosion resistant material or effective protective linings.

2.6 TESTING AND INSPECTION

2.6.1 General

Each section of a sanitary sewer shall be tested for exfiltration and/or infiltration. A section is the length of pipe between successive manholes or termination points, including service connections.

Each section of a sewer, and it's related appurtenances, shall be flushed prior to testing. The method of testing shall be as described in the construction specifications. In the absence of such specifications the following testing method will apply.

2.6.2 Exfiltration Test

Each sewer section shall be filled with water and a nominal head shall remain on the section for twenty-four hours immediately prior to testing.

Water shall be added to the section to establish a test head of 1.0 m over either the crown of the pipe, measured at the highest point of the section, or the level of static groundwater, whichever is greater. This may be increased by the inspector in order to satisfy local conditions.

The test head shall be maintained for one hour. The volume of water required to maintain the head during the test period shall be recorded.

2.6.3 Infiltration Test

Infiltration tests shall be conducted in lieu of exfiltration tests where the level of static groundwater is 750 mm or more above the crown of the pipe, measured at the highest point in the section.

A 90 degree V-notch weir shall be placed in the invert of the pipe at the downstream end of the section. The total volume of flow over the weir for one hour shall be measured and recorded.

2.6.4 Allowable Leakage

Allowable leakage shall be determined by the following formula:

$$L = F \times D \times \frac{S}{100}$$

where:

L = allowable leakage in litres per hour
D = diameter in mm
S = Length of section, in metres
F = leakage factor, (litres per hour per mm of diameter per 100 metres of sewer):

Exfiltration Test:

Porous Pipe	F = 0.12 litre
Non-Porous Pipe	F = 0.02 litre

Infiltration Test:

Porous Pipe	F = 0.10 litre
Non-Porous Pipe	F = 0.02 litre

2.6.5 Low Pressure Air Testing

Air testing equipment shall be designed to operate above ground. No personnel will be permitted in the trench during testing. Air testing will not be permitted on pipes with diameter greater than 600 mm.

The test section shall be filled with air until a constant pressure of 28 kPa is reached. After a two minute period the air supply shall be shut off, and the pressure decreased to 24 kPa. The time required for the pressure to reach 17 kPa shall be measured.

2.6.6 Allowable Time for Air Pressure Decrease

Minimum times allowed for air pressure drop are provided in the following table:

TABLE 2.2 – MINIMUM TIMES ALLOWED FOR PRESSURE DROP

Pipe Diameter (mm)	Minimum Time Min: Sec
100	1:53
150	2:50
200	3:47
250	4:43
300	5:40
375	7:05
450	8:30
525	9:55
600	11:20

2.6.7 Sewer Inspection

The specifications shall include a requirement for inspection of manholes and sewers for watertightness, prior to placing into service.

2.6.7.1 Video Inspection

Inspection on 100% of the sewer using the closed circuit television method and recorded on videotape should be specified. This should be conducted within the one-year guarantee period. This inspection should be carried out preferably during the periods of high ground water table in the spring or fall, or at the discretion of the regulatory agencies.

2.6.7.2 Inspection Record

The complete record of the inspection shall be the property of the owner or the municipality. The original videotape and one edited copy of the tape of the sections showing defects shall be turned over to the owner or municipality.

2.6.7.3 Record Content

The maximum speed of the television camera through the pipe shall be 0.30 meters per second with a 5-second minimum stop at each defective location and a 15 - second minimum stop at each lateral showing a flow discharging into the pipe. The audio part shall include the recording of distances at a maximum interval of three meters and a brief description of every defective location and of each service connection.

2.7 INVERTED SIPHONS

Inverted siphons should have not less than two barrels with a minimum pipe size of 150 mm and shall be provided with necessary appurtenances for convenient flushing and maintenance. The manholes shall have adequate clearances for rodding; and in general, sufficient head shall be provided and pipe sizes selected to secure velocities of at least 0.9 m/s for average flows. The inlet and outlet details shall be so arranged that the normal flow is diverted to one barrel and that either barrel may be cut out of service for cleaning. The vertical alignment should permit cleaning and maintenance.

2.8 PROTECTION OF WATER SUPPLIES

2.8.1 Water-Sewer Cross Connections

There shall be no physical connection between a public or private potable water supply system and a sewer, or appurtenance thereto which would permit the passage of any sewage or polluted water into the potable supply. No water pipe shall pass through or come in contact with any part of a sewer manhole, gravity sewer or sewage forcemain.

2.8.2 Relation to Water Works Structures

While no general statement can be made to cover all conditions, it is generally recognized that sewers shall be kept remote from public water supply wells or other water supply sources and structures.

2.8.3 Relation to Water Mains

2.8.3.1 Horizontal and Vertical Separation

Whenever possible, sewers should be laid at least three meters horizontally, from any existing or proposed water main. Should local conditions prevent a lateral separation of three meters a sewer may be laid closer than three meters to a water main if:

a. it is laid in a separate trench, or if;

b. it is laid in the same trench, with the water main located at one side with a minimum horizontal separation of 300 mm and on a bench of undisturbed earth and if;

c. in either case the elevation of the top (crown) of the sewer is at least 300 mm below the bottom (invert) of the water main.

d. Where a water main must be installed paralleling a gravity sewer and at a lower elevation than the gravity sewer, the water main must be installed in a separate trench. The soil between the trenches must be undisturbed.

2.8.3.2 Crossings

Whenever sewers must cross under the water mains, the sewer shall be laid at such an elevation that the top of the sewer is at least 450 mm below the bottom of the water main. When the elevation of the sewer cannot be varied to meet the above requirement, the water main shall be relocated to provide this separation or reconstructed with mechanical - joint pipe for a distance of three meters on each side of the sewer. One full length of water main should be centered over the sewer so that both joints will be as far from the sewer as possible.

2.8.3.3 Special Conditions

When it is impossible to obtain proper horizontal and vertical separation as stipulated above, the sewer shall be designed and constructed equal to water pipe and shall be pressure-tested to assure water-tightness.

2.9 SEWERS IN RELATION TO STREAMS

2.9.1 Location of Sewers on Streams

2.9.1.1 Cover Depth

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

a. 0.3 m of cover is required where the sewer is located in rock;

b. 2 meters of cover is required in other material.

c. in paved stream channels, the top of the sewer line should be placed below the bottom of the channel pavement.

Less cover will be approved only if the proposed sewer crossing will not interfere with the future improvements to the stream channel. Reasons for requesting less cover should be given in the project proposal.

2.9.1.2 Horizontal Location

Sewers located along streams shall be located outside of the stream bed and sufficiently remote therefrom to provide for future possible stream widening and to prevent pollution by siltation during construction.

2.9.1.3 Structures

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

2.9.1.4 Alignment

Sewers crossing streams should be designed to cross the stream as nearly perpendicular to the stream flow as possible and shall be free from change in grade. Sewer systems shall be designed to minimize the number of stream crossings.

2.9.2 Construction

2.9.2.1 Materials

Sewers entering or crossing streams shall be constructed of cast or ductile iron pipe with mechanical joints; otherwise they shall be constructed so they will remain watertight and free from changes in alignment or grade. Material used to backfill the trench shall be stone, coarse aggregate, washed gravel or other materials which will not cause siltation.

2.9.2.2 Siltation and Erosion

Construction methods that will minimize siltation and erosion shall be employed. The design engineer shall include in the project specifications the method(s) to be employed in the construction of sewers in or near streams to provide adequate control of siltation and erosion. Specifications shall require that cleanup, grading, seeding and planting or restoration of all work areas shall begin immediately. Exposed areas shall not remain unprotected for more than seven days.

2.10 AERIAL CROSSINGS

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

Precautions against freezing, such as insulation and increased slopes shall be provided. Expansion jointing shall be provided between above-ground and below-ground sewers.

For aerial stream crossings the impact of flood waters and debris shall be considered. The bottom of the pipe shall be placed no lower than the elevation of the fifty (50) year flood.

2.11 ALTERNATIVE WASTEWATER COLLECTION SYSTEMS

2.11.1 Applications

Under a certain set of circumstances, each alternative system has individual characteristics which may dictate standards for usage. Each potential application should be analyzed to determine which system is most cost effective and which will comply with local requirements. The following features of various sewerage alternatives are considered in planning a project.

2.11.1.1 Population Density

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

2.11.1.2 Ground Slopes

Where the ground profile over the main slopes continuously downward in the direction of flow, conventional or small diameter gravity sewers are normally preferred. If intermittent rises in the profile occur, conventional sewers may become cost-prohibitively deep. The variable grade gravity sewer variation of small diameter gravity sewers, by use of inflective gradients and in conjunction with septic tank effluent pump (STEP) pressure sewer connections, can be economically applied. Vacuum sewers may be particularly adaptable to this topographic condition, so long as head requirements are within the limits of available vacuum.

In flat terrain conventional sewers become deep due to the continuous downward slope of the main, requiring frequent use of lift stations. Both the deep excavation and the lift stations are expensive. Small Diameter Gravity Sewers (SDGS) are buried less deep, owing to the flatter gradients permitted. Pressure sewers or vacuum sewers are often found to be practical in flat areas, as ground slope is of little concern. In areas where the treatment facility or interceptor sewer are higher than the service population, pressure sewers and vacuum sewers are generally preferred, but should be evaluated against SDGS systems with lift stations.

2.11.1.3 Subsurface Obstacles

Where rock excavation is encountered, the shallow burial depth of alternative sewer mains reduces the amount of rock to be excavated.

Deep excavations required of conventional sewers sometimes encounter groundwater. Depending on severity, dewatering can be expensive and difficult to accomplish.

2.11.2 Pressure Sewer Systems

Pressure sewers are small diameter pipelines, buried just below frost level, which follow the profile of the ground. Each home connected to the pipeline requires either a grinder pump (GP) or a STEP. Main diameters typically range from 50 - 150 mm with service lateral diameters of 25 - 38 mm. Polyvinyl Chloride (PVC) is the most common piping material.

2.11.2.1 System Layout

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

a. Branched layout rather than gridded or looped.

b. Maintain cleansing velocities especially when grinder pump type pressure sewers are used.

c. Minimize high head pumping and downhill flow conditions.

d. Locate on lot facilities close to the home for ease of maintenance.

e. Provide for each home to have its own tank and pump.

2.11.3 Vacuum Sewer Systems

Vacuum sewer systems consist of a vacuum station, collection piping, wastewater holding tanks, and valve pits. In these systems, wastewater from an individual building flows by gravity to the location of the vacuum ejector valve. The valve seal the line leading to the main in order to maintain required vacuum levels. When a given amount of wastewater accumulates behind the valve, the valve opens and then closes allowing a liquid plug to enter the line. Vacuum pumps in a central location maintain the vacuum in the system.

2.11.3.1 Services

Each home on the system should have its own holding tank and vacuum ejector valve. Holding tank volume is usually 115 L. As the wastewater level rises in the sump, air is compressed in a sensor tube which is connected to the valve controller. At a preset point, the sensor signals for the vacuum valve to open. The valve stays open for an adjustable period of time and then closes. During the open cycle, the holding tank contents are evacuated. The timing cycle is field adjusted between 3 and 30 seconds. This time is usually set to hold the valve open for a total time equal to twice the time required to admit the wastewater. In this manner, air at atmospheric pressure is allowed to enter the system behind the wastewater. The time setting is dependent on the valve location since the vacuum available will vary throughout the system, thereby governing the rate of wastewater flow.

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

2.11.3.2 Collection Piping

The vacuum collection piping usually consists of 100 mm and 150 mm mains. Smaller 75 mm mains are not recommended as the cost savings of 75 mm versus 100 mm mains are considered to be insignificant.

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

2.11.3.3 Vacuum Station

Vacuum stations are typically two-storey concrete and block buildings approximately 7.5 m x 9 m in floor plan. Equipment in the station includes a collection tank, a vacuum reservoir tank, vacuum pumps, wastewater pumps, and pump controls. In addition, an emergency generator is standard equipment, whether it is located within the station, outside the station in an enclosure, or is of the portable, truck mounted variety.

The collection tank is made of either steel or fibreglass. The vacuum reservoir tank is connected directly to the collection tank to prevent droplet carryover and to reduce the frequency of vacuum pump starts. Vacuum pumps can be either liquid ring or sliding vane type and are sized for a 3 - 5 hr/d run-time. The wastewater discharge pumps are non-clog pumps with sufficient net positive suction head to overcome tank vacuum. Level control probes are installed in the collection tank to regulate the wastewater pumps. A fault monitoring system alerts the system operator should a low vacuum or high wastewater level condition occur.

2.11.4 Small Diameter Gravity Sewers

Small diameter gravity sewers (SDGS) require preliminary treatment through the use interceptor or septic tanks upstream of each connection. With the solids removed, the collector mains need not be designed to carry solids as conventional sewers must be. Collector mains are smaller in diameter and laid with variable or inflective gradients. Fewer manholes are used and most are replaced with cleanouts except at major junctions to limit infiltration/inflow and entry of grit. The required size and shape of the mains is dictated primarily by hydraulics rather than solids carrying capabilities.

2.11.4.1 House Connections

House connections are made at the inlet to the interceptor tank. All household wastewaters enter the system at this point.

2.11.4.2 Interceptor Tanks

Interceptor tanks are buried, watertight tanks with baffled inlets and outlets. They are designed to remove both floating and settleable solids from the waste stream through quiescent settling over a period of 12-24 hours. Ample volume is provided for storage of the solids which must be periodically removed through an access port. Typically, a single-chamber septic tank, vented through the house plumbing stack vent, is used as an interceptor tank.

2.11.4.3 Service Laterals

Service Laterals connect the interceptor tank with the collector main. Typically, they are 75-100 mm in diameter, but should be no larger than the collector main to which they are connected. They may include a check valve or other backflow prevention device near the connection to the main.

2.11.4.4 Collector Mains

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

2.11.4.5 Cleanouts, Manholes, and Vents

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

2.11.4.6 Lift Stations

Lift stations are necessary where the elevation differences do not permit gravity flow. Either STEP units (see 2.11.1) or mainline lift stations may be used. STEP units are small lift stations installed to pump wastewater from one or a small cluster of connections to the collector main, while a mainline lift station is used to service all connections in a larger drainage basin.

2.11.5 Detailed Design Guidelines

The above are general design considerations only. For detailed design refer to:

Alternative Sewer Systems, Manual of Practice no. FD-12, Facilities Development, Water Pollution Control Federation, Alexandria, VA, 1986.

U.S. Environmental Protection Agency: Manual: *Alternative Wastewater Collection Systems*, EPA-625/1-91/024, Office of Research and Development, Washington, DC,1991.

Chapter 3 SEWAGE PUMPING STATIONS

3.1 GENERAL

3.1.1 Location

Sewage pumping station structures and electrical and mechanical equipment shall be protected from physical damage from the one hundred (100) year flood. Sewage pumping stations should remain fully operational and accessible during the twenty-five (25) year flood.

During preliminary location planning, consideration should be given to the potential of emergency overflow provisions and as much as practically possible the avoidance of health hazards, nuisances and adverse environmental effects.

3.1.2 Design Capacity

3.1.2.1 Separate Sewer Systems

Sewage pumping stations should be able to pump the expected twenty-five year peak sewage flows, under normal growth conditions, with the largest capacity pump out of operation. See Section 2.3 for the recommended approach for the calculation of peak sewage flows. In certain cases, where it can be shown that staging of construction will be economically advantageous, lesser design periods may be used provided it can be demonstrated that the required capacity can be "on-line" when needed. Pumping station overflows shall be permitted under the requirements of Section 3.3.

If only two pumps are provided, they should have the same capacity. Each shall be capable of handling the expected peak sewage flow. Where three or more units are provided, they should be designed to fit actual flow conditions and must be of such capacity that with any one unit out of service the remaining units will have capacity to handle maximum sewage flows, taking into account head losses associated with parallel operation.

3.1.2.2 Combined Sewer Systems

It may be impractical or economical to design a sewage pumping station on a combined sewer system to pump the expected twenty-five year peak sewage flow, with the largest capacity pump out of operation. Under these conditions the following shall be considered in determining the appropriate design capacity:

a) the minimization of combined sewer overflows.

b) the minimization of pumping station overflows as outlined in Section 3.3.

3.1.3 Accessibility

Sewage pumping stations shall be readily accessible by maintenance vehicles during all weather conditions. The facility should be located off the traffic way of streets and alleys.

3.1.4 Grit

Where it may be necessary to pump sewage prior to grit removal, the design of the wet wells should receive special attention and the discharge piping shall be designed to prevent grit settling in pump discharge lines of pumps not operating.

3.1.5 Sewer Entry

If more than one sewer enters the site of the pumping station, a junction manhole should be provided so that only a single sewer entry to the wet well is required.

3.1.6 Fencing

All above ground pumping stations and associated facilities may require fencing. The fence shall have an opening gate for entry of vehicles and equipment, and the gate shall be kept locked to prevent vandalism.

3.1.7 Heating

Automatic heating may be required at pumping stations, to prevent freezing in cold weather and to maintain a comfortable working temperature (there may be exceptions in the case of small below ground wet well or manhole type lift stations).

3.1.8 Piping System

The design of the pumping and piping systems should account for the potential of surge, water hammer, and special requirements for pump seals associated with wastewater service.

Suction and discharge piping should be sized to accommodate expected peak hourly flows with velocities ranging from 0.8 m/s to 2.0 m/s, where feasible velocities at the low end of the range are preferable. Consideration should be given to providing access ports for such things as sampling, swabbing, and/or flushing discharge pressure gauge(s).

3.1.9 Electrical

All wiring shall be in accordance with the requirements of the Canadian Electrical Code and the local inspection authority.

Adequate heating should be intalled to provide a minimum ambient temperature of 15 ^{o}C to permit the provision of dehumidification equipment in the dry side of wet well/dry well pumping stations.

3.1.10 Lighting

Ligthing levels should be provided in accordance with IES (Illuminating Engineering Society) recommended practice for similar area and use classifications.

3.1.11 Safety

The design and construction of all components of wastewater pumping stations shall conform to the safety provisions of the Occupational Health and Safety and Construction Safety Legislation in the region where the pumping station is located.

3.1.12 Construction Materials

Due consideration shall be given to the selection of materials and equipment because of the presence of hydrogen sulphide and other corrosive and inflammable gases, greases, oils and other constituents present in sewage.

3.2 DESIGN

3.2.1 Types of Pumping Systems

The type of sewage pumping station should be selected on the basis of such considerations as reliability and serviceability; operation and maintenance factors; relationship to existing stations/equipment; sewage characteristics; flow patterns and discharge; and long-term capital, operating and maintenance costs.

For large main pumping stations, wet well/dry well type stations are recommended. For smaller stations and in cases for which wet well/dry well types are not feasible, wet well (submersible) pump stations may be used if pumps can be easily removed for replacement or repairs.

3.2.2 Structures

3.2.2.1 Separation

Wet and dry wells including their superstructure shall be completely separated.

3.2.2.2 Equipment Removal

Provision shall be made to facilitate removing pumps, motors and other mechanical and electrical equipment.

3.2.2.3 Access

Suitable and safe means of access shall be provided to dry wells of pump stations and to wet wells or to other parts of the building containing bar screens or mechanical equipment requiring inspection or maintenance. Stairways should be installed, with rest landings not to exceed 3 m vertical intervals.

3.2.3 Pumps and Pneumatic Injectors

3.2.3.1 Duplicate Units

At least two pumps or pneumatic ejectors shall be provided. A minimum of three pumps should be provided for stations handling flows greater than 4500 m^3/d.

3.2.3.2 Protection Against Clogging

The need for and the type of screening facilities required for pumping stations varies with the characteristics of the sewage, size of sewers and the requirements of the operating authority. For wet well/dry well stations, it is generally accepted practice to provide screening in the form of a basket screen or a manually or mechanically cleaned bar screen. Although basket screens may be cumbersome to remove and empty, they have the advantage of not requiring entry of operating staff into the wet well for cleaning operations. With basket screens, guide rails should be tubular and similar to submersible pump guide rails. Manually cleaned bar screens should be provided with 38 mm clear openings in the inclined (60°) and horizontal bars. The vertical sides should be solid. The minimum width should be 600 mm. A drain platform should be provided for screenings. Pumps handling separate sanitary sewage from 750 mm or larger diameter sewers shall be protected by bar screens meeting the above requirements.

3.2.3.3 Pump Openings

Pumps shall be capable of passing spheres of at least 75 mm in diameter. Pump suction and discharge openings shall be at least 100 mm in diameter.

3.2.3.4 Priming

The pump shall be so placed that under normal operating conditions it will operate under a positive suction head, except as specified in Section 3.2.10.

3.2.3.5 Electrical Equipment

The wet wells of sewage pumping stations may occasionally contain flammable mixtures presenting a potentially hazardous (explosive) environment. As a minimum, electrical installations in these areas should comply with the requirements of the Canadian Electrical Code, Class 1 Zone 2 Hazardous areas, or as otherwise required by the local inspection authority.

3.2.3.6 Intake

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

3.2.3.7 Constant Speed vs. Variable Speed Pumps

In certain instances, such as pumping stations discharging directly into mechanical sewage treatment plants or into other pumping stations, some means of flow pacing may be required. This can be provided by various means, depending upon the degree of flow pacing necessary. If even minor pressure transients caused by pump starting and stopping would have serious effects, solid state, soft start and stop motor starters should be considered. Where flow surges to treatment plants may be detrimental to the treatment process, variable speed control drives should be considered. If minor surges can be tolerated, two-speed pumps or multiple constant speed pumps can be used.

3.2.3.8 Controls

Control systems shall be of the air bubbler type or the encapsulated float type. Where PLC (Programmable logic Controllers) form the basis of the station control system, consideration should be given to continuous level measurement via ultrasonic or submersible level transmitters. Pump control setpoints are derived from the analog level signal in the PLC. For this type of installation, emergency start and stop float switches should be included to maintain station operation in the event of instrument failure.

Level control devices located in the station wet wells are to be designed as Intrinsically Safe systems. Float control should be positioned as per Section 3.2.5.5.

3.2.3.9 Alternation

Provisions shall be made to automatically alternate the pumps in use. In the event of pump failure, the alternat pump shall operate as the lead pump.

3.2.4 Valves

3.2.4.1 Suction Line

Suitable shutoff valves shall be placed on the suction line of each pump except on submersible and vacuum-primed pumps.

3.2.4.2 Discharge Line

Suitable shutoff and check valves shall be placed on the discharge line of each pump. The check valve shall be located between the shutoff valve and the pump. Check valves shall be suitable for the material being handled. Valves shall be capable of withstanding normal pressure and water hammer.

Where limited pump backspin will not damage the pump and low discharge head conditions exist, short individual force mains for each pump may be considered in lieu of discharge valves.

3.2.4.3 Location

Valves shall not be located in the wet well.

3.2.5 Wet Wells

3.2.5.1 Divided Wells

Where continuity of pumping station operation is important, consideration should be given to dividing the wet well into multiple sections, properly interconnected, to facilitate repairs and cleaning. Divided wet wells should also be considered for all pumping stations with capacities in excess of 100 L/s.

3.2.5.2 Pump Cycle

For any pumping station, the wet well should be of sufficient size to allow for a minimum of a fifteen minute cycle time for each pump. For a two-pump station, the volume in cubic meters, between pump start and pump stop should be 0.225 times the pumping rate of one pump, expressed in L/s. For other numbers of pumps, the required volume depends upon the operating mode of the pumping units. Maximum recommended starts per hour are 6 for dry pit motors and 12 for submersible motors.

3.2.5.3 Size

Wet well size and control settings should be based on consideration of the volume required for pump cycling; the design fill time, dimensional requirements to avoid turbulence problems; vertical separation between pump control points; inlet sewer elevation; capacity required between alarm levels and basement flooding and/or overflow elevations; etc. To avoid septicity problems, wet wells should not provide retention times greater than 30 minutes.

3.2.5.4 Floor Slope

The wet well floor shall have a minimum slope of 1 to 1 to the hopper bottom. The horizontal area of the hopper bottom shall be no greater than necessary for proper installation and function of the inlet.

3.2.5.5 Float Controls

Float controls should be at least 300 mm vertically and 125 mm horizontally apart and positioned against a wall away from turbulent areas.

3.2.5.6 Pump Start Elevation

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

3.2.5.7 Pump Stop Elevation

Low water level (pump shut-down) should be at least 300 mm or twice the pump suction diameter, whichever is greater, above the center line of the pump volute.

3.2.5.8 Bottom Elevation

The bottom of the wet well should be no more than D/2, nor less than D/3 below the mouth of the flared intake where turned-down, bell-mouth inlets are used. "D" being the diameter of the mouth of the flared intake.

3.2.5.9 Air Displacement

Covered wet wells shall have provisions for air displacement such as an inverted "j" tube or other means which vents to the outside.

3.2.6 Dry Wells

3.2.6.1 Dry Well Dewatering

A separate sump pump equipped with dual check valves shall be provided in the dry wells to remove leakage or drainage, with the discharge above the overflow level of the wet well. A connection to the pump suction is also recommended as an auxiliary feature. Water ejectors connected to a potable water supply will not be approved. All floor and walkway surfaces should have an adequate slope to a point of drainage. Pump seal water shall be piped to the sump.

3.2.6.2 Maintenance

The dry well should be equipped with a lifting beam to facilitate removal of pump motors. A roof hatch is recommended to provide access for removal of the entire pump and motor.

3.2.7 Ventilation

3.2.7.1 General

Adequate ventilation shall be provided for all pump stations. Where the pump pit is below the ground surface, mechanical ventilation is required, so arranged as to independently ventilate the dry well and the wet well. There shall be no interconnection between the wet well and dry well ventilation systems. Ventilation must avoid dispensing contaminants throughout other parts of the pumping station, and vents shall not open into a building or connect with a building ventilation system.

3.2.7.2 Air Inlets and Outlets

In dry wells over 4.6 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.

3.2.7.3 Electrical Controls

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. Consideration should be given also to automatic controls where intermittent operations is used. The manual lighting ventilation switch shall override the automatic controls.

3.2.7.4 Fans, Heating, and Dehumidification

The fan wheels shall be fabricated from non-sparking material. Automatic heating and dehumidification equipment shall be provided in all dry wells. The electrical equipment and components shall meet the requirements in Section 3.2.3.5.

3.2.7.5 Wet Wells

Ventilation may be either continuous or intermittent. Ventilation, if continuous, should provide at least 12 complete air changes per hour; if intermittent, at least 30 complete air changes per hour. Fresh air shall be forced into the wet well, by mechanical means, at a point 300 mm above the expected high liquid level. There shall be a provision for automatic blow-by to elsewhere in the well, should the fresh air outlet become submerged. Portable ventilation equipment shall be provided for use at submersible pump stations.

3.2.7.6 Dry Wells

Ventilation may be either continuous or intermittent. Ventilation, if continuous, should provide at least six complete air changes per hour; if intermittent, at least 30 complete air changes per hour. Ventilation shall be forced into the dry well at a point 150 mm above the pump floor, and allowed to escape through vents in the roof superstructure. A system of two speed ventilation with a initial ventilation rate of 30 changes per hour for 10 minutes and automatic change over to 6 changes per hour may be used to conserve heat.

3.2.7.7 Flow Measurement

Suitable devices for measuring wastewater flow shall be provided at all pumping stations. Indicating, totalizing, and recording flow measurement shall be provided at pumping stations with a 75 L/s or greater design peak hourly flow. Elapsed time meters used in conjunction with pumping rate tests may be acceptable for pumping stations with a design peak hourly flow up to 75 L/s.

3.2.8 Water Supply

There shall be no physical connection between any potable water supply and a sewage 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 shall be protected with a suitable backflow prevention device (see Section 4.7.2).

3.2.9 Suction Lift Pumps

3.2.9.1 General

Suction lift pumps shall be of the self-priming or vacuum-priming type and shall meet the applicable requirements of Section 3.2. Suction lift pump stations using dynamic suction lifts exceeding the limits outlined in the following sections may be approved by the appropriate reviewing agency upon submission of factory certification of pump performance and detailed calculations indicating satisfactory performance under the proposed operating conditions. Such detailed calculations must include static suction lift as measured from "lead pump off" elevation to center line of pump suction, friction and other hydraulic losses of the suction piping, vapor pressure of the liquid, altitude correction, required net positive suction head and a safety factor of at least 1.8 meters.

The pump equipment compartment shall 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. Wet well access shall not be through the equipment compartment. Valving shall not be located in the wet well.

3.2.9.2 *Self-Priming Pumps*

Self-priming pumps shall be capable of rapid priming and re-priming at the "lead pump on" elevation. Such self-priming and re-priming shall be accomplished automatically under design operating conditions. Suction piping should not exceed the size of the pump suction and shall not exceed 7.6 m in total length. Priming lift at the "lead pump on" elevation shall 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 shall not exceed 6.7 m.

3.2.9.3 *Vacuum-Priming Pumps*

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

3.2.10 Submersible Pump Stations

3.2.10.1 *General*

Submersible pump stations shall meet the applicable requirements under Sections 3.2.1 to 3.2.9 except as modified in this section.

3.2.10.2 *Construction*

Submersible pumps and motors shall be designed specifically for raw sewage use, including totally submerged operation during a portion of each pumping cycle. An effective method to detect shaft seal failure or potential seal failure shall be provided and the motor shall be of squirrel-cage type design without brushes or other arc-producing mechanisms.

3.2.10.3 *Pump Removal*

Submersible pumps shall be readily removable and replaceable without dewatering the wet well or disconnecting any piping in the wet well.

3.2.10.4 *Power Supply*

Pump power cables, control and alarm circuits shall be designed to provide strain relief and to allow disconnection from outside the wet well. Cable terminations shall be made outside the wet well in enclosures suitably rated for the ambient environment.

3.2.10.5 *Controls*

The pump controller shall be located outside the wet well. Conduit sealing is required at the entry to field junction boxes or pump controllers and shall be in accordance with the specific requirements of the Inspection Authority. If conventional conduit EY type seal fittings are utilized, they shall be located such that the pump power and/or control cables can be removed and electrically disconnected without disturbing the seal.

3.2.10.6 Power Cables

Pump motor cables shall be designed for flexibility and serviceability under conditions of extra hard usage and shall meet the requirements of the Canadian Electrical Code. The ground fault system shall be used to de-energize the circuit in the event of any failure in the electrical integrity of the cable.

3.2.10.7 Valves

Valves required under Section 3.2.4 shall be located in a separate valve pit. Accumulated water shall be drained to the wet well or to the soil. If the valve pit is drained to the wet well, an effective method shall be provided to prevent sewage from entering the pit during surcharged wet well conditions.

3.2.10.8 Ventilation

Gravity ventilation may be acceptable provided that maintenance crews carry suitable portable ventilation equipment when visiting the site.

3.2.11 Cathodic Protection

Steel fabricated pumping stations shall require cathodic protection for corrosion control. Impressed current or magnesium anode packs are generally used for this purpose in conjunction with a suitable protective coating on underground surfaces, applied in accordance with the manufacturer's directions. The unit should be electrically isolated by dielectric fittings placed on inlet and outlet pipes, anchor bolts and electrical conduit boxes.

Upon completion of the installation, the capability of the anti-corrosion system should be verified by instrumentation. Such inspection should be carried out by a person approved by the reviewing agencies.

3.2.12 Alarm Systems

Alarm systems shall be provided for pumping stations. The alarm shall be activated in cases of power failure, pump failure, use of the lag pump, unauthorized entry, or any cause of pump station malfunction. Pumping station alarms shall be telemetered, including identification of the alarm condition, to a municipal facility that is manned 24 hours a day. If such a facility is not available and 24 hour holding capacity is not provided, the alarm may be telemetered to municipal offices during normal working hours or to the home of the person(s) in charge of the pumping station during off-duty hours. Audio visual alarm systems with a self-contained power supply may be acceptable in some cases in lieu of the telemetering system outlined above, depending upon location, station holding capacity and inspection frequency.

3.3 EMERGENCY OPERATION

Pumping stations and collection systems shall be designed to prevent or minimize bypassing of raw sewage. For use during periods of extensive power outages, mandatory power reductions, or uncontrolled storm events, 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 sewage 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 shall also be given to the installation of storage/detention tanks, or basins, which shall be made to drain to the station wet well. Storage capacity should be related to frequency and length of power outages for the area. Power outage history can be obtained from the power supply company for the grid where the pump station is to be located. This data can then be utilized with peak design flows for design of storage facilities to minimize overflow. Where elimination of overflows is not practical this data can be used to predict frequency and quantities of overflows. Where public water supplies, shellfish production, or waters used for culinary or food processing purposes exist, overflows shall not be permitted.

3.3.1 Overflow Prevention Methods

A satisfactory method shall be provided to prevent or minimize overflows. The following methods should be evaluated on a case by case basis:

a. storage capacity, including trunk sewers, for retention of wet weather flows (storage basins must be designed to drain back into the wet well or collection system after the flow recedes); and

b. an in-place or portable pump, driven by an internal combustion engine meeting the requirements of Section 3.3.3 below, capable of pumping from the wet well to the discharge side of the station.

3.3.2 Overflows

If the avoidance of overflows is not possible, provision shall be made for chlorination of the overflow raw sewage unless waived by the regulatory agencies. The overflow facilities should be alarmed and equipped to indicate frequency and duration of overflows, and designed to permit manual flow measurement. Overflows should be recorded and reported to the regulatory agencies.

3.3.3 Equipment Requirements

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

3.3.3.1 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 shall be capable of shutting down the engine and activating an alarm on site and as provided in Section 3.2.13. Protective equipment shall monitor for conditions of low oil pressure and overheating, except that oil pressure monitoring will not be required for engines with splash lubrication.

3.3.3.2 Size

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

3.3.3.3 Fuel Type

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

3.3.3.4 Engine Ventilation

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

3.3.3.5 *Routine Start-up*

All emergency equipment shall be provided with instructions indicating the need for regular starting and running of such units at full loads.

3.3.3.6 *Protection of Equipment*

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

3.3.4 Engine-Driven Pumping Equipment

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

3.3.4.1 *Pumping Capacity*

Engine-driven pumps shall meet the design pumping requirements unless storage capacity is available for flows in excess of pump capacity. Pumps shall be designed for anticipated operating conditions, including suction lift if applicable.

3.3.4.2 *Operation*

The engine and pump shall be equipped to provide automatic start-up and operation of pumping equipment. Provisions shall also be made for manual start-up. Where manual start-up and operation is justified, storage capacity and alarm systems must meet the requirements of Section 3.3.4.3.

3.3.4.3 *Portable Pumping Equipment*

Where part or all of the engine-driven pumping equipment is portable, sufficient storage capacity to allow time for detection of pump station failure and transportation and hookup of the portable equipment shall be provided. A riser from the force main with quick-connect coupling and appropriate valving shall be provided to hook up portable pumps.

3.3.5 Engine-Driven Generating Equipment

Where permanently-installed or portable engine-driven generating equipment is used, the following requirements in addition to general requirements shall apply.

3.3.5.1 *Generating Capacity*

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

3.3.5.2 *Operation*

Provisions shall be made for automatic and manual start-up and load transfer. The generator must 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 systems must meet requirements of Section 3.3.4.3

3.3.5.3 Portable Generating Equipment

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

3.4 INSTRUCTIONS AND EQUIPMENT

The operating authority of sewage pumping stations shall be supplied with a complete set of operational instructions, including emergency procedures, maintenance schedules, tools and such spare parts as may be necessary.

3.5 FORCE MAINS

3.5.1 Velocity

At design average flow, a cleansing velocity of at least 0.6 meters per second shall be maintained.

3.5.2 Air Relief Valve and Blowoff

An automatic air relief valve shall be placed at high points in the force main to prevent air locking. Blowoffs should be provided at all low points in pressure sewers.

3.5.3 Termination

Force mains should enter the gravity sewer system at a point not more than 0.6 m above the flow line of the receiving manhole. A 45° bend may be considered to direct the flow downward.

3.5.4 Design Pressure

The force main and fittings, including reaction blocking, shall be designed to withstand normal pressure and pressure surges (water hammer).

3.5.5 Size

Force mains shall be sized to provide sufficient flow velocity, required capacity at the available head and to withstand operating pressures as outlined in Sections 3.5.1 and 3.5.4.

3.5.6 Slope and Depth

Force main slope does not significantly affect the hydraulic design or capacity of the pipeline itself. Under no circumstance, however, shall any force main be installed at zero slope. Zero slope installation makes line filling and pressure testing difficult, and promotes accumulation of air and wastewater gases.

A forcemain should have a minimum cover of 1.8 m.

3.5.7 Special Construction

Force main construction near watercourses or used for aerial crossing shall meet applicable requirements of Sections 2.9 and 2.10.

3.5.8 Design Friction Losses

Friction losses through force mains shall be based on the Hazen Williams formula or another acceptable method. When the Hazen Williams formula is used, the following values for "C" shall be used for design.

Unlined iron or steel	-	100
All other	-	120

When initially installed, force mains will have a significantly higher "C" factor. The "C" factor of 120 should be considered in calculating maximum power requirements for smooth pipe.

3.5.9 Separation from Water Mains

Water mains and sewage force mains are to be installed in separate trenches. The soil between the trenches shall be undisturbed. Force mains crossing water mains shall be laid to provide a minimum vertical distance of 450 mm between the outside of the force main and the outside of the water main. The water main shall be above the force main. At crossings, one full length of water pipe shall be located so both joints will be as far from the force main as possible. Special structural support for the water main and the force main may be required.

3.5.10 Identification

Where force mains are constructed of material which might cause the force main to be confused with potable water mains, the force main should be appropriately identified.

3.6 TESTING

3.6.1 General

The entire length of a force main shall be tested for leakage. If the length of a force main exceeds 400 m, the allowable leakage must not exceed the allowable leakage for a similar force main 400 m in length. All valves in the force main must be opened immediately prior to testing.

3.6.2 Leakage Test

The force main shall be filled with water, and a test pressure of 1035 kPa or equal to 1.5 times the working pressure shall be applied, measured at the lowest point in the test section. The pressure shall be maintained by pumping water from a suitable container of known volume. The amount of water used for a period of two hours shall be recorded.

3.6.3 Allowable Leakage

Allowable leakage for a force main shall be determined by the following formula:

$$L = \frac{(SD) \times P^{0.5}}{727,500}$$

where:

L = allowable leakage in litres/hour
S = length of pipe in metres
D = nominal diameter of pipe in mm

P = test pressure in kPa

Allowable leakage for closed metal seated valves is 1.2 mL per mm of nominal valve diameter per hour.

Chapter 4 SEWAGE TREATMENT FACILITY

4.1 PERFORMANCE EXPECTATIONS

Treatment, the extent of which will depend upon local conditions, shall be provided in connection with all sewer installations. The engineer should confer with the regulatory agencies before proceeding with the design of detailed plans.

4.1.1 Preliminary Treatment

Coarse screens, bar screens and comminutors are generally provided so as to protect downstream pumps and other equipment from damage caused by the movement of large solids and trash.

Usually, grit chambers remove all particles that have settling velocities (in a quiescent settling column) greater than 1.5 to 3 cm/sec at 20°C. Sand particles of specific gravity 2.65 and size 0.2 mm (retained on a 65-mesh screen) are known to have a settling velocity of approximately 3 cm/sec and grit chambers are designed to remove all sand (and gravel) particles of size greater than 0.2 mm. However, particles of specific gravity lower than 2.65 can also have settling velocity greater than 3 cm/sec when they are of very coarse size, and such solids are also removed in a grit chamber whether they are inorganic or organic. Design engineers have no control over this. The only controlling factor is the settling (or subsiding) velocity of a solid particle and this depends on particle size as well as specific gravity (at a given temperature). Consequently, particles collected even in properly designed and operated grit chambers have been known to have wide ranges of specific gravity, size, shape, character and organic content.

Pre-aeration of wastewater can be used to achieve the following objectives:

a. odour control;

b. grease separation and increased grit removal;

c. prevention of septicity;

d. grit separation;

e. flocculation of solids;

f. maintenance of dissolved oxygen (D.O.) in primary treatment tanks at low flows;

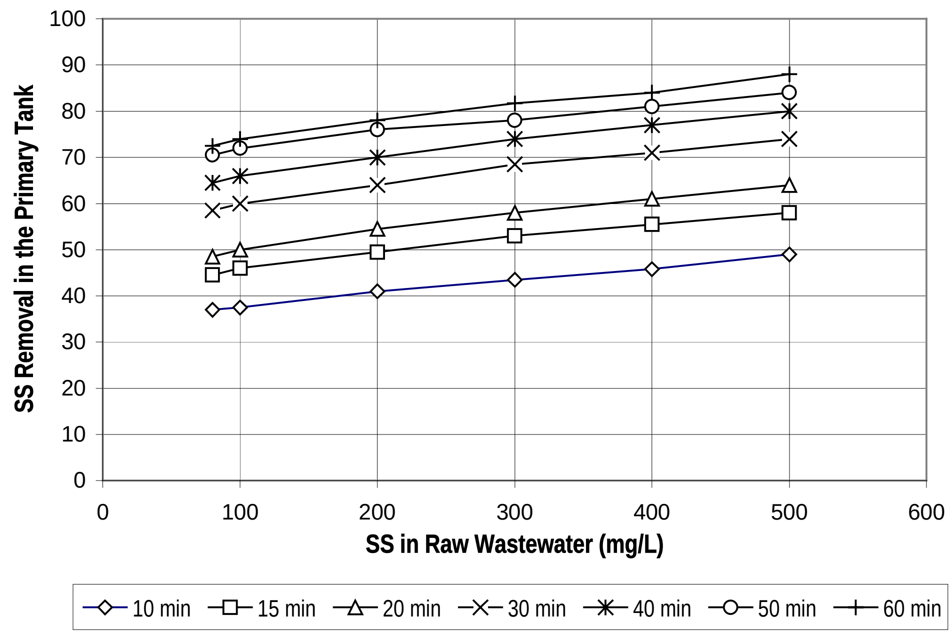
g. increased removal of BOD and SS in primary units (see Figure 4.1); and

h. to minimize solids deposits on side walls and bottom of wetwells.

Pre-chlorination (the practice of applying chlorine at the plant headworks) is used principally to control odour, corrosion and septicity, and to aid in grease removal. The provision to pre-chlorinate influent wastewater should be provided for all wastewater treatment plants.

Figure 4.1

Effect of Preaeration on Removal of SS in a Primary Settling Tank



Note: Removal based on a 2 hr retention time in the settling basins

4.1.2 Primary, Secondary and Tertiary Treatment

Table 4.1 lists expected effluent quality produced by well operated treatment facilities treating typical municipal sanitary sewage. The table can be used to illustrate potential effluent quality for selected processes, and as a guide for performance comparisons. Specific facilities may have different treatment objectives and quality requirements.

TABLE 4.1 – SEWAGE TREATMENT PROCESS TYPICAL EFFLUENT QUALITY

PROCESS	BOD_5 mg/L	TSS mg/L	Total P, mg/L	Total N, mg/L
Primary (including anaerobic lagoons)	75-150	50-110	5-5	25-45
- With P Removal	45-85	25-50	1-2	20-40
Secondary				
- Biological (Mech.)	10-25	10-25	3.5-6.5	15-35
- Aerated Lagoons	15-30	20-35	4-7	20-40
- Facultative Lagoons - Winter to Late Spring - Summer to Late Fall	25-70 10-30	20-60 10-40	3.5-7 2-5	20-35 5-10
Advanced				
- Secondary with chemical treatment (P control)	5-15	10-30	0.5-1.5	15-35

4.2 SITE CONSIDERATIONS

4.2.1 Plant Location

The following items shall be considered when selecting a plant site:

a. proximity to residential areas;

b. direction of prevailing winds;

c. accessibility;

d. area available for expansion;

e. local zoning requirements;

f. local soil characteristics, geology, hydrology and topography available to minimize pumping;

g. access to receiving stream;

h. downstream uses of the receiving stream; and

i. compatibility of treatment process with the present and planned future land use, including noise, potential odours, air quality and anticipated sludge processing and disposal techniques.

j. proximity to surface water supplies and water wells.

Where a site must be used which is critical with respect to these items, appropriate measures shall be taken to minimize adverse impacts.

4.2.2 Separation Distances

Separation distances should be designed to prevent the occurrence of objectionable odours in subdivisions and surface water and groundwater contamination, when sewage treatment plants are operated normally and within designed capacities. They should not be designed to accommodate unusual upset conditions that may occur from time to time.

Separation distances will be measured from the proposed odour producing source to the nearest neighbouring lot line. Specific separation distances are as follows:

a. Mechanical plants (including aerated stabilization ponds) shall be located a minimum of 150 m from residences, 30 m from commercial-industrial developments and 30 m from any other adjacent property boundaries. Under special circumstances a lesser separation distance to residences may be adopted, provided provision for odour control equipment is provided at the plant; and

b. Waste stabilization ponds shall be located at least 150 m from isolated human habitation and 300 m from built-up areas.

4.2.3 Flood Protection

The treatment works structures, electrical and mechanical equipment shall be protected from physical damage from the one hundred (100) year flood. Treatment works should remain fully operational and accessible during the twenty-five (25) year flood. This applies to new construction and to existing facilities undergoing major modifications.

4.2.4 General Plant Layout

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

In the layout of the plant, the designer should orient the buildings to provide adequate allowances for future linear expansions of the various treatment sections and orient the plant so that the best advantage can be taken of the prevailing wind and weather conditions to minimize odour, misting and freezing problems and energy consumption. The plant layout should also allow for the probability of snow drifting, with entrances, roadways and open tankage located so that the effect of snow drifting on operations will be minimized.

It is not recommended that construction of any of the facilities be in close proximity to a shore line, except where this is unavoidable. Suitable measures must be taken to adequately protect the structures from the effects of wave action and shore erosion.

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

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

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

Sewage treatment works sites must be adequately fenced and posted to prevent persons from obtaining unauthorized access.

4.2.5 Provision For Future Expansion

In addition to the general site considerations outlined in Section 4.2.4, 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 exchangers, etc. Adequate working space shall 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 WATER QUALITY OBJECTIVES AND WATER USE GUIDELINES

The typical level of treatment required for any new treatment plant in the Atlantic Provinces is secondary treatment with disinfection. However, each new plant will be evaluated on a case by case basis. In the province of Nova Scotia effluent requirement are outlined in the Sewage Treatment Plant Effluent Discharge Policy which is located in an appendix at the back of this manual. Required levels of treatment may be determined to be higher or lower than secondary treatment based on waste assimilation studies. The procedure for carrying out these studies is described in the following section.

4.3.1 Waste Assimilation Study Procedures

a) Level of Effort

As part of the pre-design evaluation (described in Section 1.2), the engineer shall determine from the applicable regulatory agencies the level of effort required for the particular waste assimilation study.

The regulatory agencies may conclude that the effects of the proposed project on the receiving water will be minimal. In this case, the regulatory agencies will set effluent limitations based upon a simple model (possibly basic dilution calculations). In this case, only a minimal level of effort is required for the receiving water study (RWS). The regulatory agencies will determine which parameters will require measurement.

When the regulatory agencies are unsure of the possible effects of a project on the receiving water, they may require that an intermediate RWS and model simulation be conducted as a preliminary assessment tool. If the results of this study indicate that the proposed project would have only a minor effect on the receiving water, the regulatory agencies may, at that point, set effluent limitations. The regulatory agencies shall set the data requirements for the intermediate RWS's.

The third level of effort that may be required is a detailed RWS and complex modelling application. The results of this procedure will determine required effluent limitations. This approach will be required when the regulatory agencies believe that a proposed project may have a significant impact on the receiving water quality. The regulatory agencies will determine RWS data requirements.

b. Water Sampling Procedures

Instruments for electronic *in situ* determination of water quality parameters should be calibrated at least before and after each sampling trip. For example, samples should be collected for salinity to verify field measurements and samples fixed in the field for dissolved oxygen to verify dissolved oxygen probes.

All field collection equipment should be listed and prepared before each sampling trip, insuring that all collection containers are clean and proper log forms and labelling equipment are available. Different containers should be available for metals, nutrients, organics, dissolved oxygen, etc. due to their cleaning and preservation requirements.

An established sequence of collection should be developed and maintained throughout the monitoring effort, insuring that new personnel are trained in the proper methods and sequence of data collection. All samples should be logged and sample log sheets should include station location, time, depth, results of *in situ* sampling, and container numbers for each type of sample. Datum should always be clearly specified (e.g. time of day standard, datum for water surface elevations).

All samples should be preserved on board, where the preservation technique will vary with the type of analysis required, but may involve icing, acidification, organic extraction, etc. The preservation techniques should be documented prior to implementation to the monitoring study. For some samples that do not preserve well it may be necessary to either conduct analyses on board or quickly transfer them to nearby on-shore facilities.

Additional samples should be collected to determine sampling variability and individual samples may be split prior to analysis to determine analytical variability. The number of replicate samples should be established as part of the planning for the monitoring effort. Field samples may also be spiked with a known amount of a standard prior to analysis. The identity of the spiked, split and duplicate samples should be kept on separate logs and the analyst should not be aware of their identity.

The samples should be transferred from the field to the laboratory in a timely manner. The field logs should be recorded and a laboratory log kept of the samples and their arrival. Custody sheets may be kept to further document the transferral of samples.

4.3.2 Waste Assimilation Capacity

4.3.2.1 General

In essence, a waterbodys dilution/assimilative capacity for wastes depends on waste characteristics and a host of physical, chemical and biological factors, such as the flow or volume of the waterbody and the waste discharges, dispersion of effluent, depth and width of the waterbody, type of substrate, algal growths, benthic deposits or organic sludges, etc.

A waste assimilation study is the mechanism to be used in estimating a waterbody's assimilative capacity and establishing effluent requirements to meet the the Canadian Environmental Quality Guidelines (CEQG). Either simple dilution formulae or more sophisticated mathematical models can be used as assessment techniques, depending on the circumstances. For example, with a dilution ratio greater than 20 to 1, simple dilution formulae may be adequate for estimating effluent requirements for discharges with a high degree of treatment (e.g. secondary treatment) and which do not contain hazardous substances. With a dilution ratio less than 20 to 1, more complex assessment techniques may be required to estimate assimilative capacity. Further, under complex situations (e.g. multiple uses of water, flood control requirements, etc.) sophisticated mathematical models may be used to estimate assimilative capacity and effluent requirements.

In areas with existing water quality better than CEQG, it is a good general principle not to allocate the entire assimilative capacity of a receiving waterbody. The need for maintenance of a reserve capacity should be established on a case-by-case basis.

In addition to meeting the CEQG Guidelines a thorough receiving water assessment may be required before the discharge of effluent containing toxic substances will be permitted. Such an assessment should include studies of the potential accumulation and concentration of the substances in the environment (such as bed sediments and aquatic flora and fauna), synergistic effects with other substances and physical factors (such as temperature changes or radiant energy) that may affect the environmental impact of contaminants.

4.3.2.2 Dilution Ratio

Dilution ratio is a simple measure of a receiving water's assimilative capacity. Dilution ratios should be based upon the 7 consecutive day average low streamflow occurring once in 20 years (7Q20), and the peak hourly effluent discharge rate (both expressed in the same units).

4.3.2.3 Mixing Zone

4.3.2.3.1 General

It may not be practical to treat all effluents so they meet the Water Quality Objective concentrations. Therefore, some volume of water must be provided for dilution or modification of the waste effluent before the Objectives can be met.

A mixing zone is a region of a waterbody in which an effluent discharge of quality (chemical/physical/biological) characteristics different from those of the receiving water is in transit and is progressively assimilated from the immediate outfall area to the outer limits of the region. At the boundaries or outer limits of the mixing zone, water quality objectives established by the regulatory authorities to protect beneficial water uses should be achieved. Within the mixing zone, where the objectives are not met, there will be some damage or loss to the aquatic environment. Nevertheless, at no point should conditions be immediately lethal so that swimming organisms cannot evade the area.

4.3.2.3.2 Mixing Zone Requirements

Terms and conditions related to the mixing zones may be outlined in the "Permit to Operate," based on the minimum requirements outlined below. Inherent in these conditions, a mixing zone may not be used as an alternative to adequate treatment.

1 The mixing zone should be as small as practicable, and shall not be of such size or shape to cause or contribute to the impairment of existing or likely water uses. Mixing zone size shall be established on a case-by-case basis, but in no case shall it exceed the following:

   a) in streams and rivers the mixing zone shall be apportioned no more than 25% of the cross-sectional area or volume of flow, nor more than one-third of the river width at any transect in the receiving water during all flow regimes which equal or exceed the 7Q20 flow for the area;

   b) in lakes and other surface impoundments the mixing zone volume shall not exceed 10% of that part of the receiving water available for mixing, and surface water quality objectives must be achieved at all points beyond a 100m radius from the effluent outfall.

2 The mixing zone shall be:

   a) Free from substances in concentrations or combinations which may be harmful to human, animal or aquatic life;

   b) free from substances that will settle to form putrescent or otherwise objectionable sludge deposits, or that will adversely affect aquatic life or waterfowl;

   c) free from debris, oil, grease, scum or other materials in amounts sufficient to be noticeable in the receiving water;

   d) free from colour, turbidity or odour-producing materials that would:

      i) adversely affect aquatic life or waterfowl;
      ii) significantly alter the natural colour of the receiving water;
      iii) directly or through interaction among themselves or with chemicals used in water treatment, result in undesirable taste or odour in treated water, and;

   e) free from nutrients in concentrations that create nuisance growths of aquatic weeds or algae or that results in an unacceptable degree of eutrophication of the receiving water;

3. The presence of a mixing zone should in no way pose a threat to the species survival of any organism in the receiving water outside the mixing zone.

4. No conditions within the mixing zone should be permitted which:

   a) are rapidly lethal to important aquatic life (resulting in conditions which result in sudden fish kills and mortality of organisms passing through the mixing zones) ; or

   b) cause irreversible responses which could result in detrimental post-exposure effects; or

   c) result in bioconcentration of toxic materials which are harmful to the organism or its consumer; or

   d) attract organisms to the mixing zones, resulting in a prolonged and lethal exposure period.

5. The mixing zone should be designed to allow an adequate zone of passage for the movement or drift of all stages of aquatic life; specific portions of a cross-section of flow or volume may be arbitrarily allocated for this purpose;

6. Mixing zones should not interfere with the migratory routes, natural movements, survival, reproduction, growth, or increase the vulnerability to predation, of any representative aquatic species, or endangered species;

7. Mixing zones should not interfere with fish spawning and nursery areas;

8. Rapid changes in the water quality which could kill organisms by shock effects must not be present. Such conditions could have the effect of creating a higher toxicity value;

9. Municipal and other water supply intakes and recreational areas, as a general rule, should not lie within a mixing zone. However, knowledge of the effluent characteristics and the type of discharge associated with the mixing zone could allow such a mixture of uses;

10. Mixing zones may overlap unless the combined effects exceed the conditions specified in these mixing zone guidelines;

11. Limitations on mixing zones should be established by the regulatory authorities on a case-by-case basis, where "case" refers to both local considerations and the waterbody as a whole or segments of the waterbody;

12. Existing biological, chemical, physical and hydrological conditions should be known when considering the location of a new mixing zone or limitations on an existing one;

13. The design and location of the outfall should be considered on a case-by-case basis to reduce the impact of the mixing zone on the receiving waters;

14. Total loadings into all the mixing zones within a river, lake or segment thereof, must not exceed the acceptable loadings from all point-source discharges required to maintain satisfactory water quality;

15. Mixing zones should not result in contamination of natural sediments so as to cause or contribute to exceedences of the water quality objectives outside the mixing zone.

4.3.3 Waste Assimilation Study Field Procedures For Streams and Rivers

4.3.3.1 General Strategies

4.3.3.1.1 Defining Problem and Objectives

Before any field work is carried out on a stream, the problem(s) should first be defined, and the objective(s) of the study laid out.

4.3.3.1.2 Preliminary Office Planning

Maps and aerial photographs of the survey area should be obtained, as well as any previous reports on the waterbody and municipal and/or industrial discharges. All discharges to the waterbody should be pinpointed on the map. The drainage area of the stream or river should also be calculated.

All existing data on water quality monitoring, streamflow, water takings and consumption, water uses, and volumes and characteristics of waste discharges should be obtained.

4.3.3.1.3 Preliminary Field Studies

The entire reach of stream to be studied should be inspected. Waste discharges, dispersion patterns of effluent and tributaries, physical characteristics of the stream (depth, width, water velocity, type of substrate), water uses, algal growths, the presence of benthic deposits or organic sludges and any other pertinent characteristics should be noted.

Preliminary chemical data (BOD_5, nutrients, day and night-time DO) should be obtained to determine the length of stream to be studied, expected concentrations of chemical parameters, and whether night-time sampling is necessary. This data should be collected far enough in advance of the proposed intensive survey data to allow for the lab analyses to be completed and evaluated.

Based on the preliminary information, sampling stations can be selected. Times of travel (TOT) should be determined between sampling stations, also noting times of waste inputs and tributary confluences. Observations should be made at three or more different stream flow stages over a suitable range from low flow to average summer flows and data plotted on log-log graph paper (Times of Travel vs. Streamflow) which will permit interpolation or limited extrapolation for the streamflow conditions to be modelled.

Mixing studies should be carried out on waste discharges and tributary inflows to determine if lateral and/or vertical multi-point sampling is necessary at any downstream stations where complete mixing of the inflows with the stream water has not occurred.

Stream cross-sections should be obtained at about 300 m intervals and at those points where the physical character of the stream changes significantly.

At each sampling location, a staff gauge or other stage measuring device (e.g. bench mark) should be installed. Each time the station is visited the gauge height and date should be recorded to develop a gauge height vs. flow curve to provide stage height at any flow. Flow may be obtained by streamflow gauging and/or may be extrapolated by drainage area ratios.

The low flow to be used in the loading designs must be determined or obtained from a hydrologist. The recommended design streamflow for continuous discharge to a stream is the seven consecutive day average low streamflow occurring once in 20 years (7Q20). Physical data affected by streamflow, such as depth, TOT, mixing, etc., must be extrapolated to this flow. It is therefore important to conduct assimilation studies under flow conditions as close as possible to low flow so extrapolation of data to design conditions (low flow) will be minimal.

Where benthic deposits are believed to exert an appreciable demand on the oxygen resources of the stream, benthic respiration studies should be carried out.

4.3.3.1.4 Pre-Intensive Survey Planning

Based on the preliminary data and the objectives of the study, the type of survey to be conducted will be determined. If 24 hour dissolved oxygen concentrations fluctuate greatly or if wastewater loadings or streamflows fluctuate greatly, then "round-the-clock" sampling will be necessary. However, if streamflows and waste loadings are stable and dissolved oxygen concentrations do not fluctuate greatly (i.e. more than 2 mg/l) from day to night then a simple water quality study with limited sampling may be sufficient (enough samples should be taken, however, to provide statistically valid data). Generally a minimum of 12 samples per station will provide statistically valid data.

Once the type and scope of the study has been determined, the number of samples that will be submitted to the laboratories can be determined. Parameters that may be measured during an assimilation study are:

- pH
- turbidity
- dissolved oxygen (DO) and temperature
- total and fecal coliforms
- BOD_5
- total phosphorous
- soluble reactive phosphorus
- filtered ammonium nitrogen
- total kjeldahl nitrogen
- nitrate and nitrite nitrogen
- BOD_{20}
- plus any other waste component that may affect water quality and use such as heavy metals or volatile suspended solids

A tentative survey date, when streamflow conditions are low and water temperatures high (mid-July to early September), should now be selected. This date should be verified with all participating personnel and the laboratories. At this time the laboratories will need to know the number of samples expected and parameters requested. Water Pollution Control Plant (WPCP) operators and/or industrial contacts must be informed of the expected date of the survey and access to their plants should be arranged for sampling personnel.

A few days before the survey, the local police departments, the local conservation authority and municipal offices should be informed of survey plans. If dye is to be used in the river during the study, these same agencies should be informed. Regional and district regulatory agency staff and Federal DFO/EPS officers must also be informed of the survey.

4.3.3.1.5 The Field Survey

The culmination of the preliminary field work and the office planning is a well-executed field survey. This survey generally extends for 30 to 60 hours with samples collected at selected time intervals (generally 3 to 6 hours apart depending on manpower, equipment and time constraints). The duration of the survey depends upon various factors such as:

a) manpower constraints
b) laboratory capabilities
c) if Photosynthesis and Respiration modelling will be attempted
d) time constraints
e) length of time for sampling run.

Enough samples must be collected to provide statistical validity to the data (generally 10 to 12 samples at each station). If discrete sampling (grab samples taken at a specific location and specific time) cannot be carried out, composite samples may be collected with the approval of the regulatory agency.

If sampling runs can be conducted every 3 hours, then 12 samples can be collected during a 36 hour period providing statistically valid data and adequate DO data for modelling. The study could be conducted from 9:00 pm. one day to 9:00 a.m. two days hence, to obtain the proper DO curve. If time constraints dictate sampling every 4 hours, then a 48-hour study should be conducted. Timing of the beginning of a 48 hour survey is not too critical as only one complete 30-hour DO cycle can be obtained. However, if data is collected at intervals greater than 4 hours it is difficult to obtain a good diurnal-nocturnal DO graph. A 60 hour study with 4 hour sampling intervals will provide 15 samples and if started at 9:00 p.m. and finished at 9:00 a.m. 60 hours later, will provide two complete diurnal-nocturnal cycles for modelling needs. In any event a survey should not be started or ended at a critical low point on the DO graph (i.e. early morning - 4:00 am to 8:00 a.m.).

Situations may arise which may force cancellation or modification of a study. Among them are:

- overcast weather - if a survey had been planned to obtain data during hot, dry weather, then overcast weather will change the DO curve if diurnal-nocturnal effects of algae are of significance.

- streamflow changes (including rain) - concentrations of parameters change greatly during a rainfall, especially during the first 1/2 hour of the rain.

- abnormal changes in waste loading - waste loadings are averaged over the survey period; large changes in loads result in changes of concentrations of parameters in the stream during the survey.

These three situations result in changes in the "steady-state" conditions which are desirable for ease of modelling.

Cancellation of surveys because of these conditions may be minimized by:

a) checking with the weather bureau prior to conducting the study;

b) arranging with the proper authorities if streamflow is controlled by dams to ensure steady streamflows for the period of the survey;

c) arranging with the WPCP or Industrial Plant operator to attempt to ensure that atypical loading conditions do not prevail during the study, due to construction, plant or equipment breakdown etc.

Duties and manpower generally required to carry out an assimilation study are approximately as follows:

- day sampling - one person and vehicle per route
- night sampling - two people and 1 vehicle per route
- time of travel - one person and 1 vehicle
- co-ordinator - one person and 1 vehicle
- sample transport - one person and 1 vehicle
- flows by velocity measurement (if necessary) - two people and 1 vehicle.

Some pointers for carrying out an intensive survey are:

- DO meters should be calibrated to a Winkler titration before each run to ensure accuracy of data. Meters should be calibrated to DO levels in the same range as those expected to be measured.

- Recording DO/temperature meters and/or automatic samplers should be installed prior to the start of the survey.

- Care should be taken when installing recording DO meters in the streams to ensure that water is moving past the probe with a velocity of at least 1 cm per second to prevent oxygen depletion at the membrane.

- DO recorders should be calibrated and automatic samplers checked at least once per day. Samples should be removed from the sampler at least once per day as levels of non-conservative parameters may change with time and temperature.

- Samples at each location should be collected at least 12 times during the survey (tributaries may not need to be sampled as often). This number of samples should provide statistical reliability to the data.

- DO readings should be taken no more than 4 hours apart to provide a smooth representative DO graph.

- Data should be recorded on field data cards (e.g. Figure 2.1 a) and b) and any observation out of the ordinary should be noted on the cards (e.g. turbid discharge, rainfall, etc.).

- Laboratory submission sheets should be completed at the end of each sampling run to avoid confusion and a back-log of samples with no lab sheets.

- Time of travel data can be collected during the survey. If graphs of TOT vs. flow have been previously drawn up and encompass the flow regime at the time of the study and if manpower is limited, this function may be eliminated.

- The survey co-ordinator should be available for consultation any time of the day or night during the intensive survey. Portable radios may prove valuable in maintaining communication with crews.

- Data should be reviewed as it is collected and changes made to plans if the data warrants such changes (e.g. re-locating stations, changing sampling frequency etc.).

4.3.3.2 Survey Procedures

4.3.3.2.1 Station Selection

Many factors are involved in deciding the location of stations for water quality sampling. These include: objectives of the survey, accessibility to the streams, time of travel from waste source, mixing of wastes with the stream, physical characteristics of the stream (e.g. is the location on a sharp bend in the river, upstream or downstream of a dam, in the middle of a large impoundment etc.), length of survey reach, and personnel and equipment availability.

An ideal sampling station is one which would yield the same concentration of parameter no matter where in the cross-section the sample is taken. However, immediately downstream of waste inputs, complete lateral mixing of wastes and stream water rarely exists, so samples should be collected at various locations across the stream. Sometimes vertical mixing does not occur necessitating vertical sampling. Ideally, samples should be collected at various locations and the concentration at each location multiplied by the fraction of flow at each location to give the total loading. Dye dispersion studies can be very helpful in establishing sampling points in a cross-section of stream.

It is good practice to sample at quarter points in the cross-section. Samples should be collected from exactly the same location at each station. Multi-point sampling locations should be marked with paint (if on a bridge) or with poles driven into the steam bed (if shallow enough) or with anchored floats in the stream.

Time of travel between stations should be approximately 2 hours (less for small streams, more for larger).

Sampling from the edge of a stream should be avoided. However, if a sample must be collected from the river bank, select a location on the outside of a bend where the current flows along the bank.

Stations should be spaced far enough apart that changes in water quality are measurable, but not too far apart that too great a change in water quality has occurred since the previous station.

All waste sources and tributaries should be sampled where they enter the stream and their volumes should be known. Samples of final effluent shall also be taken prior to chlorination. The mainstream should also be sampled immediately upstream of the waste source or tributary to properly assess the effects of that load on water quality. If direct sampling of the waste source or tributary is impossible, then the river should be sampled and gauged directly above the source and at an appropriate distance downstream (immediately downstream for gauging and after complete mixing for sampling). The differences between the downstream and upstream loadings can then be attributed to the input.

Control samples (upstream from waste sources) should be obtained for comparative purposes.

4.3.3.2.2 Stream Cross-Sections

Cross-sections of the stream bed should be obtained wherever there is a significant change in the physical characteristics of the stream channel. These are used for calculating the reaeration coefficient (K_a) for each different section of the stream.

Cross-sections may be obtained using a graduated pole, a weighted rope or depth sounder. Stream widths must also be measured (using a tape, graduated rope or stadia and transit). The distances between cross-sections must also be obtained (from a map, by stadia or with a graduated rope).

If stream profiles must be known for flows greater than those under which the cross-sections were obtained, then the profiles should be continued up the stream bed to the height required.

4.3.3.2.3 Flows

Flow information must be known when carrying out time of travel and dispersion studies, cross-sectioning and intensive sampling. For modelling purposes it is necessary to know the stream flow, tributary flows and the volumes of all waste inputs.

Streamflows may be obtained from the DOE regional hydrologist for rivers that are gauged by the Water Survey of Canada or the DOE (either by continuously recording or staff gauges.) If flow data are not available from this agency, it will be necessary to gauge the stream. Flow data for the stream and major tributaries should be obtained at least once per day during the survey. WPCP and industrial wastewater flows can normally be obtained from the plant operators (usually in the form of continuous flow chart). Otherwise, temporary weirs, upstream-downstream flow gauging or some other method of measuring the wastewater flows must be implemented.

4.3.3.2.4 Time of Travel Studies

Time of travel (TOT) data are a vital part of a water quality study (e.g. K_d rates are calculated from BOD vs TOT graph, K_n rates from TKN vs TOT graph). Time of travel is an important consideration when selecting sampling station locations.

Time of travel data will not change unless some physical change is made to, or occurs naturally in the stream (e.g. straightening oxbows, installing dams etc.) Varying densities of aquatic weed growths can change TOT values under similar flow conditions.

Extrapolation of TOT data to low flows may result in erroneous values if data has not been collected over a reasonably large flow range that includes low streamflows.

Time of travel may be determined for river reaches by any one of three methods:

a) floats;
b) volume-displacement method;
c) tracers.

4.3.3.2.5 Reaeration Coefficient Measurement

Occasionally, the expense and effort required to measure stream reaeration coefficients directly in the field may be justified. A technique which has been used with a fair degree of success is the modified tracer-ethylene gas technique developed by Rathbun, Schultz and Stephens.[1]

1 *Rathbun, R.E., Schultz and Stephens; 1975; "Preliminary Experiments with Modified Tracer Technique for Measuring Stream Reaeration Coefficients"; U,S. Department of Interior Geological Survey Open File Report No. 75-256; Bay St., Louis, Miss., U.S.A.*

4.3.3.2.6 Sample Transportation

Samples for chemical and bacteriological analyses should be transported to the laboratories within 24 hours following sampling; all samples should be refrigerated and shipped in a cooler if possible. Samples for BOD_5 analysis should be kept cool.

4.3.3.2.7 Field Notes

To aid in recording field data in intensive surveys, a separate card should be made up for each station and kept in a note book. Each card should clearly indicate the station number, its location and any special sampling instructions (Figure 2.1a). The run number, time of day, DO and temperature and any notable remarks should be entered as the data are collected (Figure 2.1b).

4.3.3.2.8 Sample Numbering

It is very important to clearly label samples to identify the exact time and place the sample was taken.

WPCP and industrial effluents should be clearly labelled as such so the laboratory will recognize these samples and be able to give them special attention.

If there is more than one sampling point at a location, number the points A, B, C, etc. (or L, C, R) from left to right side of the stream facing upstream.

4.3.3.3 Water sampling procedures

4.3.3.3.1 Chemical

a) Manual

Samples should be collected, if possible, directly in the bottle submerged in the river. If sampling by immersing the bottle is not possible then the sample should be collected with a stainless steel, plastic or brass sampling apparatus and transferred to the bottle taking care not to contaminate the sample with the hands or with water from previous sampling.

When sampling by hand (i.e. wading into the stream) the sampler should face upstream and grasp the bottle so the water coming into contact with the hands does not enter the bottle, thereby contaminating the sample. The bottle should be immersed below the water surface (except phenol samples which are skimmed from the surface, and samples for ammonium nitrogen analysis, which should not be bubbled) and the water should be allowed to overflow the bottle to ensure a representative sample.

b) Automatic

Samples may be obtained using an automatic sampler where manpower is limited or access is poor. Automatic samplers should never be used to collect samples for bacteriological or DO analyses. The sampling hose should be placed in the main current on a stake or rod to obtain a representative sample and to allow the current to continuously cleanse the intake so the build-up of algae will be reduced on the intake. Samples should be removed daily.

4.3.3.3.2 Dissolved Oxygen (DO)

When taking a water sample for DO analysis, care should be taken to ensure that there is no bubbling of water into the bucket. After the bucket is filled with water let it settle underwater for a few seconds to allow the river water to be exchanged, thereby ensuring that a representative sample is obtained. If possible, obtain the sample for DO analysis directly from the stream using the bottle in a DO field kit.

When measuring DO with a meter, insert the probe directly into the river or sample bucket and gently agitate the probe (at a velocity of about 1 cm/sec) to prevent oxygen depletion around the membrane. When the readings have stabilized, record the DO and temperature values. Take care to avoid banging the probe on rocks or the sides of the bucket as erratic readings will result.

When installing a DO meter for continuously recording DO, care should be taken to ensure that the probe is situated in the mainstream to avoid oxygen depletion around the membrane and to cleanse the probe. Fouling of the probe by floating debris can be minimized by positioning a 30 cm by 30 cm mesh screen about 2.5 m upstream from the probe. This screen must be cleaned periodically.

When calibrating DO meters, 3 Wet Winkler DO determinations should be made and the values averaged. DO meter readings must also be adjusted for differences in elevation (atmospheric pressure).

4.3.3.3.3 *BOD_{20} (for k_1 Rate)*

A 20 day incubation test shall be performed to determine the relationship between BOD_5 and the ultimate BOD (BOD_u) of the streamwater. Samples for incubation tests should be taken at the first station downstream from the waste discharge where complete mixing of the waste and streamwater has taken place. If possible, samples for another BOD_{20} test should also be taken in the effluent and also near the downstream end of the survey reach.

A total of 12-1 litre bottles should be taken at each site and all numbered identically. The lab submission sheet should be labelled "For k_1 Analysis - complete nitrogen analyses to be performed on the incubated sample on days 0, 5, 10, 15, and 20 days." The nitrogen analyses are performed concurrently to establish a true k_1 attributable to carbonaceous BOD (CBOD).

4.3.3.3.4 *BOD_5 with Nitrogen in Parallel*

To determine the portion of the BOD_5 that is actually carbonaceous, BOD_5 analyses, with complete nitrogen analyses on the first and fifth days, should be taken at least three times at all stations. The difference between the total BOD_5 and the BOD attributed to nitrogen is the carbonaceous BOD_5 (CBOD) which is used to calculate the true CBOD loadings from which a true L_o and K_d can be calculated.

4.3.3.3.5 *Special Sampling Techniques and Handling of Samples*

a) Input Sampling

One of the most important samples collected during an assimilation study is from the final effluent of an STP or industrial plant. This effluent often contains residual chlorine for disinfection purposes. Chlorine is a strong oxidizing agent which inhibits the BOD test. Although the chlorine may be eliminated by the addition of the proper amount of sodium thiosulphate, the titration procedure is somewhat complicated to perform in the field. It is therefore very important that all samples of final effluents be taken prior to chlorination to ensure a representative BOD_5 concentration.

It is wise to obtain a good number of samples (at least 12) of the final effluent, as the BOD_5 and nutrient levels form a very important input to the dissolved oxygen model. It is also a good idea to obtain samples directly upstream from the inputs (including tributaries) and after complete mixing of the stream with the input as a check on the input loading (using mass balance equation).

b) Sampling for Ammonium Nitrogen

As ammonia gas is released by bubbling action (or converted to other forms of nitrogen) care should be taken to minimize bubbling of the sample for ammonium nitrogen analysis. This may be accomplished by special sampling devices (such as a Kemmerer sampler) which can be used to obtain a representative sample below the surface of the water, special pumps which will not bubble the sample, or by carefully obtaining a sample from the surface of the water by allowing the water to gently flow down the sides of the bottle. Wide-mouthed bottles can best be used to sample by this method.

4.3.3.4 Equipment

4.3.3.4.1 Water Sampling

a) Buckets

Sampling buckets should be made from relatively non-contaminating materials such as stainless steel or plastic (e.g. PVC). However, care should be taken when sampling for certain parameters that the bucket material does not contaminate the sample. The sampling bucket should always be rinsed thoroughly before taking the sample.

b) Depth Samplers

Various types of samplers can be used to sample water from depths. These are made from brass, PVC or acrylic; some are nickel-plated or teflon-lined to virtually eliminate any chance of sample contamination.

4.3.3.4.2 DO Meters

Electronic DO meters considerably reduce the manpower required to perform DO tests.

To save effort and reduce chance of errors, consideration should only be given to the use of temperature compensated models when purchasing DO meters.

4.3.3.4.3 Fluorometer

Approved fluorescent dyes used in time of travel, effluent dispersion and flow measuring studies may be detected in the stream using a fluorometer with filters which measure the appropriate wavelength of fluorescence. Fluorometers are usually portable and may be powered in the field by a small A.C. generator. Fluorometers should be calibrated before each use.

4.3.3.4.4 Pyranograph

This portable instrument measures and records solar radiation in g cal/cm^2/min, for either a 24 hour or 7-day period. The recorded data will provide the time of sunrise and sunset and the intensity of sunlight for that day. This information is necessary for input to the dissolved oxygen balance when modelling for photosynthesis and respiration.

4.3.3.4.5 Depth Sounder

This equipment is useful for determining the cross-sectional area of the stream. The data is used in K_a calculations and time of travel calculations in large rivers. Some points to consider when purchasing a depth sounder are:

- chart speed (the faster the better)
- printout - perpendicular printout rather than circular to prevent distortion of recording
- ensure that scale provides adequate profiles at shallow depths
- portability (including the necessary power supply-battery/generator)
- accuracy
- precision

- durability
- splash proof case

4.3.3.4.6 Current Meter

If flows must be obtained manually a suitable current meter should be used.

4.3.4 Waste Assimilation Study Field Procedures For Lakes

4.3.4.1 Conditions Necessitating a Waste Assimilation Study

When sewage discharge is proposed to a lake, a waste assimilation study will be required when any of the following conditions are expected or present:

a) The waste discharge exceeds 2.2 L/s;

b) The average lake outflow rate is less than 200 times the effluent design average flow;

c) The average lake outflow rate is less than 1000 times the effluent design average flow and the lake's theoretical retention time is greater than 10 years; or

d) The proposed discharge increases the total discharge to a lake to greater than 44 L/s.

In addition, a waste assimilation study will be required where local conditions indicate that sewage effluent may drift into certain areas. These could include areas which might reasonably be utilized for water contact recreation or where water is extracted for any purpose.

4.3.4.2 General Strategies

4.3.4.2.1 Defining the Problem and Objectives

Before any field work is carried out on a lake, the problem(s) should first be defined, and the objective(s) of the study laid out.

4.3.4.2.2 Preliminary Office Planning

Maps and aerial photographs of the survey area should be obtained, as well as any previous reports on the waterbody and municipal and/or industrial discharges. All discharges to the waterbody should be pinpointed on the map. All existing data on water quality monitoring, water takings and consumption, water uses and volumes, and characteristics of waste discharges should be obtained.

An outline map should be prepared showing the lake outline, all inflowing and outflowing waters, any rock outcrops and shoal locations, and any musky, bog and flooded areas. Man-made features such as settlements, railroads, highways, bridges, docks, dams and access points should also be shown. From this map the surface water area and perimeter of the lake and islands should be measured and recorded. The lake's drainage area should also be calculated and shown on the map.

4.3.4.2.3 *Preliminary Field Studies*

A shoreline cruise of the lake should be conducted. The shoreline cruise is intended to collect and record specific information about the physical attributes of a waterbody. These particular attributes are as follows:

a. Fluctuation
b. Number of Resorts and available discharge data
c. Number of Cottages and available discharge data
d. Dams (type and location)
e. Slope Angle of Terrain
f. Inlets and Outlets
g. Shoreline Soil Types
h. Bottom Soil Types
i. Pollution
j. Vegetation

The collection of this information should be carried out prior to the field survey, so as to familiarize the survey personnel with the lake.

a. Fluctuation

Fluctuation refers to the change in water levels. However, in a number of cases it will be difficult to detect water fluctuations if there is no exposed rock, so it will be up to the discretion of the survey crew to obtain the measurement. The vertical distance in metres between the high water mark and the low water mark is the fluctuation. These marks are usually stains on the rock. The high water mark is indicated by the growth of lichens which give rocks a dark stain and a light exposed surface below. The low water level is recognized by a light area exposed to weathering and a dark stain below indicating permanent submergence by water. Of course, if there is a permanent monitoring station such as a power dam on the lake, the appropriate authorities can be contacted and the accurate measurements can be obtained.

b. Number of Resorts

The number of resorts is the total number of commercial establishments on the lake.

c. Number of Cottages

The number of cottages is the total number of private residences (year round living), cottages and cabins. A distinction should be noted between the number of year round and seasonal residences.

d. Dams

Dams are any structures, natural or man made, that impede the flow of water into or out of a lake. These can be beaver dams, log jams, hydro dams, etc.

e. Slope Angle of Terrain

Slope angle of terrain is the inclination at which the backshore is elevated from the water surface. The inclination is normally measured on-site at regular intervals using a clinometer and is expressed in degrees.

To accurately measure the slope angle with a clinometer, the boat must be at the shoreline. A reading is then taken from the front seat of the boat.

f. Inlets and Outlets

Inlets and outlets are the streams and rivers flowing in and out of a lake. These waters should be divided into two groups: ***perennial***, being a stream which has a continuous flow and ***non-perennial***, being a stream which has a discontinuous flow. The discharge from each inlet and outlet should be calculated.

Since a quick approximation is needed to calculate flows into and out of lakes, the following formula can be used.

$$\text{Discharge} = \frac{W\ D\ L\ K}{\Delta t}$$

W = average width (m)	K = velocity correction factor
D = average depth (m)	= 0.8 (rough bottom)
L = length of station (m)	= 0.9 (smooth bottom)
Δt = average time(s)	

The section of stream selected should be as straight as possible, without obstructions, of approximately the same width and the same type of bottom throughout and far enough away from the lake that there is no effect of wind or wave action. Ten metres of stream is a convenient length.

A floating object should be clocked to determine the time it takes to float the length of the station. The float should be timed over the distance at least three times then averaged.

g. Shoreline Soil Type

Shoreline soil type is the material on the foreshore.

h. Bottom Soil Type

Bottom soil type is the material on the offshore.

i. Pollution

Pollution should be checked for and noted. Any **visible** signs of contamination and possible sources should be noted. Examples could range from litter to algal blooms caused by sewage. All waste discharge to the lake should be pinpointed on the map.

j. Vegetation

Vegetation is important to inventory but only in a generalized manner. Vegetation whether it be terrestrial or aquatic enhances the environment by impeding the erosion of soils, providing food whether direct or indirect, producing oxygen and increasing surface area for larger populations of animals. The type and quality of vegetation is limited by climate and soils. Soils derived from infertile hard rock have significantly less abundant vegetation than do soils derived from glacial till and sedimentary rocks.

4.3.4.2.4 *Pre-Intensive Survey Planning*

Parameters that should be measured during an assimilation study are:

- total and soluble phosphorus
- total and soluble nitrite
- total and soluble nitrate
- total and soluble ammonia
- total and soluble organic nitrogen
- pH
- dissolved oxygen (DO)
- temperature
- algal biomass in the upper mixing zone
- alkalinity
- suspended and dissolved solids
- BOD_5
- Bacteria
  - Fecal coliforms
  - Fecal streptococcus
- chlorophyll a
- pheophytin a
- Turbidity, colour and Secchi depth throughout **all** vertical Zones

4.3.4.2.5 *The Field Survey*

The field survey should be conducted over the period of one year. The actual duration of the survey depends upon various factors such as:

a) manpower constraints
b) laboratory capabilities
c) time constraints

Enough samples, at each station, must be collected to provide statistical validity. Samples should be collected on a monthly basis during the months of September through April and bi-weekly during May through August.

The bi-weekly samples must be scheduled to coincide with the period of elevated biological activity. If possible, a set of samples should be collected immediately following spring turnover of the lake. Samples should be collected between the hours of 1130 and 1400.

The preparation of a contour map is essential to develop the profile characteristics of the lake basin and to calculate the volume of water. To prepare the contour map, the various depths of the water basin should be measured by an echo sounder. Transects must be **straight** runs from and to visible shoreline features which can be identified on the field map, (i.e. projections, centre of indentations, landmarks, etc.) Transects must be kept as short as possible and perpendicular to the shore. Transects must be run at a **constant** outboard motor speed. Speeds may vary from line to line but **never** while running the **same** transect. If variation occurs, return to the starting point and recommence the sounding run. If the echo sounder has adjustable paper speed, it should **not** be changed during a sounding run. Line number, direction of run and distance from shoreline(s) of the sounding recording terminals must be indicated on the transect map and on the tape. This eliminates any doubt in the interpretation of the sounding tape.

4.3.4.3 Survey Procedures

4.3.4.3.1 Station Selection

The following criteria are recommended as guidelines in establishing chemistry stations:

1) One chemistry station must be conducted in the deepest basin of the waterbody. If the echo sounding tapes indicates more than one basin of equal depth (within ± 5 metres), then sample each basin at the deepest part.

2) If there is a basin in proximity to an inlet(s) and/or and outlet(s), a sample station should be set up. However, do **not** sample where there is movement of water since these data will reflect the inlet characteristics and not those of the lake. Also, if the discharge is less than $0.5m^3/s$ a station is not required.

3) If a basin is 75% of the maximum depth of the deepest basin of the waterbody, a sample station should be set up.

4) If there are bays that are isolated from the main body of water, it is advisable to set up a station.

5) As a rule of thumb, there should be at least one chemistry station for every 250 hectares of water surface area.

4.3.4.3.2 Sample Transportation

Samples for chemical and bacteriological analysis should be transported to the laboratories within 24 hours following sampling. All samples should be refrigerated and shipped in a cooler if possible.

4.3.4.3.3 Sample Numbering

It is very important to clearly label samples to identify the exact time and place the sample was taken.

4.3.4.4 Water Sampling Procedures

4.3.4.4.1 Water Transparency

Transparency describes the extent of light penetration into water. As light plays an important part in many biological processes, this measurement must be performed accurately. Transparency is always measured at the time of each and every chemistry test. If the chemistry tests are done before noon the transparency tests are done on completion of the chemistry tests. If in the afternoon, transparency is done first so as to get the reading as close to high noon as possible. The actual times of Secchi depth readings should be recorded.

Transparency in lakes is measured with a weighted metal disc, twenty centimetres in diameter with alternate white and black quadrants. This instrument is known as a **Secchi disc**. A line, graduated in metres and centimetres is attached to the disc.

To obtain a Secchi disc reading:

1) Lower the disc into the water on the shady side of the boat and note the depth at which the disc **disappears** from view in descent.

2) Raise the disc and note the depth at which the disc **reappears** in ascent.

3) Calculate and record the arithmetic mean of these two readings. **This** is the Secchi disc reading.

4) If the lake bottom is contacted during visual descent, record station depth for Secchi depth and indicate that this depth is bottom.

Surface conditions will alter the Secchi disc readings. Therefore, it is important that these conditions are recorded when the test is performed. The various categories are listed below:

Calm	-	complete absence of wind glass like appearance of water
Rippled	-	lightly ruffled - not more than a 2 cm rise in the undulations
Wavy	-	ruffled - not more than 5 cm rise in the undulations
Rough	-	waves more than 5 cm in height

N.B. Chemistry tests should not normally be done in rough conditions.

4.3.4.4.2 Water Temperature Series

A vertical temperature series in lakes should be recorded at all water chemistry stations. The procedure should be as follows:

1) Measure and record the air temperature by use of a **dry** bulb thermometer, in the shade over the side of the boat and near the water surface. Don't let the thermometer get wet.

2) Observe and record the water temperature at the surface and 1.5 meters below the surface. Continue at 1.5 metre intervals, until the lake bottom is reached or the water temperature reads 4°C.

3) Water temperature recordings are waived in the 4°C temperature depths - BUT temperatures will be taken and recorded **one** metre above the bottom and **midway** between the bottom and upper 4°C temperature recording.

4) Stations less than 10 m deep should have the temperature recorded at one metre intervals.

5) In addition, water temperatures must be taken and recorded at **all** depths where water chemistry tests are made, e.g. mid-thermocline and the four mg/L dissolved oxygen level.

4.3.4.4.3 Chemistry Measurement Series

A vertical chemistry measurement series shall be developed at each water chemistry station. A Secchi disc reading and the accompanying data are recorded for each sample site.

Samples must be collected between 0.5 m below the surface and 0.5 m above the bottom. They must also be collected at intervals of every 1.5 m or at six equal depth intervals, whichever number of samples is less.

4.3.4.4.4 Total Dissolved Solids

The concentration of total dissolved solids is calculated by measuring specific conductance within a lake. Conductivity is a numerical expression of the ability of an aqueous solution to carry an electric current. This ability depends on the presence of ions, their total concentration, mobility, valence, and on the temperature of measurement. Electrolytic conductivity increases with temperature.

4.3.4.5 Equipment

4.3.4.5.1 Water Sampling

Two devices for collecting the water sample for chemistry measurements are recommended. These are the Kemmerer Bottle and the Van Dorn Bottle.

The vertical type Kemmerer bottle is brass or plastic in construction and is available in different sizes. The unit is secured by a graduated braided nylon line not exceeding 5.0 mm in diameter for lowering and/or suspension in water.
The mechanism to close the stoppers and isolate the water at desired depths is controlled by the striking impact of a split or solid brass component called a messenger. It is released by the operator once the bottle is located at the desired depth.

The Van Dorn bottle is a sampler made of non-metallic components. The body and drain valves are constructed of P.C.V. plastic or clear acrylic. Closure of the bottle is by means of a messenger similar to the Kemmerer Bottle. The Van Dorn bottles are available in many sizes and come in two types. One is a vertical sampler, the other is a horizontal type.

4.3.4.5.2 Conductivity Measuring Bridges

There are a number of conductivity measuring bridges available with a variety of cell types. It is important that the user becomes familiar with the unit before proceeding to take measurements.

4.3.4.5.3 Depth Sounder

Some points to consider when using a depth sounder are:

- chart speed (the faster the better)
- printout - perpendicular printout rather than circular to prevent distortion of recording
- ensure that the scale provides adequate profiles at shallow depths
- portability (including the necessary power supply)
- accuracy
- precision
- durability
- splash proof case

4.3.4.5.4 Thermometers

Hydrographic thermometers and maximum/minimum thermometers are used to determine water temperatures. Hydrographic thermometer units consist of a sensitive direct current meter and a calibrated bridge network that contains a temperature sensitive resistor called a thermistor. All thermometers must be standardized.

4.3.5 Waste Assimilation Study Field Procedures for Coastal Waters

4.3.5.1 General Strategies

4.3.5.1.1 Defining the Problem and Objectives

Before any field work is carried out on any coastal water, the problem(s) should first be defined, and the objective(s), of the study laid out.

4.3.5.1.2 Preliminary Office Planning

Maps and hydrographic charts of the proposed discharge area shall be obtained, as well as any previous reports on the waterbody and any nearby municipal and/or industrial discharges.

All existing data on water quality monitoring, water uses, current speed and direction, receiving water density distribution, and volumes and characteristics of waste discharges should be obtained.

4.3.5.2 Pre-Design Surveys vs. Monitoring Surveys

Oceanographic surveys for waste-water disposal systems can be placed in one of two general categories - predesign or monitoring. Each of the two types of surveys has a different objective and possesses unique requirements that demand careful consideration.

Predesign surveys must provide not only the necessary information to determine the proper alignment of the submarine outfall, the location and orientation of the wastewater diffuser system, but also the final design criteria for the outfall and diffuser system which will assure the protection and enhancement of the receiving environment. In addition, accurate bathometric profiles, benthic soil characteristics for outfall placement and foundation, and sediment erosion and deposition behaviour must be determined.

Monitoring surveys are of several types. Predischarge monitoring surveys are conducted to establish the baseline or natural conditions of the receiving water, receiving sediments and adjacent shoreline prior to discharge of waste waters. Post discharge monitoring surveys are conducted for the purpose of determining the effects of the wastewater discharge on the receiving environment or for the purpose of assessing the performance of the disposal system for verification of design criteria and future design improvements.

4.3.5.3 Pre-Design Waste Assimilation Studies

4.3.5.3.1 General Objectives

Three major objectives must be satisfied in the conduct of a pre-design oceanographic survey. Firstly, a pre-design survey must determine the dispersion or diluting characteristics of the receiving water. Secondly, a pre-design survey must provide sufficient information on the ecosystem in the proposed discharge area to assure that biologically significant or sensitive areas will not be adversely affected by the disposal system, both during construction of the outfall and during the continuing discharge of the wastewater. Thirdly, foundation conditions for outfall and diffuser placement must be determined prior to preparation of engineering plans and specifications on the waste disposal system.

To satisfy all three objectives a general area for placement of the disposal system must be surveyed and potential alternative outfall sites chosen.

Because the rational determination of an outfall length necessary to meet particular receiving environment requirements for a specified level of treatment will be dependent upon the survey results, the predesign oceanographic survey should be designed to provide the greatest practical flexibility in the selection of outfall alignments and lengths.

4.3.5.3.2 Parameters Requiring Measurement

Those parameters which are most critical to the design should be most thoroughly measured in the conduct of the oceanographic survey, with less effort expended in assessing perhaps more traditional parameters which have a much lesser effect on design considerations.

a) Density of Receiving Water

Density measurements throughout the water column are required in order to estimate the extent of initial dilution occurring over a diffuser. Water density may be determined by calculation from temperature and salinity, conductivity or specific measurements.

b) Horizontal Ocean Currents

Horizontal current velocity may be measured by using propeller or cone-type meters suspended at a specific depth.

c) Horizontal Eddy Dispersion

Measurement and prediction of the magnitude of horizontal eddy dispersion requires determination of horizontal ocean currents occurring in the horizontally moving waste-water field and of the appropriate eddy diffusivity. Measurements of the diffusivity, or diffusion coefficient, is very difficult and requires sophisticated and complex techniques. The relative dilution effected by eddy dispersion of concern to most wastewater disposal systems, however, is small, permitting a rather gross estimate of the diffusivity without affecting the design substantially. The effect of wave climate on surface and alongshore currents should also be evaluated. The effect of ice accumulation around outfall pipes should also be analyzed.

d) Decay/Disappearance Rates

Determination of disappearance rates for specific non-conservative constituents requires, in most cases, special studies wherein a mass of the discharged wastewater containing the constituent is monitored in the receiving environment over a period of time to determine its decay as a function of time. The observed diminution must be corrected for physical dilution that has occurred by eddy dispersion over the period of observation. A tracer material, usually non-toxic fluorescent dye, should be employed in these studies to obtain physical dilution.

e) Wind Velocity and Direction

f) Receiving Water Quality Parameters

To assure placement of an outfall in an area which will provide the least deleterious effects on the environment and to predict what effects the wastewater discharge will have on the receiving environment, it is necessary to characterize the physical and chemical conditions of the receiving environment and the indigenous flora and fauna in the intertidal zones, in the benthic sediments and in the overlying waters.

Water quality characterization should include dissolved oxygen, pH, temperature, salinity, transparency or turbidity, total and fecal coliforms, BOD, and in some cases, nitrogenous and phosphorous forms. Benthic sediments should be characterized with respect to particle size distribution, organic carbon and nitrogen, dissolved sulfide, heavy metals, and chlorinated hydrocarbons.

Microplankton, macroplankton and nekton populations in the receiving waters may be sampled and their diversity determined. The benthos should be properly examined for specific enumeration, and the extent of biological productivity and diversity should be determined.

g) Benthic Soil

Prior to preparation of engineering plans and specifications the structural characteristics of the benthic soils must be determined. For ease and economics of construction of the outfall, the alignment should not encounter rock outcroppings, and other submerged obstructions, abrupt vertical discontinuities or escarpments. Coring and soil analysis should be performed to determine footing characteristics, and bathometric profiles should be made over a period of time to determine if problems with shoaling and shifting sediments are likely to occur.

4.3.5.3.3 Survey Procedure

Because oceanographic surveys for pollution control facilities are usually restricted by economic and time constraints, it is extremely important to specify a survey program that will provide the most useful and meaningful data within the resources available.

a) Sampling Stations

Sampling station location should be selected to provide an adequate areal coverage of the receiving environment and should be for the most part located at coordinates representing potential diffuser locations and outfall alignments.

Sampling points for certain measurements within the water column should be selected to provide the best representative sample, or samples, of the entire column or of the characteristic under consideration.

b) Current Measurements

Synoptic current measurements throughout the receiving water mass over an extended period of time will provide the best description of the current structure and circulation pattern, but sufficient resources are generally not available for such extensive sampling. A reasonable assessment of a current regime at any particular time can be obtained by taking measurements of current speed and direction near the bottom and top of the water column, and at mid-depth. If the number of current metering points is further limited because of time or other constraints, single-depth current measurements will be most useful at a depth representative of the currents responsible for horizontal movement away from the diffuser following the initial dilution process. In those instances where a pronounced pyenocline exists, the best single depth for measurement would be several meters below the pyenocline. Where there is no such pyeriocline, single-depth measurements at several meters below the surface would usually provide the most useful information.

Coastal currents are affected by lunar tides, major oceanic currents, wind stress, and tributary freshwater discharges, which are all variable in magnitude and effect with respect to time. To obtain a reasonable estimate of the current velocity distribution, therefore, measurements should be taken over a sufficiently long period (preferably one year) to account for the diurnal and seasonal changes.

c) Water Quality Measurements

Water quality characteristics are less variable than current characteristics, but do vary somewhat diurnally and greatly seasonally. Thus, only one or two measurements of water quality characteristics are generally required during a single day several times during a year.

Water quality characteristics can be measured either *in situ* with direct reading or recording devices or from discrete collected samples. Existing equipment allows *in situ* simultaneous measurement of DO, pH, transmittance, temperature and conductivity, and easily provides the necessary vertical definition to establish density gradients and pycnoclines. Other measurements, such as nutrient concentrations, must be made from discrete samples collected with an appropriate water sampling device.

d) Biological Parameters

Of the biological parameters, the benthic flora and fauna are the least affected by diurnal and seasonal factors, and thus can be characterized adequately by sampling only several times during a year. Benthic samples can be collected either remotely from a vessel or directly by divers. Remote sampling is performed either with a dredge, which allows for recovery of a disturbed sample, or a coring tool, which provides a relatively undisturbed specimen. Generally, dredges are used for biological characterization of the sediments and corers are employed for physical and chemical assay of the benthic materials.

Several samples are usually taken per time in order to provide an indication of localized sample variation.

A more systematic and specific approach, but also more biased, to benthos characterization is provided by divers who can observe and report on general biological conditions and obtain samples and specimens for later analysis that are highly controlled with respect to size and location.

Because the distributions of the most motile and the floating forms, e.g. microplankton, macroplankton and nekton, are highly time and space dependent, a large number of samples for these organisms must be collected to obtain a statistically significant characterization.

Plankton sampling is usually accomplished by vertical or horizontal tows with appropriately sized netting. Discrete samples collected with conventional water samplers can also be used for plankton enumeration and identification, but this procedure allows for much greater sampling error due to the generally large spatial variations encountered in plankton populations and distributions.

4.3.5.3.4 Equipment

Current Meters

Current meters can be classed either as moored or non-moored, and can be direct reading or recording. Moored recording meters provide an almost continuous record of current velocity, but are restricted because each individual meter is fixed in both the vertical and horizontal plane. Meters operated from a vessel, on the other hand, provide measurements from a variety of depths and locations, but present only an instantaneous sampling of the current regime.

4.3.6 Waste Assimilation Study Field Procedures for Estuaries

4.3.6.1 General Strategies

4.3.6.1.1 Defining Problem and Objectives

An estuary is defined as the tidal mouth of a large river.

Before any field work is carried out on an estuary, the problem(s) should first be defined, and the objectives of the study laid out.

4.3.6.1.2 Preliminary Office Planning

Maps and aerial photographs of the survey area should be obtained, as well as any previous reports on the waterbody and municipal and/or industrial discharges. All discharges to the waterbody should be pinpointed on the map. All existing data on water quality monitoring, water takings and consumption, water uses, flows, and volumes and characteristics of waste discharges should be obtained.

4.3.6.1.3 Preliminary Field Studies

If manpower and time permit, the entire reach of the estuary to be studied should be inspected. Waste discharges, dispersion patterns of effluents, physical characteristics of the estuary, water uses, algal growths, the presence of benthic deposits or organic sludges and any other pertinent characteristics should be noted.

4.3.6.1.4 Boundary Conditions

Boundary condition data are **external** to the model domain and are driving forces for model simulations. For example, atmospheric temperature, solar radiation and wind speeds are not modelled but are specified to the model as boundary conditions and drive modelled processes such as mixing, heat transfer, algal growth, reaeration, photolysis, volatilization, etc. Nonpoint and point source loadings as well as inflow water volumes are model boundary input. The boundaries at the upstream end of the estuary and the open boundary at the ocean provide major driving forces for change. Models do not make predictions for the boundary conditions but are affected by them.

4.3.6.1.5 Pre-Intensive Survey Planning

In setting limits on wastewater quantity and quality, the following factors affecting estuarine water quality should be assessed: salinity, sediment, bacteria and viruses dissolved oxygen depletion, nutrient enrichment and over-production, aquatic toxicity, toxic pollutants and bioaccumulation and human exposure.

a. **Salinity**

Salinity is important in determining available habitat for estuarine organisms. Large wastewater discharges into relatively small estuaries or embayments can alter the local salinity regime through dilution. Even when the salinity is not affected by the discharge, it is measured and modelled in order to quantify advection and dispersion. These processes help determine how wastewater is assimilated into the estuary.

b. **Sediment**

Sediment enters estuaries from many sources, and can alter the habitat of benthic organisms. Sediment is also an important carrier of such pollutants as hydrophobic organic chemicals, metals, and nutrients. Sediment transport can move pollutants upstream, or between the water column and the underlying bed. Even when wastewater does not introduce excess sediment into an estuary, it is often measured and modelled in order to quantify the transport of sediment-bound pollutants.

c. **Bacteria and Viruses**

Bacteria and viruses may enter estuaries in runoff from farms and feedlots and in effluent from marinas as well as from municipal or industrial wastewater discharges. These pathogens may be transported to bathing beaches and recreational areas, causing direct human exposure and possibly disease. Pathogens also may be transported to shellfish habitat; there they may accumulate in oysters, clams, and mussels and, subsequently, cause disease when eaten by humans.

d. **Dissolved Oxygen Depletion**

Adequate, sustained DO concentrations are a requirement for most aquatic organisms. Seasonal or diurnal depletion of DO, then, disrupts or displaces estuarine communities. Ambient DO levels are affected by many natural processes, such as oxidation of organic material, nitrification, diagenesis of benthic sediments, photosynthesis and respiration by phytoplankton and submerged aquatic vegetation, and reaeration. The natural balance can be disrupted by excessive wastewater loads of organic material, ammonia, and nutrients. Other sources of nutrients, such as runoff from agricultural, residential, and urban lands and atmospheric deposition, can also disrupt the DO balance. Excessive heat input from power plants can aggravate existing problems. Because of its intrinsic importance, and because it is affected by so many natural and man-influenced processes, DO is perhaps the best conventional indicator of water quality problems.

e. **Nutrient Enrichment and Overproduction**

Adequate concentrations of nitrogen and phosphorus are important in maintaining the natural productivity of estuaries. Excessive nutrient loading, however, can stimulate overproduction of some species of phytoplankton, disrupting the natural communities. Periodic phytoplankton "blooms" can cause widely fluctuating DO concentrations, and DO depletion in benthic and downstream areas. Nutrient loads can be introduced in wastewater and runoff and through atmospheric deposition.

f. **Aquatic Toxicity**

High concentrations of ammonia, many organic chemicals, and metals can disable or kill aquatic organisms. Acute toxicity is caused by high exposure to pollutants for short periods of time (less than four days). The toxicity of a chemical can be affected by such environmental factors as pH, temperature, and sediment concentrations. Overall toxicity results from the combined exposure to all chemicals in the effluent and the ambient waters.

g. **Bioaccumulation and Exposure to Humans**

Lower concentrations of organic chemicals and metals that do not cause aquatic toxicity can be taken up and concentrated in the tissues of estuarine organisms. As fish predators consume contaminated prey, bioaccumulation of these chemicals can occur. This food chain contamination can persist long after the original chemical source is eliminated. Humans that regularly consume tainted fish and shellfish can receive harmful doses of the chemical.

Human exposure to harmful levels of organic chemicals and metals can also occur through drinking water withdrawals from fresh water tidal rivers.

4.3.6.1.6 The Field Survey

Two general types of surveys may be utilized in estuary modelling. These are described as: those used to identify short-term variations in water quality and those used to estimate trends or mean values.

a. **Intensive Surveys**

Intensive surveys are intended to identify intra-tidal variations or variations that may occur due to a particular event in order to make short-term forecasts. Intensive surveys should encompass at least two full tidal cycles of approximately 25 hours duration. Intensive surveys should usually be conducted regardless of the type of modelling study being conducted.

Wherever possible, all stations and depths should be sampled synoptically. For estuaries that are stationary wave systems (high water slack occurs nearly simultaneously everywhere), this goal may be difficult to achieve due to the logistics and manpower required. Synoptic sampling schemes are constrained by distance between stations, resources in terms of manpower and equipment, and other factors which may limit their applicability. Where it is not possible to sample synoptically, careful attention should be given to the time of collection. For some estuaries, where movement of the tidal wave is progressive up the channel, sampling the estuary at the same stage of the tide may be possible by moving upstream with the tide to obtain a synoptic picture of the water quality variations at a fixed tide stage, that is a lagrangian type of sampling scheme. Sampling should not be conducted during unusual climatic conditions in order to ensure that the data is representative of normal low flow, tidal cycle and ambient conditions.

Boundary conditions must be measured concurrently with monitoring of the estuary. In addition, a record of waste loads during the week prior to the survey may be critical. It is necessary to identify all of the waste discharging facilities prior to the survey so that all waste discharged can be characterized. Estimates of non-point loads are also required.

Where project resources limit the number of samples, an alternative may be to temporally integrate the samples during collection or prior to analysis. This will, however, not provide information on the variability associated with those measurements.

b. **Trend Monitoring**

Trend monitoring is conducted to establish seasonal and long-term trends in water quality. Intensive data is not sufficient to calibrate and validate a model which will be used to make long-term projections, due to differences in the time scales of processes affecting those projections. Trend sampling may take place on a bi-weekly or monthly basis. Stations should be sampled at a consistent phase of the tide and time of day to minimize tidal and diurnal influences on water quality variations. Diurnal variations must still be considered, however, tidal affects may be less important in wind dominated estuarine systems. Care should be exercised to sample during representative conditions and not during unusual climatic events in order to allow comparison between sampling times. Some stations may be selected for more detailed evaluation. Intensive surveys, spaced over the period of monitoring, should also be considered where the trend monitoring will be used to track changes in parameters between intensive surveys.

Boundary data should generally be measured at a greater frequency than estuarine stations used for monitoring trends. Boundary conditions are critical in that they will drive the model used for waste load allocation. The rate at which the boundary conditions are expected to change will indicate the time scale required for boundary sampling. Tiered or stratified sampling programs may be required which include different sampling strategies, such as between low and high flow periods. The more intensive boundary data will provide an estimate of the mean driving forces for the model as well as their associated variability.

4.3.6.2 Survey Procedures

4.3.6.2.1 Spatial Coverage

An intensive spatial coverage of the estuary for some indicator or surrogate water quality parameter, such as: salinity or turbidity, is generally needed in order to estimate spatial variability, as well as determine the model type and segmentation required.

Generally, the spatial grid for an estuarine model should extend from above the fall line, or zone of tidal influence, to the open boundary of the estuary. The last gauging station is often a good upper boundary since they are typically placed outside of the region of tidal influence. In some cases the ocean boundary will extend beyond the estuary into the ocean to insure a representative boundary condition or to allow use of tidal gauge information collected at some point away from the estuary.

Where simple waste load allocation studies are planned on a portion of an estuary, and it is unrealistic to model the entire estuary, then the spatial grid may be delimited by some natural change in depth or width, such as a restriction in the channel or regions where the velocity and water quality gradients are small. The spatial grid must encompass the discharges of interest in all cases.

Sampling stations should generally be located along the length of the estuary within the region of the model grid, with stations in the main channel margins and subtidal flats for the intensive surveys. Lateral and longitudinal data should be collected, including all major embayments. The spatial coverage required is governed by the gradients in velocities and water quality constituents. Where no gradients exist, then a single sample is sufficient. Some caution should be exercised in the selection of the indicator parameter for this decision. For example, strong vertical dissolved oxygen gradients may occur in the absence of velocity, thermal or salinity gradients. Two areas where cross-channel transects are generally required are the upper and lower boundaries of the system.

The spatial coverage should consider the type of model network to be used. For model networks with few, large segments, several stations (e.g. 3-6) should be located in each model segment in order to estimate spatial variability. For detailed models with many segments, it may not be possible to determine the parameters for each segment. For initial conditions and model evaluation, sufficient samples should be collected to estimate missing data by interpolation.

Where resources are limited, one possible monitoring strategy is to spatially integrate samples, such as over depth or width depending on the modelling approach used. Careful consideration will need to be given to the integration scheme for this type of monitoring. For example, a flow weighted integration would require some prior knowledge of the fraction of the total flows associated with all sampling stations.

4.3.6.2.2 Estuary Bathometry

Data are always required to determine model morphometry. Morphometry affects the characterization of the estuary and the type of modelling approach required. Estuarine depth controls propagation of the tidal wave. Shallow channels and sills increase vertical mixing while deep channels are more likely to be stratified with greater upstream intrusion. Deep fjords with shallow sills usually have little circulation and flushing in bottom waters. The length of the estuary determines the type of tidal wave, phase between current velocities and tidal heights. The width effects velocities (narrow constrictions increase vertical mixing and narrow inlets restrict tidal action). Wind-induced circulation is transient and interacts with channel geometry to produce various circulation patterns and affects vertical mixing and sediment transport. Bathometric data are available from the Canadian Hydrographic Service.

For certain estuaries, the affects of tidal marshes can dramatically effect estuarine circulation and water quality. These are generally some of the more difficult systems to model. An initial decision may be whether to measure flows and quality and provide information to the model as boundary conditions or to attempt to model them. Where modelling is required then the corresponding bathometry data must be collected.

4.3.6.2.3 Transport

Either description or prediction or transport is essential to all waste load allocation studies. All mechanistic waste load allocation models are based on mass balance principles, and both concentrations and flows are required to compute mass rates of change. For example, a loading to the system is expressed in units of mass/time, not concentration. Essential physical data required for prediction of description of transport are listed as follows:

Morphometry Data:	Channel Geometry, "roughness" or bottom type.
Hydrodynamic Data:	Water surface elevations Velocity and direction Incoming flow Point and distributed flows

Meteorological Data: Solar radiation
Air temperature
Precipitation
Wind speed and direction
Wave height, period and direction
Relative humidity
Cloud cover

Water Quality Data: Salinity
Water temperatures
Suspended sediments
Dye studies

The type of data used to quantify transport depends upon the model application and the characteristics of the system.

For complex estuaries, time varying flows, depths, and cross sections will make estimation of flows and dispersion from field data difficult. Then the flows have to be measured, estimated from dye studies, estimated by trial and error methods, or obtained from hydrodynamics studies. However these parameters are determined they must adequately reflect the flushing characteristics of the system. Data requirements for flow measurement and hydrodynamic modelling are discussed below.

Flow Measurement

Flow measurements can be used directly in waste load allocation models or be used to aid in the calibration and validation of hydrodynamic models. Tidal current is determined by placing a network of current meters at selected stations and depths throughout the estuary and measuring velocities over time. A tidal velocity curve can then be constructed. The data measured at different points can be integrated over space (i.e. laterally or vertically) and/or time depending on the needs of the water quality model. Data from the flow measurements should be evaluated when incorporated into models to insure that continuity is maintained and that constituents are properly transported.

Freshwater inflow measurements may be available for major tributaries from Federal or Provincial agencies. The frequency at which data are required must be assessed in the context of how rapidly flows are changing. Generally, hourly and often daily data are sufficient. Flows must be estimated for ungauged tributaries and where the influence of ungauged tributaries is appreciable, a flow monitoring program initiated. Groundwater inflows or flows from direct runoff may be estimated from flow gauges available in the fluvial portion of most large drainage basins. Inflows from point source discharge including municipal and industrial sources and combined sewer overflows are essential input to any model.

Dye Studies

Dye and time of travel studies are often one of the better sources of data for estimating dispersion coefficients, computing transport or for calibration and confirmation data for hydrodynamic models. Dye studies can be conducted with injections toward the mouth of the estuary or in areas where there is the greatest uncertainty in model predictions. For example, dye studies can be used to estimate mixing in the freshwater portion of a tidal river where no salinity gradients occur.

The type of dye study conducted varies with the study objectives. Studies may involve continuous or slug releases of the tracer dye. Continuous discharges are particularly useful in estimating steady-state dilution levels while slug studies are often useful for estimating dispersion coefficients or for calibrating and testing hydrodynamic models.

Continuous tracer studies generally release dye over one or more tidal cycles or discharge periods, which is then monitored within the estuary at selected locations over a series of tidal cycles. Monitoring of continuous dye releases may be continuous or concentrate on initial dilution and successive slack tides to obtain wastewater dilution levels for initial dilution, high and low slack tides or tidally averaged conditions.

Some caution should be exercised in that dyes injected at a point will have different travel times from those mixed over the modeled dimensions. For example, for a one-dimensional (longitudinal) model it may be preferable to distribute the dye as a vertically mixed band across the estuary.

A variety of dye types may have been used. The most common dye presently in use is Rhodamine WT. Generally boat mounted continuous flow fluorometers can be best used to locate and track a dye cloud or to obtain dye concentrations at discrete stations. Some consideration should be given to the toxicity of the dye as well as to its degradation by chlorine in studies of treatment facilities or its absorption onto particulates and macrophytes. Rhodamine WT is also slightly more dense than water and may require adjustment to obtain neutral buoyancy. The background florescence should be determined to aid in determining quantities of dye to be released and subtracted from field measurements. Care should also be exercised to schedule dye studies to avoid non-representative meteorological conditions.

4.3.6.2.4 Water Quality

The water quality data required, beyond that needed to quantify transport will vary depending on how the variables will be used and their anticipated impacts on the waste load allocation analysis. In addition, the water quality data required will vary depending on the anticipated response time of the system to changes in the value of the variable. For example, processes that vary over long time scales, in relation to the period of modelling, are often assumed to have a constant effect over the period of simulation. Sediment oxygen demand and sediment release rates are often treated in this way.

Data requirements will vary if the waste load allocation is intended for dissolved oxygen, eutrophication or toxics. Variables critical for an analysis of toxicity, such as pH for ammonia and metals, may not be required if the parameter of interest is DO. If the waste load is not expected to impact particular variables, such as pH, then it may be sufficient to use available data to determine their effects. If however, data are not available for conditions of interest, or if the variable is expected to change, either directly or indirectly, in response to the loading, then modelling may be required as well as collection of additional supporting data.

Without limiting the number of possible water quality parameters, some of the more commonly measured variables include:

Salinity or Conductivity

Ammonia

Nitrogen

Temperature
Suspended Solids
Dissolved Oxygen
BOD_5
Total Phosphorus
Total Kjeldahl Nitrogen
Nitrate-Nitrogen
Nitrite-Nitrogen
pH
Toxicity
Coliform Bacteria

Some variables, such as dissolved oxygen (DO) are suggested for all studies. DO can provide general information about the estuaries capacity to assimilate polluting materials and support aquatic life. The specific type of data for a particular application will vary depending on the level of effort required for the study. Concentrations for all pertinent water quality variables should be provided at the model boundaries, as well as at stations within the model system.

Measurements of processes impacting water quality may be required in addition to concentration measurements. For example, strongly sorbed contaminants are strongly affected by sediment interactions, including re-suspension, settling, and sedimentation. Some independent measurement of these processes may be required to reduce model uncertainty.

4.3.6.2.5 Station Selection

All stations for data collection should be well described and documented in order to insure that they are re-established during subsequent sampling periods. Stations can be established using an easily determined distance from some permanent structure or landmark. However, care should be exercised to insure that the stations are not located near some structure which would make them unrepresentative. Stations can be relocated using electronic positioning equipment such as range instruments, radar or Loran if they are sufficiently accurate to allow relocation within an acceptable distance. Methods should be established for maintaining positions at stations during sampling. Records of arrival and departure times for each site as well as surface observations should be made during each sampling period.

4.3.7 Waste Load Allocation Modelling

4.3.7.1 General

Because of the wide array of variable elements that must be considered in assessing a receiving water's assimilative capacity, computerized mathematical models are generally employed to make the necessary calculations. In the simplest situations, manual calculations can be performed. In most cases, however, the use of computerized mathematical models will be much more convenient.

4.3.7.2 Model Selection

4.3.7.2.1 General

The initial step of any waste load allocation study is to define the nature and the extent of the problem. Once this is done, the preferred approach in model selection is to use the simplest model that can be applied to a particular case. Ideally, the model should include only those phenomena that are operative and important in the receiving water being modeled. The most appropriate procedure for selecting a model is, therefore, to first define the phenomena that are important for the particular site-specific analysis to be performed. Activities that help to define phenomena that should be incorporated include the following:

a) review of existing data on waste loads, and receiving water quality;

b) preliminary mass balance calculations using simple models or equations that provide analytical solutions for various load sources (combined sewer overflows, nonpoint sources, sediment) and reaction phenomena.

It is also desirable to attempt to anticipate the technical issues with respect to control actions (level of treatment, alternate discharge locations, etc.) and determine whether this will influence the types of reactions that will be important. From the foregoing, the analyst will generally be able to establish the phenomena that should be included in the selected model and the time and space scale of the analysis which is most appropriate.

Under ideal circumstances, one would select a formal model or analysis approach that included all the phenomena determined to be important in the study area, and which excluded those reactions that are insignificant in the case in question. While this guidance should be followed as much as possible, in practice a calculation framework or model may be selected because it is available or familiar to the analyst.

In such cases, two criteria are important to apply. First, the model selected must be capable of handling all of the important site-specific phenomena considering the time and space scale of the analysis and using the equations and formulations specified. Secondly, provision should be made, where possible, to eliminate from the calculation framework the effect of any phenomena that are insignificant in the site-specific analysis. In some cases, inclusion of phenomena judged to be unimportant on a site-specific basis can increase the level of uncertainty of the analysis and thus directly affect decisions. In these situations, additional data collection, sensitivity runs, and other aspects of the overall waste load allocation program must be considered, in order that phenomena contained in the calculations are adequately addressed.

Additional evaluation criteria for model selection include completeness of computer program documentation, costs for manpower, and computer time.

4.3.7.2.2 Model Selection Guidelines

Guidelines for selection of a model fall under two categories: technical and operational. The technical guidelines ultimately are concerned with matching the model capabilities to the important physical and biochemical processes of the prototypical system. The operational guidelines are concerned with the ease and cost associated with model operation.

The following is the sequence of model selection guidelines, with a brief discussion of the considerations involved.

Technical Guideline #1

Determine Important Features of the Prototypical System That are Required in the Analysis

Site-specific data should be collected and reviewed to understand the system and establish the important factors associated with the identified problem. Valuable information can also be obtained from other experienced professionals, especially those who have modelling experience or site-specific field experience, and from personal site visits.

Technical Guideline #2

Review Available Models and Model Capabilities

There are a wide number of models available capable of performing waste load allocations. It is important to be aware of those capabilities that involve a substantial increase in complexity.

Technical Guideline #3

Match Important Features of the Prototypical System With Model Capabilities

An important step in model selection is comparing the important features of the prototypical system with the model capabilities and selecting, as technically acceptable, those models whose capabilities match the features of the system. A rule of thumb is to select the simplest model(s) that retains all important features in the prototypical system. Choosing a more complex model is not cost effective since data requirements and computer cost tend to increase rapidly. An overly complex program will not usually result in an improved simulation and may increase uncertainty in the analysis.

Technical Guideline #4

Confirm Selection of Technically Acceptable Models

To confirm that the models are indeed technically appropriate, the potential user should consult the user's manual and other support documents, contact and discuss the potential application with members of the support agency, and consult with other experienced professionals.

Operational Guideline #1

Selection of Candidate Models Based on Ease of Application

Once a technically acceptable model has been selected, it is necessary to estimate the ease of applying it. However, it is very difficult to evaluate the adequacy of documentation and support and realistically estimate costs without prior experience with the model. Therefore, it is recommended that the support agency be consulted. It may be possible that special support arrangements (including short courses or informational or personnel exchanges) are available under existing agreements or otherwise could be made available to the potential user. The support agency may also be able to provide the potential user with a list of local users who could be contacted for information regarding their past or current experience with the computer program associated with the model.

Operational Guideline #2

Selection of Candidate Models Based on Cost of Application and Problem Significance

It is difficult to estimate overall costs involved in a model application because each application differs in scope and complexity, and the ability to solve or avoid certain problems is very dependent on the experience and technical background of the analysts involved. However, machine requirements and costs associated with typical runs are usually estimated in the program documentation. As a rule, the simpler the model, the less expensive it is to apply. Again, it is essential that the support agency and other experienced professionals be contacted for information or assistance.

Once an estimate of the costs of application has been made, it should be compared with the benefits of using the program as part of the water quality modelling effort and the overall importance of the problem. In other words, the WLA study costs should be consistent with the economic, social, or environmental values associated with the problem and its solution.

Operational Guideline #3

Selection of Candidate Models Based on Data Availability and Data Acquisition Costs

All models require data for input, calibration, and verification. It is best if model selection is not restricted by availability of data and the decision is made to acquire the specific type of data required for the model. On the other hand, if data availability is a constraint, selection of a less sophisticated model than would be warranted on technical grounds may be appropriate.

SUMMARY:

The first step in model selection is to determine which programs are technically acceptable, based on an understanding of the important physical and biochemical processes in the prototypical system. The second step is to determine the ease and costs of application of those which are technically acceptable. The result of the second step is a list of candidate models which may or may not be ranked according to convenience and cost. The final selection of the preferred model from the list of candidates is based on the overall judgment of the potential user taking into account all of the factors discussed.

4.3.7.3 Modelling Procedures

Figure 2.2 outlines the typical development of a site specific water quality model.

4.3.7.3.1 Initial Assessment

The first three steps in Figure 2.2 contribute to the initial assessment activity. The historical data are reviewed and employed in conjunction with initial model runs, which compare calculated and observed water quality to:

- confirm existing or future water quality problems.
- define the loads, sources, and sinks that control water quality.
- define the important reactions that control water quality.
- define issues in the area of transport that must be resolved.

The initial assessment is the first step of the process aimed at understanding the factors controlling water quality. The initial assessment activity is a first full step in understanding quantitatively the factors controlling water quality. It is not a preliminary analysis; instead, the initial understanding is translated into a field and experimental program whose data output begins to challenge and strengthen the understanding of the system.

4.3.7.3.2 Field Program

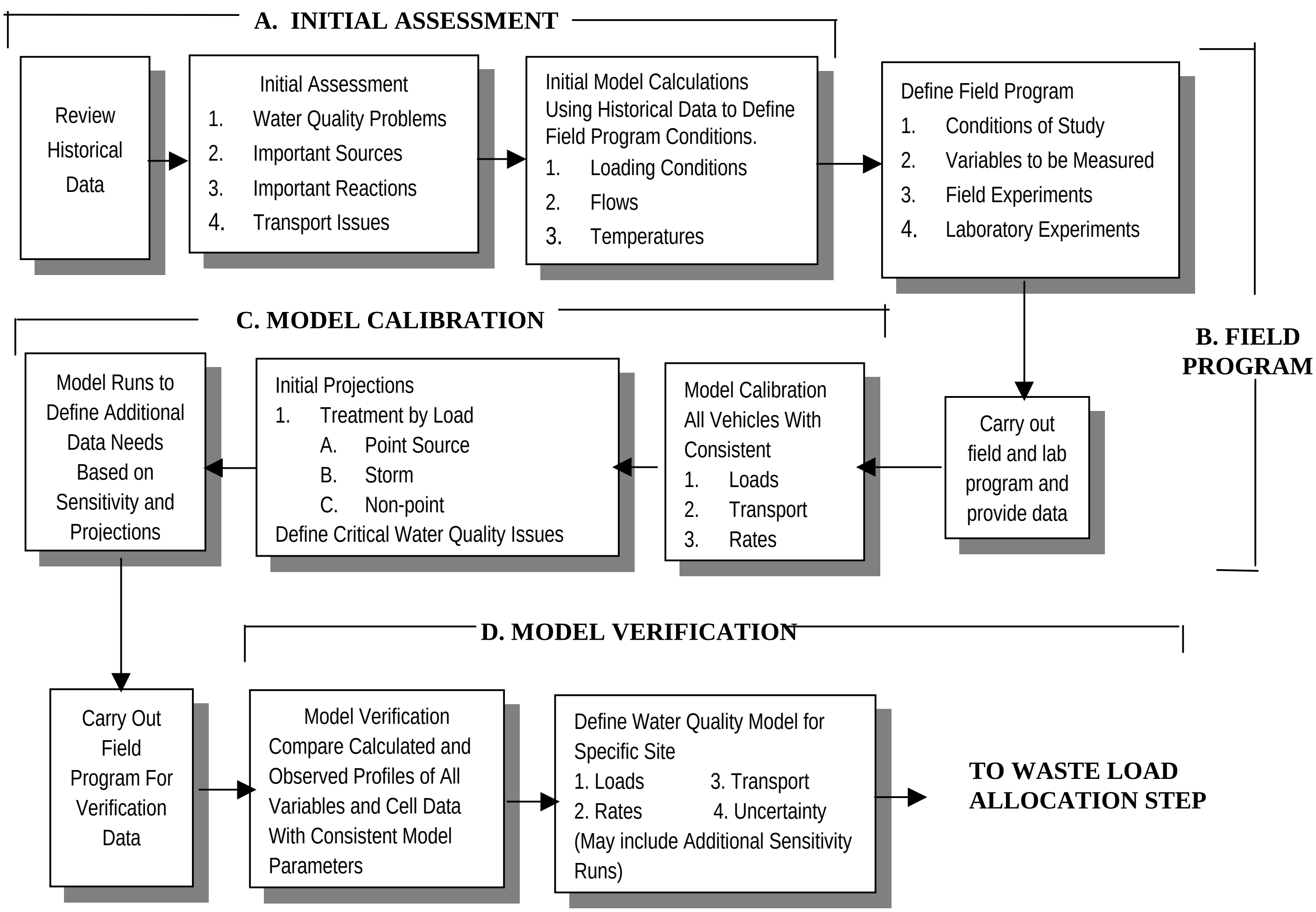
This task translates the results of the initial assessment into a practical field program that can be carried out on the receiving water and in the laboratory using the resources and manpower required and/or available.

4.3.7.3.3 Model Calibration

Following the selection of an appropriate model and the collection of the relevant field data, it is necessary to calibrate the model. Model calibration is necessary because of the semi-empirical nature of present day water quality models.

In model calibration activities, the data from the field program are employed to define model criteria, constants and equations. Water quality calculations using the model are developed for the various conditions associated with each of the water quality data sets. These conditions include those associated with the historical data and the data collected in the field program. Adjustments in the value of model parameters must be made in a consistent fashion for all conditions. The results of these activities are a set of consistent model parameters, which are then employed to develop water quality calculations for the conditions associated with all available data sets. Comparisons of calculated and observed water quality profiles should be developed. The model runs and calculations employed to search for and define the series of consistent coefficients should be retained since they can provide an indication of system sensitivity.

FIGURE 4.2
STEPS IN THE DEVELOPMENT OF A SITE SPECIFIC WATER QUALITY MODEL



4.3.7.3.4 Model Verification

At this stage in the modelling process, a calibrated model has been developed. The next step involves a test of the adequacy of the model in terms of decisions required in the waste load allocation study. Without validation testing, the calibrated model remains a description of the conditions defined by the calibration data set. The uncertainty of any projection or extrapolation of a calibrated model would be unknown unless this is estimated during the validation procedure.

Model verification efforts should contain activities similar to those discussed under model calibration. In general, model parameters should not be altered in the verification analysis. If changes in these model inputs are required, the changes should be entered for all data sets including historical, calibration, and verification data.

Comparison between observed and calculated water quality for all data sets should be developed. Sensitivity analyses should be conducted on model parameters so as to determine which parameters have the greatest impact on model predictions.

The next step in the process is to define a site-specific water quality model that consists of:

- A single set of model parameters that were developed and used in the calibration and verification analysis. These parameters should be uniform in space and time varying only as defined below.
- A set of rules for variation of model parameters in terms of measured information, such as temperature, flow, loads, geometry, etc. The rules for variations of parameters should be those used in the calibration and verification activities.
- A range of values for model parameters that cannot be adequately defined by a single value. The range of parameters, determined from sensitivity analysis, should be used in all projections.
- The quantitative and qualitative measures of model adequacy, including graphs, statistics and appropriate discussions.

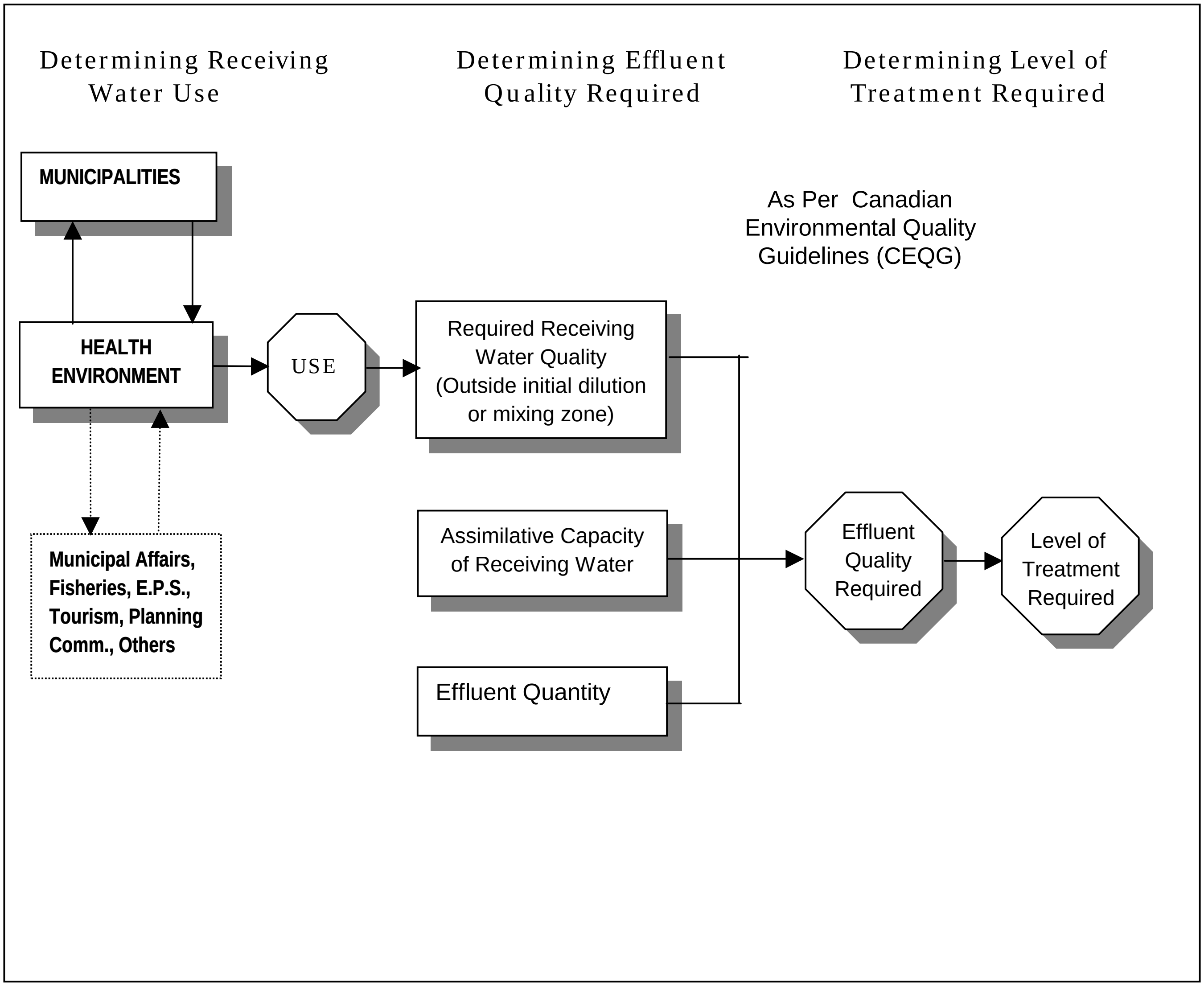
4.3.7.3.5 Allocating Waste Loads

The purpose of the waste load allocation analysis is to define the quantity of waste that may be discharged into a receiving water while meeting the water quality objectives.

The initial requirement is to quantitatively define the critical conditions that will control waste load allocations. There may be one or more critical conditions that should be considered.

Once the critical conditions are established, the calibrated model can be used to predict the water quality response due to various loads. With the model in place, the regulatory agencies shall determine the receiving water use, and thus the required receiving water quality (as outlined in the CCREM guidelines). By varying the loads from the calibrated model, it is then possible to determine required effluent quality and the associated level of treatment required. This procedure is illustrated in Figure 2.3.

FIGURE 4.3
SCHEMATIC REPRESENTATION OF PROCEDURE FOR ESTABLISHING REQUIRED LEVEL OF TREATMENT



4.4 GENERAL DESIGN REQUIREMENTS

4.4.1 Type of Treatment

A process should be capable of providing the necessary treatment and effluent discharge control to protect the adjacent and receiving environment.

As a minimum, the following items shall be considered in the selection of the type of treatment:

a. present and future effluent requirements;

b. location of and local topography of the plant site;

c. space available for future plant construction;

d. the effects of industrial wastes likely to be encountered;

e. ultimate disposal of sludge;

f. system capital costs;

g. system operating and maintenance costs, including basic energy requirements;

h. process complexity governing operating personnel requirements;

i. environmental impact on present and future adjacent land use;

j. sewage characteristics and the results of any treatability or pilot plant studies; and

k. reliability of the process and the potential for malfunctions or bypassing needs.

4.4.2 Engineering Data for New Process Evaluation

The policy of the reviewing authority is to encourage rather than obstruct the development of any methods or equipment for the treatment of wastewater. The lack of inclusion in these standards of some types of wastewater treatment processes or equipment should not be construed as precluding their use. The reviewing authority may approve other types of wastewater treatment processes and equipment under the condition that the operational reliability and effectiveness of the process or device shall have been demonstrated with a suitably-sized prototype unit operating at its design load conditions, to the extent required.

The reviewing authority may require the following:

a. monitoring observations, including tests results and engineering evaluations, demonstrating the efficiency of such processes;

b. detailed description of the test methods;

c. testing, including appropriately composite samples, under various ranges of strength and flow rates (including diurnal variations) and waste temperatures over a sufficient length of time to demonstrate performance under climatic and other conditions which may be encountered in the area of the proposed installations; and

d. other appropriate information.

The reviewing authority may require that appropriate testing be conducted and evaluations be made under the supervision of a competent process engineer other than those employed by the manufacturer or developer.

4.4.3 Design Loads

4.4.3.1 Design Period

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

- population growth rates;
- prevailing financing interest rates;
- inflation rates;
- ease of expansion of facilities; and
- time requirements for design and construction or expansion.

Wherever possible, sewage treatment plants should be designed for the flows expected to be received 20 years hence, under normal growth conditions. In certain cases, where it can be shown that staging of construction will be economically advantageous, lesser design periods may be used provided it can be demonstrated that the required capacity can be "on line" when needed.

4.4.3.2 Hydraulic Design

Flow conditions critical to the design of the treatment plant are described in Section 1.3.5.8.

Initial low flow conditions must be evaluated in the design to minimize operational problems with freezing, septicity, flow measurements and solids dropout. The design peak hourly flows must be considered in evaluating unit processes, pumping, piping, etc.

The treatment plant design flow selected shall meet the appropriate effluent and water quality standards that are set forth in the permit to operate. The design of treatment units that are not subject to peak hourly flow requirements shall be based on the design average flow. For plants subject to high wet weather flows or overflow detention pumpback flows, the design maximum day flows that the plant is to treat on a sustained basis should be specified.

4.4.3.2.1 New Systems

Hydraulic Capacity for Wastewater Facilities to serve New Collection Systems.

i) The sizing of wastewater facilities receiving flows from new wastewater collection systems shall be based on an average daily flow of 340 L per capita plus wastewater flow from industrial plants and major institutional and commercial facilities unless water use data or other justification upon which to better estimate flow is provided.

ii) The 340 L/cap·d figure shall be used, in conjunction with an extraneous flow allowances (see Section 2.3) intended to cover infiltration.

iii) If the new collection system is to serve existing development the likelihood of I/I contributions from existing service lines and non-wastewater connections to those service lines shall be evaluated and wastewater facilities designed accordingly.

4.4.3.2.2 *Existing Systems*

Hydraulic Capacity for Wastewater Facilities to serve existing Collection Systems:

i) Projections shall be made from actual flow data to the extent possible.

ii) The probable degree of accuracy of data and projections shall be evaluated. This reliability estimation should include an evaluation of the accuracy of existing data, as well as an evaluation of the reliability of estimates of flow reduction anticipated due to infiltration/inflow (I/I) reduction or flow increases due to elimination of sewer bypasses and backups.

iii) Critical data and methodology used shall be included. It is recommended that graphical displays of critical peak wet weather flow data (refer to Section 1.3.5.8) be included for a sustained wet weather flow period of significance to the project.

iv) At least one year's flow data should be taken as the basis for determining the various critical flow conditions.

4.4.3.2.3 *Wet Weather Flows*

If unusually high flows are encountered during wet weather periods, a thorough investigation of the collection system should be made and a program for corrective action initiated.

4.4.3.2.4 *Flow Equalization*

Facilities for the equalization of flows and organic shock loads shall be considered at all plants which are critically affected by surge loadings. The sizing of the flow equalization facilities should be based on data obtained herein and from Section 2.3.

4.4.3.3 *Organic Design Loads*

4.4.3.3.1 *Organic Load Definitions and Identification*

The following organic loads for the design year shall be identified and used as a basis for design of wastewater treatment facilities. Where any of the terms defined in this Section are used in these design standards, the definition contained in this Section applies.

a. Biochemical Oxygen Demand Defined

The 5-day Biochemical Oxygen Demand (BOD_5) is defined as the amount of oxygen required to stabilize biodegradable organic matter under aerobic conditions within a five day period in accordance with latest edition of **Standard Methods**. Total 5-day Biochemical Oxygen demand ($TBOD_5$) is equivalent to BOD_5 and is sometimes used in order to differentiate carbonaceous plus nitrogenous oxygen demand from strictly carbonaceous oxygen demand.

The carbonaceous 5-day Biochemical Oxygen Demand ($CBOD_5$) is defined as BOD_5 less the nitrogenous oxygen demand of the wastewater. See the latest edition **Standard Methods**.

b. Design Average BOD_5

The design average BOD_5 is generally the average of the organic load received for a continuous 12 month period for the design year expressed as weight per day. However, the design average BOD_5 for facilities having critical seasonal high loading periods (e.g., recreational areas, campuses, industrial facilities) shall be based on the daily average BOD_5 during the seasonal period.

c. Design Maximum Day BOD_5

The design maximum day BOD_5 is the largest amount of organic load to be received during a continuous 24 hour period expressed as weight per day.

d. Design Peak Hourly BOD_5

The design peak hourly BOD_5 is the largest amount of organic load to be received during a one hour period expressed as weight per day.

4.4.3.3.2 New Systems

Organic Capacity of Wastewater Treatment Facilities to Serve New Collection Systems.

a. Domestic waste treatment design shall be on the basis of at least 0.08 kg of BOD_5 per capita per day and 0.09 kg of suspended solids per capita per day, unless information is submitted to justify alternate designs.

b. When garbage grinders are used in areas tributary to a domestic treatment plant, the design basis should be increased to 0.10 kg of BOD_5 per capita per day and 0.11 kg pounds of suspended solids per capita per day.

c. Where appreciable amounts of industrial waste are included, consideration shall be given to the character of wastes in the design of the plant.

d. Data from similar municipalities may be utilized in the case of new systems. However, thorough investigation that is adequately documented shall be provided to the reviewing authority to establish the reliability and applicability of such data.

4.4.3.3.3 Existing Systems

Design of Organic Capacity of Wastewater Treatment Facilities to Serve Existing Collection Systems:

a. Projections shall be made from actual wasteload data to the extent possible. Laboratory analyses shall be made on composite samples taken over 24 hour periods. This data shall include composite samples for the maximum significant period of sewage and industrial waste discharge, and shall cover a significant period of time to be representative of actual conditions. It is recommended that the designing engineer confer with the reviewing agency for details concerning the collection and analysis of samples. Recognition should be given to flow patterns for institutions, schools, motels, etc.

b. Projections shall be compared to Section 4.4.3.3.2 and an accounting made for significant variations from those values.

c. Impact of industrial sources shall be documented. For projects with significant industrial contributions, pretreatment may be required.

4.4.3.4 Shock Effects

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

4.4.3.5 Design by Analogy

Data from similar municipalities may be utilized in the case of new systems; however, a thorough investigation that is adequately documented shall be provided to the reviewing authority to establish the reliability and applicability of such data.

4.4.3.6 Design Capacity of Various Plant Components (Without Flow Equalization)

In general, all components of mechanical sewage treatment plants should be hydraulically capable of handling the anticipated peak sewage flow rates without overtopping channels and/or tankage. From a process point-of-view, however, the design of various sections of sewage treatment plants should be based upon the following hydraulic, organic and inorganic loading rates:

Sewage Pumping Stations

- peak hourly flow rate.

Screening

- peak flow rate.

Grit Removal

- peak flow rate, peak grit loading rate.

Primary Sedimentation

- peak flow rate, peak suspended solids loading rate.

Aeration (without nitrification)

- average diurnal BOD_5 loading rate is usually sufficient with predominantly domestic wastes, but the presence of significant industrial waste loadings may create sufficient diurnal variations to warrant consideration. Daily or seasonal variations in domestic and/or industrial BOD loading rates should be taken into consideration. Except for short detention treatment systems, such as contact stabilization or high rate processes, hydraulic detention time is seldom critical.

Aeration (with nitrification)

- average diurnal BOD_5 loading rate is usually sufficient with predominantly domestic wastes, but the presence of significant industrial waste loadings may create sufficient BOD diurnal variations to warrant consideration. Diurnal peak flow rate and diurnal peak ammonia (total Kjeldahl nitrogen with extended aeration) loading rates must be designed for. Daily or seasonal variations in BOD_5, ammonia, total Kjeldahl (with extended aeration) and peak flow rates should also be taken into consideration.

Secondary Sedimentation

- peak hourly flow rate or peak solids loading rate, whichever governs.

Sludge Return

- capacity requirements will vary with the treatment system (see Section 6.1.4).

Disinfection Systems

- peak flow rate.

Chemical Feed Systems

- peak flow rate.

Effluent Filtration

- peak flow rate, peak solids loading rate.

Outfall Sewer

- peak flow rate.

Sludge Treatment (digestion, thickening, dewatering, incineration, etc.)

- average loading rates (hydraulic, total solids, volatile solids) unless sustained peaks are of significance to the individual treatment process.

4.4.4 Conduits

All piping and channels should be designed to carry the maximum expected flows. The incoming sewer should be designed for unrestricted flow. Bottom corners of the channels must be filleted. Conduits shall be designed to avoid creation of pockets and corners where solids can accumulate. Suitable gates should be placed in channels to seal off unused sections which might accumulate solids. The use of shear gates or stop planks is permitted where they can be used in place of gate valves or sluice gates. Non-corrodible materials shall be used for these control gates.

4.4.5 Flow Division Control

Flow division control facilities shall be provided as necessary to insure organic and hydraulic loading control to plant process units and shall be designed for easy operator access, change, observation and maintenance. Appropriate flow measurement shall be incorporated in the flow division control design.

4.4.6 Wastewater Flow Measurement

Facilities for measuring and recording all wastewater flows through the treatment works shall be provided. All plant and process unit bypasses should also be equipped with flow measuring devices, such that hydraulic balances around each treatment process unit and the total plant are possible. Flow measuring devices should be located so that the flows measured are meaningful and recordable.

4.4.6.1 Location

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

a. Plant influent or effluent flow;

b. Plant influent flow: If influent flow is significantly different from effluent flow, both shall be measured. This would apply for installations such as stabilization ponds, and plants with excess flow storage or flow equalization;

c. Excess flow treatment facility discharges;

d. Other flows required to be monitored under the provisions of the permit to operate; and

e. Other flows such as returned activated sludge, waste activated sludge, recirculation, and recycle required for plant operational control.

4.4.6.2 Facilities

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

4.4.6.3 Hydraulic Conditions

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

4.4.7 Component Back-up Requirements

The components of sewage 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.

To achieve this, critical treatment processes should be provided in multiple units so that with the larger unit out of operation, the hydraulic capacity (not necessarily the design rated capacity) of the remaining units shall be sufficient to handle the peak wastewater 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. Similarly, it should be possible to distribute the flow of all of the units in the treatment process downstream of the affected process. In addition, where feasible, it should be possible to operate the sections of treatment plants as completely separate process trains to allow full-scale loading tests to be carried out.

4.4.8 Sampling Equipment

Effluent composite sampling equipment shall be provided at all mechanical plants with a design average flow of 380 m^3/day or greater and at other facilities where necessary to meet "permit to operate" monitoring requirements. Composite sampling equipment shall also be provided as needed for influent sampling and for monitoring plant operations.

4.4.9 Plant Hydraulic Gradient

The hydraulic gradient of all gravity flow and pumped waste streams within the sewage treatment plant, including by-pass 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, Wye branches and tees;

d. head losses due to sudden changes in slope, or drops;

e. head losses due to obstructions in conduits;

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;

k. head allowances for expansion requirements and/or process changes; and

l. head allowances due to maximum water levels in receiving waters.

4.4.10 Arrangement of Units

Component parts of the plant should be arranged for greatest operating and maintenance convenience, flexibility, economy, continuity of maximum effluent quality and ease of installation of future units.

4.5 PLANT DETAILS

4.5.1 Installation of Mechanical Equipment

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

4.5.2 By-Passes

4.5.2.1 General

Except where duplicate units are available, properly located and arranged by-pass structures shall be provided so that each unit of the plant can be removed from service independently. The by-pass design shall facilitate plant operation during unit maintenance and emergency repair so as to minimize deterioration of effluent quality and insure rapid process recovery upon return to normal operational mode. By-pass systems should also be constructed so that each unit process can be separately by-passed.

4.5.2.2 Unit By-Pass During Construction

Final plan documents shall include construction requirements as deemed necessary by the reviewing agency to avoid unacceptable temporary water quality degradation.

4.5.3 Overflows

If sewage entering the treatment plant must be pumped into the treatment units, an emergency overflow for the pumping station should be provided, if it is physically possible (reference to section 3.3.2). The purpose of this overflow is to prevent basement flooding by back-ups in the sewer system in the event of pumping station failure. Wherever possible, this overflow should be routed through the chlorine contact chamber and plant outfall sewer. If this is not possible, provision should be made for chlorination of such overflows.

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

4.5.4 Drains

Means shall be provided to dewater each unit to an appropriate point in the process. Due consideration shall be given to the possible need for hydrostatic pressure relief devices to prevent flotation of structures. Pipes subject to clogging shall be provided with means for mechanical cleaning or flushing.

4.5.5 Construction Materials

Due consideration should be given to the selection of materials which are to be used in sewage treatment works because of the possible presence of hydrogen sulphide and other corrosive gases, greases, oils and similar constituents frequently present in sewage. This is particularly important in the selection of metals and paints. Dissimilar metals should be avoided to minimize galvanic action.

4.5.6 Painting

The use of paints containing lead or mercury should be avoided. In order to facilitate identification of piping, particularly in the large plants, it is suggested that the different lines be color-coded. The following color scheme is recommended for purposes of standardization.

Raw sludge line - brown with black bands
Sludge recirculation suction line - brown with yellow bands
Sludge draw off line - brown with orange bands
Sludge recirculation discharge line - brown
Sludge gas line - orange (or red)
Natural gas line - orange (or red) with black bands
Nonpotable water line - blue with black bands
Potable water line - blue
Chlorine line - yellow
Sulfur Dioxide - yellow with red bands
Sewage (wastewater) line - grey
Compressed air line - green
Water lines for heating digesters or buildings - blue with a 150 mm red band spaced 760 mm apart

The contents and direction of flow shall be stencilled on the piping in a contrasting color.

4.5.7 Operating Equipment

A complete outfit of tools and accessories and spare parts necessary for the plant operator's use shall be provided. A portable pump is desirable. Readily accessible storage space and work bench facilities shall be provided and consideration given to provision of a garage area which would also provide space for large equipment, maintenance and repair.

4.5.8 Grading and Landscaping

Upon completion of the plant, the ground should be graded. Concrete or gravel walkways should be provided for access to all units. Where possible, steep slopes should be avoided to prevent erosion. Surface water shall not be permitted to drain into any unit. Particular care shall be taken to protect trickling filter beds, sludge beds and intermittent sand filters, from surface wash. Provision should be made for landscaping, particularly when a plant is located near residential areas.

4.5.9 Erosion Control During Construction

Effective site erosion control shall be provided during construction as outlined in the Nova Scotia Department of the Environment publication, "Erosion and Sedimentation Control Handbook for Construction Sites". An approved erosion control plan is required before construction begins.

4.5.10 Cathodic Protection

Steel fabricated sewage treatment plants shall require cathodic protection for corrosion control as specified in Section 3.2.11.

4.6 PLANT OUTFALLS

4.6.1 Dilution

Outfall sewers shall consist of a completely piped system conforming to the requirements of Section 1 of these guidelines and shall not discharge into any ditch or watercourse in which adequate assimilative capacity is not available. In assessing the available assimilative capacity the proximity of other outfalls must be taken into consideration. The outfall sewer shall be designed to discharge to the receiving water in a manner acceptable to the reviewing authority. Consideration should be given to each of the following: a) utilization of cascade aeration of effluent discharge to increase dissolved oxygen levels and b) limited or complete across-stream dispersion as needed to protect aquatic life movement and growth in the immediate reaches of the receiving stream.

4.6.2 Outlet

The outfall sewer, where practicable, shall be extended to the low water level of the receiving body of water in such a manner to insure the satisfactory dispersion of the effluent thereto and insofar as practicable, it shall have its outlet submerged. Where greater depths are available, one meter should be the achieved depth of submergence.

4.6.3 Protection and Maintenance

The outfall sewer shall be so constructed and protected against the effects of flood water, tides, ice or other hazards as to reasonably insure its structural stability and freedom from stoppage.

A manhole should be provided at the shore end of all gravity sewers extending into the receiving waters, complete with a flow measuring device.

Hazards to navigation must be considered in designing outfall sewers.

4.6.4 Dispersion of Flow

Where conditions exist that a point discharge of effluent could have deleterious effects on the receiving body of water, consideration shall be given to providing a means of effective submerged dispersion of the effluent into the water course.

4.6.5 Sampling Provisions

All outfalls shall be designed so that a sample of the effluent can be obtained at a point after the final treatment process and before discharge to or mixing with the receiving water.

4.7 ESSENTIAL FACILITIES

4.7.1 Emergency Power Facilities

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

- reliability of primary power source;
- number of power feeder lines supplying grid system, number of alternate routes within the grid system, and the number of alternate transformers through which power could be directed to the sewage treatment plant;
- whether sewage enters the plant by gravity or is pumped;
- type of treatment provided;
- pieces of equipment which may become damaged or overloaded following prolonged power failure;
- assimilation capacity of the receiving waters and ability to withstand higher pollution loadings over short time periods; and
- other uses of the receiving water.

Each specific installation should provide for the following considerations:

- means for illuminating working areas to ensure safe working conditions; and
- standby power source or equivalent to power pumps, motorized valves and control panels that are necessary to maintain the sewage flow through the treatment plant.

Standby generating capacity normally is not required for aeration equipment used in the activated sludge process. In cases where a history of long-term (4 hours or more) power outages have occurred, auxiliary power for minimum aeration of the activated sludge will be required. Full power generating capacity may be required by the reviewing authority on certain critical stream segments.

4.7.2 Water Supply

4.7.2.1 General

An adequate supply of potable water under pressure shall be provided for use in the laboratory, chlorination equipment and general cleanliness around the plant. The chemical quality should be checked for suitability for its intended uses such as heat exchangers, chlorinators, etc.

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. If a potable water supply is brought to the plant, it shall be protected with a suitable backflow prevention device.

4.7.2.2 *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 sink;
b. water closet;
c. laboratory sink;
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 coils.

4.7.2.3 *Indirect Connections*

Where a potable water supply is to be used for any purpose in a plant other than those listed in Section 4.7.2.2, 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 or sill cock located on the water system beyond the break tank to indicate that the water is not safe for drinking.

Consideration will also be given to backflow devices consisting of a system of check valves and relief valves which provide protection against backflow.

4.7.2.4 *Separate Potable Water Supply*

Where it is not possible to provide potable water from a public water supply, a separate well may be provided as long as sufficient pressure is available. Location and construction of the well should comply with requirements of the regulatory authorities. Requirements governing the use of the supply are those contained in Sections 4.7.2.2 and 4.7.2.3.

4.7.2.5 *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 sill cocks and hose bibs shall be posted with a permanent sign indicating the water is not safe for drinking.

4.7.3 Sanitary Facilities

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

4.7.4 Floor Slope

Floor surfaces shall be sloped adequately to a point of drainage.

4.7.5 Stairways

Stairways should be installed with a slope of 30 to 35 degrees from the horizontal to facilitate carrying samples, tools, etc. All risers in a stairway should be of equal height. Minimum tread run shall not be less than 200 mm. The sum of the tread run and riser shall not be less than 430 mm nor more than 460 mm. A flight of stairs shall consist of not more than 3 m continuous rise without a platform. Stairways shall be installed wherever possible in lieu of ladders.

4.8 SAFETY

4.8.1 General

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:

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

b. Hand rails and guards around tanks, trenches, pits, stairwells, and other hazardous structures with the tops of walls less that 1 m above the surrounding ground level;

c. Gratings over appropriate areas of treatment units where access for maintenance is required;

d. First aid equipment;

e. "No Smoking" signs in hazardous areas;

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

g. Portable blower and sufficient hose;

h. Portable lighting equipment complying with the National and Provincial Electrical Code requirements;

i. Gas detectors;

j. 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.;

k. Adequate ventilation in pump station areas in accordance with Section 3.2.7.

l. Provisions for local lockout on stop motor controls; and

m. Provisions for confined space entry in accordance with regulatory agency requirements.

4.8.2 Hazardous Chemical Handling

Reference should be made to Federal "Transportation of Dangerous Goods Act" and the Provincial "Dangerous Goods and Hazardous - Wastes Management Act".

4.8.2.1 Contaminant Materials

The materials utilized for storage, piping, valves, pumping, metering, splash guards, etc., shall be specially selected considering the physical and chemical characteristics of each hazardous or corrosive chemical.

4.8.2.2 Primary Containment

Structures, rooms, and areas accommodating chemical storage and feed equipment should be arranged to provide convenient access for chemical deliveries, equipment servicing and repair, and observation of operation. It is recommended that wherever possible the storage area be separated from the main plant, and that segregated storage be provided for each chemical. Where two, or more, chemicals could react with undesirable effects, the drainage piping (if provided) from the separate chemical handling areas should not be interconnected. For dangerous materials such as gaseous chlorine, either floor drains in the storage and scale rooms should be omitted entirely, with the floors sloped towards the doors, or floor drains installed, but kept totally separated from the drainage systems for the rest of the building.

4.8.2.3 Secondary Containment

Chemical storage areas shall be enclosed in dykes or curbs which are capable of containing 110% of the stored volume until it can be safely transferred to alternate storage or released to the wastewater at controlled rates which will not damage facilities, inhibit the treatment processes or contribute to stream pollution. Liquid polymer should be similarly contained to reduce areas with slippery floors, especially to protect travel-ways. Non-slip floor surfaces are desirable in polymer handling areas.

4.8.2.4 Underground Storage

Underground storage and piping facilities for fuels or for chemicals such as alum or ferric chloride shall be constructed in accordance with applicable provincial and federal regulations on underground storage tanks for both fuels and hazardous materials.

4.8.2.5 Liquified Gas Chemicals

Properly designed isolated areas shall be provided for storage and handling of chlorine and sulfur dioxide and other hazardous gases. Gas detection kits, alarms, controls, safety devices, and emergency repair kits shall also be provided.

4.8.2.6 Eye Wash Fountains and Safety Showers

Eye wash fountains and safety showers utilizing potable water shall be provided in the laboratory and on each floor level or work location involving hazardous or corrosive chemical storage, mixing (or shaking), pumping, metering, or transportation unloading. These facilities are to be as close as practical to possible chemical exposure sites and are to be fully useful during all weather conditions.

The eye wash fountains shall be supplied with water of moderate temperature (10 to 30°C), separate from the hot water supply, suitable to provide 15 to 30 minutes of continuous irrigation of the eyes.

The emergency showers shall be capable of discharging 2 to 4 litres per second of water at moderate temperature at pressures of 140 to 350 kPa. The eye wash fountains and showers shall be not more than 7.0 m from points of hazardous chemical exposure.

4.8.2.7 *Splash Guards*

All pumps or feeders for hazardous or corrosive chemicals shall have guards which will effectively prevent spray of chemicals into space occupied by personnel. The Splash Guards are in addition to guards to prevent injury from moving or rotating machinery parts.

4.8.2.8 *Piping, Labelling, Coupling Guards, Location*

All piping containing or transporting corrosive or hazardous chemicals shall be identified with labels every 3 m and with at least two labels in each room, closet or pipe chase. Colour coding may also be used but is not an adequate substitute for labelling. All connections (flanged or other type), except adjacent to storage or feeder areas, shall have guards which will direct any leakage away from space occupied by personnel. Pipes containing hazardous or corrosive chemicals should not be located above shoulder level except where continuous drip collection trays and coupling guards will eliminate chemical spray or dripping on to personnel.

4.8.2.9 *Protective Clothing or Equipment*

The following items of protective clothing or equipment shall be available and utilized for all operations or procedures where their use will minimize injury hazard to personnel:

a. self-contained air supply system recommended for protection against chlorine;

b. chemical worker's goggles or other suitable goggles (safety glasses are insufficient);

c. face masks or shields for use over goggles;

d. rubber gloves;

e. rubber aprons with leg straps;

f. rubber boots (leather and wool clothing should be avoided near caustics); and

g. safety harness and line.

h. dust mask to protect the lungs in dry chemical areas.

4.8.2.10 *Warning Systems and Signs*

Facilities shall be provided for automatic shut-down of pumps and sounding of alarms when failure occurs in a pressurized chemical discharge line.

Warning signs requiring use of goggles shall be located near chemical unloading stations, pumps and other points of frequent hazard.

4.8.2.11 Dust Collection

Dust collection equipment shall be provided to protect personnel from dusts injurious to the lungs or skin and to prevent polymer dust from settling on walkways. The latter is to minimize slick floors which result when a polymer covered floor becomes wet.

4.8.2.12 Container Identification

The identification and hazard warning data included on shipping containers, when received, shall appear on all containers (regardless of size or type) used to store, carry or use a hazardous substance. Sewage and sludge sample containers should be adequately labelled. Below is a suitable label for sewage sample:

RAW SEWAGE **Sample point no. _______** **Contains Harmful Bacteria.** **May contain hazardous or toxic material.** **Do not drink or swallow.** **Avoid contact with openings or breaks in the skin.**

4.9 LABORATORY

4.9.1 General

All treatment works shall include a laboratory for making the necessary analytical determinations and operating control tests, except in individual situations where the omission of a laboratory is approved by the reviewing agency. The laboratory shall have sufficient size, bench space, equipment and supplies to perform all self-monitoring analytical work required by the Permit to Operate and to perform the process control tests necessary for good management of each treatment process included in the design.

The facilities and supplies necessary to perform analytical work to support industrial waste control programs will normally be included in the same laboratory. The laboratory size and arrangement must be sufficiently flexible and adaptable to accomplish these assignments. The layout should consider future needs for expansion in the event that more analytical work is 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.

Treatment plant laboratory needs may be divided into the following three general categories:

I. Plants performing only basic operational testing; this typically includes pH, temperature, and dissolved oxygen;

II. Plants performing more complex operational and permit laboratory tests including biochemical oxygen demand, suspended solids, and fecal coliform analysis, and;

III. Plants performing more complex operational, permit, industrial pretreatment, and multiple plant laboratory testing.

Expected minimum laboratory needs for these three plant classifications are outlined in this section. However, in specific cases laboratory needs may have to be modified or increased due to the industrial monitoring needs or special process control requirements.

4.9.2 Category I: Plants performing only basic operational testing.

4.9.2.1 Location and Space

A floor area up to 14 m^2 should be adequate. It is recommended that this be at the treatment site. Another location in the community utilizing space in an existing structure owned by the involved sewer authority may be acceptable.

4.9.2.2 Design and Materials

The facility shall provide for electricity, water, heat, sufficient storage space, a sink, and a bench top. The lab components need not be of industrial grade materials. Laboratory equipment and glassware shall be of types recommended by Standard Methods for the Examination of Water and Wastewater and the reviewing authority.

4.9.3 Category II: Plants performing more complex operational and permit laboratory tests including biochemical oxygen demand, suspended solids, and fecal coliform analysis.

4.9.3.1 Location and Space

The laboratory size should be based on providing adequate room for the equipment to be used. In general, the laboratories for this category of plant should provide a minimum of approximately 28 m^2 of floor space. The laboratory should be located at the treatment site on ground level. It shall be isolated away from vibrating, noisy, high-temperature machinery or equipment which might have adverse effects on the performance of laboratory staff or instruments.

4.9.3.2 Floors

Floor surfaces should be fire resistant, and highly resistant to acids, alkalies, solvents, and salts.

4.9.3.3 Cabinets and Bench Tops

Laboratories in this category usually perform both the permit testing and operational control monitoring utilizing "acids" and "bases" in small quantities, such that laboratory grade metal cabinets and shelves are not mandatory. The cabinets and shelves selected may be of wood or other durable materials. Bench tops should be of acid resistant laboratory grade materials for protection of the non-acid proof cabinets. Glass doors on wall-hung cabinets are not required. One or more cupboard style base cabinets should be provided. Cabinets with drawers should have stops to prevent accidental removal. Cabinets for Category II laboratories are not required to have gas, air, vacuum, and electrical service fixtures. Built-in shelves should be adjustable.

4.9.3.4 Fume Hoods, Sinks, and Ventilation

4.9.3.4.1 Fume Hoods

Fume hoods shall be provided for laboratories in which required analytical works results in the production of noxious fumes.

4.9.3.4.2 Sinks

A laboratory grade sink and drain trap shall be provided.

4.9.3.4.3 Ventilation

Laboratories should be air conditioned. In addition, separate exhaust ventilation should be provided.

4.9.3.5 Balance and Table

An analytical balance of the automated digital readout, single pan 0.1 mg sensitivity type shall be provided. A heavy special-design balance table which will minimize vibration of the balance is recommended. It shall be located as far as possible from windows, doors, or other sources of drafts or air movements, so as to minimize undesirable impacts from these sources upon the balance.

4.9.3.6 Equipment, Supplies, and Reagents

The laboratory shall be provided with all of the equipment, supplies, and reagents that are needed to carry out all of the facility's analytical testing requirements. If any required analytical testing produces malodorous or noxious fumes, the engineer should verify that the in-house analysis is more cost-effective than use of an independent off-site laboratory. Composite samples may be required to satisfy permit sampling requirements. Permit to operate, process control, and industrial waste monitoring requirements should be considered when specifying equipment needs. References such as Standard Methods for the Examination of Water and Wastewater and the U.S.E.P.A. Analytical Procedures Manual should be consulted prior to specifying equipment items.

4.9.3.7 Utilities

4.9.3.7.1 Power Supply

Consideration should be given to providing line voltage regulation for power supplied to laboratories using delicate instruments.

4.9.3.7.2 Laboratory Water

Reagent water of a purity suitable for analytical requirements shall be supplied to the laboratory. In general, reagent water prepared using an all glass distillation system is adequate. However, some analyses require deionization of the distilled water. Consideration should be given to softening the feed water to the still.

4.9.3.8 Safety

4.9.3.8.1 Equipment

Laboratories shall provide as a minimum the following: first aid equipment; protective clothing including goggles, gloves, lab aprons, etc.; and a fire extinguisher.

4.9.3.8.2 Eyewash Fountains and Safety Showers

Eyewash fountains and safety showers utilizing potable water shall be provided in the laboratory and should be as close as practical and shall be no more than 7.0 m from points of hazardous chemical exposure.

The eyewash fountains shall be supplied with water of moderate temperature 10° to 30° C (50° to 90° F) suitable to provide 15 minutes to 30 minutes of continuous irrigation of the eyes. The emergency showers shall be capable of discharging 2 to 4 L/s of water at a moderate temperature and at pressure of 140 to 350 Kpa.

4.9.4 Category III: Plants performing more complex operational, permit, industrial pretreatment and multiple plant laboratory testing.

4.9.4.1 Location and Space

The laboratory should be located at the treatment site on ground level, with environmental control as an important consideration. It shall be located away from vibrating, noisy, high temperature machinery or equipment which might have adverse effects on the performance of laboratory staff or instruments.

The laboratory facility needs for Category III plants should be described in the engineering design report or facilities plan. The laboratory floor space and facility layout should be based on an evaluation of complexity, volume, and variety of sample analyses expected during the design life of the plant including testing for process control, industrial pretreatment control, user charge monitoring, and the permit to operate monitoring requirements.

Consideration should be given to the necessity to provide separate (and possibly isolated) areas for some special laboratory equipment, glassware, and chemical storage. At large plants, office and administrative space needs should be considered.

For less complicated laboratory needs bench-top working surface should occupy at least 35 percent of the total laboratory floor space. Additional floor and bench space should be provided to facilitate performance of analysis of industrial wastes, as required by the permit to operate and the utility's industrial waste pretreatment program. Ceiling height should be adequate to provide for the installation of wall mounted water stills, deionizers, distillation racks, hoods, and other equipment with extended height requirements.

4.9.4.2 Floor and Doors

4.9.4.2.1 Floors

Floor surfaces should be fire resistant, and highly resistant to acids, alkalies, solvents, and salts.

4.9.4.2.2 Doors

Two exit doors should be located to permit a straight egress from the laboratory, preferable at lease one to outside the building. Panic hardware should be used. They should have large glass windows for easy visibility of approaching or departing personnel.

Automatic door closers should be installed; swinging doors should not be used.

Flush hardware should be provided on doors if cart traffic is anticipated. Kick plates are also recommended.

4.9.4.3 Cabinets and Bench Tops

4.9.4.3.1 Cabinets

Wall-hung cabinets are useful for dust-free storage of instruments and glassware. Units with sliding glass doors are preferable. A reasonable proportion of cupboard style base cabinets and drawer units should be provided.

Drawers should slide out so that entire contents are easily visible. They should be provided with rubber bumpers and with stops which prevent accidental removal. Drawers should be supported on ball bearings or nylon rollers which pull easily in adjustable steel channels. All metals drawer fronts should be double-wall construction.

All cabinet shelving should be acid resistant and adjustable. The laboratory furniture shall be supplied with adequate water, gas, air, and vacuum service fixtures, traps, strainers, plugs, and tailpieces, and all electrical service fixtures.

4.9.4.3.2 Bench Tops

Bench tops should be constructed of materials resistant to attacks from normally used laboratory reagents. Generally, bench-top height should be 900 mm. However, areas to be used exclusively for sit-down type operations should be 760 mm high and include kneehole space. Twenty-five millimetre overhangs and drip grooves should be provided to keep liquid spills from running along the face of the cabinet. Tops should be furnished in large sections, 32 mm thick. They should be field-jointed into a continuous surface with acid, alkali, and solvent-resistant cements which are at least as strong as the material of which the top is made.

4.9.4.4 Hoods

4.9.4.4.1 General

Fume hoods to promote safety and canopy hoods over heat-releasing equipment shall be provided.

4.9.4.4.2 Fume Hoods

a. Location

Fume hoods should be located where air disturbance at the face of the hood is minimal. Air disturbance may be created by persons walking past the hood; by heating, ventilating, or air-conditioning systems; by drafts from opening or closing a door, etc.

Safety factors should be considered in locating a hood. If a hood is situated near a doorway, a secondary means or egress must be provided. Bench surfaces should be available next to the hood so that chemicals need not be carried long distances.

b. Design and Material

The selection, design, and materials of construction of fume hoods and their appropriate safety alarms must be made by considering the variety of analytical work to be performed. The characteristics of the fumes, chemicals, gases, or vapours that will or may be released by the activities therein should be considered. Special design and construction is necessary if perchloric acid use is anticipated. Consideration should be given to providing more than one fume hood to minimize potential hazardous conditions throughout the laboratory.

Fume hoods are not appropriate for operation of heat-releasing equipment that does not contribute to hazards, unless they are provided in addition to those needed to perform hazardous tasks.

c. Fixtures

One sink should be provided inside each fume hood. A cup sink is usually adequate.

All switches, electrical outlets, and utility and baffle adjustment handles should be located outside the hood. Light fixtures should be explosion-proof.

d. Exhaust

Twenty-four hour continuous exhaust capability should be provided. Exhaust fans should be explosion-proof. Exhaust velocities should be checked when fume hoods are installed.

4.9.4.4.3 Canopy Hoods

Canopy hoods should be installed over the bench-top areas where hot plate, steam bath, or other heating equipment or heat-releasing instruments are used. The canopy should be constructed of heat and corrosion resistant material.

4.9.4.5 Sinks, Ventilation, and Lighting

4.9.4.5.1 Sinks

The laboratory should have a minimum of two sinks (not including cup sinks). At least one of them should be a double-well sink with drainboards. Additional sinks should be provided in separate work areas as needed, and identified for the use intended.

Sinks should be made of epoxy resin or plastic materials highly resistant to acids, alkalies, solvents, and salts, and should be abrasion and heat resistant, non-absorbent, and light in weight. Traps should be made of glass, plastic, or lead and easily accessible for cleaning. Waste openings should be located toward the back so that a standing overflow will not interfere.

All water fixtures on which hoses may be used should be provided with reduced zone pressure backflow preventers to prevent contamination of water lines.

4.9.4.5.2 Ventilation

Laboratories should be separately air conditioned, with external air supplied for one hundred percent make-up volume. In addition, separate exhaust ventilation should be provided. Ventilation outlet locations should be remote from ventilation inlets. Consideration should be given to providing dehumidifiers.

4.9.4.5.3 Lighting

Good lighting, free from shadows, must be provided for reading dials, meniscuses, etc., throughout the laboratory.

4.9.4.6 Balance and Table

An analytical balance of the automatic, digital readout, single pan, 0.1 mg sensitivity type shall be provided. A heavy special-design balance table which will minimize vibration of the balance is needed. It shall be located as far as practical from windows, doors, or other sources of drafts or air movements, so as to minimize undesirable impacts from these sources upon the balance.

4.9.4.7 Equipment, Supplies, and Reagents

The laboratory shall be provided with all of the equipment, supplies, and reagents that are needed to carry out all of the facility's analytical testing requirements. Composite samplers may be required to satisfy permit sampling requirements. Permit to operate, process control, and industrial waste monitoring requirements should be considered when specifying equipment needs. Reference such as Standard Methods for the Examination of Water and Wastewater and the U.S.E.P.A. Analytical Procedures Manual should be consulted prior to specifying equipment items.

4.9.4.8 Utilities and Services

4.9.4.8.1 Power Supply

Consideration should be given to providing line voltage regulation for power supplied to laboratories using delicate instruments.

4.9.4.8.2 Laboratory Water

Reagent water of a purity suitable for analytical requirements shall be supplied to the laboratory. In general, reagent water prepared using an all glass distillation system is adequate. However, some analyses require deionization of the distilled water. Consideration should be given to softening water to the still.

4.9.4.8.3 Gas and Vacuum

Natural or LP gas should be supplied to the laboratory. Digester gas should not be used.

An adequately-sized line source of vacuum should be provided with outlets available throughout the laboratory.

4.9.4.9 Safety

4.9.4.9.1 Equipment

Laboratories shall be provide the following: first aid equipment; protective clothing and equipment such as goggles, safety glasses, full face shields, gloves, etc.; fire extinguishers; chemical spill kits; posting of "No Smoking" signs in hazardous area; and appropriately placed warning signs for slippery areas, non-potable water fixtures, hazardous chemical storage areas, flammable fuel storage areas, etc.

4.9.4.9.2 Eyewash Fountains and Safety Showers

Eyewash fountains and safety showers utilizing potable water shall be provided in the laboratory. These facilities are to be as close as practical and shall be no more than 7.0 m from points of hazardous chemical exposure.

The eyewash fountains shall be supplied with water of moderate temperature 10° to 30° C, suitable to provide 15 minutes to 30 minutes of continuous irrigation of the eyes. The emergency showers shall be capable of discharging 2 to 4 L/s of water at moderate temperature and at pressures of 140 to 350 kPa.

5.1 SCREENING DEVICES

5.1.1 Bar Racks and Screens

5.1.1.1 *Where Required*

Coarse bar racks or screens shall be provided as the first treatment stage for the protection of plant equipment against reduced operating efficiency, blockage, or physical damage.

5.1.1.2 *Selection Considerations*

When considering which types of screening devices should be used, the following factors should be considered:

- effect on downstream treatment and sludge disposal operations;
- possible damage to comminutor or barminutor devices caused by stones or coarse grit particles;
- head losses of the various alternative screening devices;
- maintenance requirements;
- screenings disposal requirements, and quantities of screenings; and
- requirements for a standby unit.

5.1.1.3 *Location*

5.1.1.3.1 *Outdoors*

Screening devices installed outside shall be protected from freezing.

5.1.1.3.2 *Indoors*

Screening devices installed in a building where other equipment or offices are located should be separated from the rest of the building, provided with separate outside entrances and provided with adequate means of ventilation.

5.1.1.3.3 *Access*

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

5.1.1.3.4 *Ventilation*

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 12 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 be provided in accordance with Section 4.8.

5.1.1.4 *Design and Installation*

5.1.1.4.1 Bar Spacing

a) Manually Cleaned Screens

Clear openings between bars should be from 25 mm to 45 mm. Design and installation shall be such that they can be conveniently cleaned.

b) Mechanical Screens

Clear openings for mechanically cleaned screens may be as small as 15 mm.

Mechanical screens are recommended where the installation is not regularly supervised or where an increase in head results in plant bypass.

5.1.1.4.2 Velocities

At the design average rate of flow, the screen chamber should be designed to provide a velocity through the screen of approximately 0.3 meters per second to prevent settling, and a maximum velocity during wet weather periods no greater than 0.75 meters per second to prevent forcing material through the openings. The velocity shall be calculated from a vertical projection of the screen openings on the cross-sectional area between the invert of the channel and the flow line.

5.1.1.4.3 Invert

The screen channel invert should be 75 to 150 mm below the invert of the incoming sewers. To prevent jetting action, the length and/or construction of the screen channel shall be adequate to re-establish hydraulic flow pattern following the drop in elevation.

5.1.1.4.4 Slope

Manually cleaned screens, except those for emergency use, should be placed on a slope of 30 to 45 degrees with the horizontal.

5.1.1.4.5 Channels

The channel preceding and following the screen shall be shaped to eliminate stranding and settling of solids and should be designed to provide equal and uniform distribution of flow to the screens. Dual channels shall be provided and equipped with the necessary gates to isolate flow from any screening unit. Provisions shall also be made to facilitate dewatering each unit.

5.1.1.4.6 Flow Measurement

Flow measurement devices should be provided at each screen channel. They should be selected based on reliability and accuracy. The effect of changes in backwater elevations, due to intermittent cleaning of screens, should be considered in locating of flow measurement equipment.

5.1.1.5 *Safety*

5.1.1.5.1 Railings and Gratings

Manually cleaned screen channels shall be protected by guard railings and deck gratings, with adequate provisions for removal or opening to facilitate raking.

Mechanically cleaned screen channels shall be protected by guard railings and deck gratings. Consideration should also be given to temporary access arrangements to facilitate maintenance and repair.

5.1.1.5.2 Mechanical Devices

Mechanical screening equipment shall have adequate removal enclosures to protect personnel against accidental contact with moving parts and to prevent dripping in multi-level installations.

A positive means of locking out each mechanical device and temporary access for use during maintenance shall be provided.

5.1.1.5.3 Drainage

Floor design and drainage shall be provided to prevent slippery areas.

5.1.1.5.4 Lighting

Suitable lighting shall be provided in all work and access areas. Refer to Section 5.1.1.6.2.

5.1.1.6 *Control Systems*

5.1.1.6.1 Timing Devices

All mechanical units which are operated by timing devices should be provided with auxiliary controls which will set the cleaning mechanism in operation at preset high water elevation. If the cleaning mechanism fails to lower the high water, a warning should be signalled.

5.1.1.6.2 Electrical Systems and Components

Electrical systems and components (i.e. motors, lights, cables, conduits, switchboxes, control circuits, etc.) in enclosed or partially enclosed spaces where flammable mixtures occasionally may be present (including all space above raw or partially treated wastewater) shall comply with the Canadian Electrical Code, Part 1 and the regulations under the applicable Provincial Power Standards. All electrical components in the headworks room must be explosion proof.

5.1.1.6.3 Manual Override

Automatic controls shall be supplemented by a manual override.

5.1.1.7 *Screenings Removal and Disposal*

A convenient and adequate means for removing screenings shall 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 must be provided for handling, storage, and disposal of screenings in a manner acceptable to the regulatory agency. Separate grinding of screenings and return to the sewage flow is unacceptable.

Manually cleaned screening facilities shall include an accessible platform from which the operator may rake screenings easily and safely. Suitable drainage facilities shall be provided for both the platform and storage area.

5.1.1.8 *Auxiliary Screens*

Where mechanically operated screening or comminuting devices are used, auxiliary manually cleaned screens shall be provided. Where two or more mechanically cleaned screens are used, the design shall provide for taking any unit out of service without sacrificing the capability to handle the peak design flow.

5.1.2 Fine Screens

5.1.2.1 *General*

Fine screens may be used in lieu of primary sedimentation providing that subsequent treatment units are designed on the basis of anticipated screen performance. Fine screens should not be considered equivalent to primary sedimentation. Where fine screens are used, additional provision for the removal of floatable oils and greases shall be considered. Selection of screen capacity should consider flow restriction due to retained solids, gummy material, frequency of cleaning and extent of cleaning.

5.1.2.2 *Design*

Tests should be conducted to determine BOD_5 and suspended solids removal efficiencies at the design maximum day flow and design maximum day BOD_5 loadings. Pilot testing for an extended time is preferred.

A minimum of two fine screens shall be provided, each unit being capable of independent operation. Capacity shall be provided to treat design peak instantaneous flow with one unit out of service.

Fine screens shall be preceded by a mechanically cleaned bar screen or other protective device. Comminuting devices shall not be used ahead of fine screens. Fine screens shall be protected from freezing and located to facilitate maintenance.

5.1.2.3 *Electrical Fixtures and Control*

Electrical fixtures and controls in screening areas where hazardous gases may accumulate shall comply with the Canadian Electrical Code and the regulations under the applicable Provincial Power Standards.

5.1.2.4 *Servicing*

Hosing equipment (with hot and cold water) shall be provided to facilitate cleaning. Provision shall be made for isolating or removing units from their location for servicing.

5.2 COMMINUTORS / GRINDERS

5.2.1 General

Provisions for location shall be in accordance with those for screening devices, Section 5.1.1.3.

5.2.2 When Required

Comminutors or grinders shall be used in plants that do not have primary sedimentation or fine screens and should be provided in cases where mechanically cleaned bar screens will not be used.

5.2.3 Design Considerations

5.2.3.1 *Location*

Comminutors or grinders should be located downstream of any grit removal equipment and be protected by a coarse screening device. Consideration for a different sequence may be given to suit individual cases.

5.2.3.2 *Size*

Comminutor or grinder capacity shall be adequate to handle the design peak hourly flow.

5.2.3.3 *Installation*

A screened bypass channel shall be provided. The use of the bypass channel should be automatic at depths of flows exceeding the design capacity of the comminutor.

Each comminutor or ginder that is not preceded by grit removal equipment should be protected by a 150 mm deep gravel trap.

Gates shall be provided in accordance with Section 5.1.1.4.5.

5.2.3.4 *Servicing*

Provision shall be made to facilitate servicing units in place and removing units from their location for servicing.

5.2.3.5 *Electrical Controls and Motors*

Electrical equipment in comminutor chambers where hazardous gases may accumulate shall comply with the Canadian Electrical Code and applicable Provincial Power Standards.

Motors in areas not governed by this requirement may need protection against accidental submergence.

5.3 GRIT REMOVAL FACILITIES

5.3.1 When Required

Grit removal is required in advance of treatment units to prevent the undue wear of machinery and the unwanted accumulation of solids in channels, settling tanks and digesters.

Grit removal facilities should be provided for all sewage treatment plants and are required for plants receiving sewage from combined sewers or from sewer systems receiving substantial amounts of grit. If a plant, serving a separate sewer system, is designed without grit facilities, the design shall include provisions for future installation. Consideration shall be given to possible damaging effects on pumps, comminutors and other preceding equipment and the need for additional storage capacity in treatment units where grit is likely to accumulate.

5.3.2 Location

Grit removal facilities should be located ahead of pumps and comminuting devices. Coarse bar racks should be placed ahead of grit removal facilities.

5.3.3 Accessibility

Consideration should be given in the design of grit chambers to provide safe access to the chamber and, where mechanical equipment is involved, to all functioning parts.

5.3.4 Ventilation

Where installed indoor, uncontaminated air shall be introduced continuously at a rate of 12 air changes per hour, or intermittently at a rate of 30 air changes per hour. Odor control facilities may also be warranted.

5.3.5 Electrical

Electrical equipment in grit removal areas where hazardous gases may accumulate shall comply with the Canadian Electrical Code and the regulations under the applicable provincial Power Standards.

5.3.6 Outside Facilities

Grit removal facilities located outside shall be protected from freezing.

5.3.7 Design Factors

5.3.7.1 *Inlet*

Inlet turbulence shall be minimized.

5.3.7.2 *Type and Number of Units*

Grit removal facilities (channel type) should have at least two hand-cleaned units, or a mechanically cleaned unit with bypass. A single manually cleaned or mechanically cleaned grit chamber with bypass is acceptable for small sewage treatment plants serving separate sanitary sewer systems. Minimum facilities for larger plants serving separate sanitary sewers should be at least one mechanically cleaned unit with a bypass. Facilities other than channel-types are desirable if provided with adequate and flexible controls for agitation and/or air supply devices and with grit collection and removal equipment.

5.3.7.3 *Grit Channels*

5.3.7.3.1 *Velocity*

Channel-type chambers shall be designed to provide controlled velocities as close as possible to 0.30 meters per second for normal variation in flow.

5.3.7.3.2 *Control Sections*

Flow control sections shall be of the proportional or Sutro Weir type.

5.3.7.3.3 *Channel Dimensions*

The minimum channel width shall be 375 mm. The minimum channel length shall be that required to settle a 0.2 mm particle with a specific gravity of 2.65, plus a fifty (50) per cent allowance for inlet and outlet turbulence.

5.3.7.3.4 *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 (weir plates that are capable of vertical adjustment are preferred since they can be moved to prevent the sedimentation of organic solids following grit cleaning). Grit storage is also a function of the frequency of grit removal.

5.3.7.4 *Detritus Tanks*

Detritus tanks should be designed with sufficient surface area to remove a 0.2 mm, or smaller, particle with a specific gravity of 2.65 at the expected peak flow rate. Detritus tanks, since they are mechanically-cleaned and do not need dewatering for cleaning, do not require multiple units, unless economically justifiable.

Separation of the organics from the grit before, during, or after the removal of the settled contents of the tank can be accomplished in one of the following ways:

- the removed detritus can be washed in a grit washer with the organic laden wash water being returned to the head of the detritus tank;
- a classifying-type conveyor can be used to remove the grit and return the organics to the detritus tank;
- the removed detritus can be passed through a centrifugal-type separator.

5.3.7.5 *Aerated Grit Tanks*

Aerated grit tanks for the removal of 0.2 mm, or larger, particles with specific gravity of 2.65, should be designed in accordance with the following parameters:

5.3.7.5.1 *Detention Time*

Detention time shall be 2 to 5 minutes at the peak sewage flow rate.

5.3.7.5.2 *Air Supply*

Air supply rates should be in the range of 4.5 to 12.4 L/s per linear meter of tank. The higher rates should be used with tanks of large cross-section (i.e. greater than 3.6 m deep). Air supply should be via air diffusers (wide band diffusion header) positioned lengthwise along one wall of the tank, 600 to 900 mm above the tank bottom. Air supply should be variable.

5.3.7.5.3 *Inlet Conditions*

Inlet flow should be parallel to induced roll in tank. There shall be a smooth transition from inlet to circulation flow.

5.3.7.5.4 *Baffling*

A minimum of one transverse baffle near the outlet weir shall be provided. Additional transverse baffles in long tanks and longitudinal baffles in wide tanks should be considered.

5.3.7.5.5 *Outlet Conditions*

The outlet weir shall be oriented parallel to the direction of induced roll (i.e. at a right angle to the inlet).

5.3.7.5.6 ***Tank Dimensions***

The lower limit of the above aeration rates are generally suitable for tanks up to 3.7 m deep and 4.3 m wide. Wider or deeper tanks require aeration rates in the upper end of the above range. Long, narrow aerated grit tanks are generally more efficient than short tanks and produce a cleaner grit. A length to width ratio of 2:5 to 5:1 is desirable. Depth to width ratios of 1:1.5 to 1:2 are acceptable.

5.3.7.5.7 ***Velocity***

The surface velocity in the direction of roll in tanks should be 0.45 to 0.6 m/s (tank floor velocities will be approximately 75 per cent of above). The velocity across the floor of the tank shall not be less than 0.3 m/s.

5.3.7.5.8 ***Tank Geometry***

"Dead spaces" in aerated grit tanks are to be avoided. Tank geometry is critical with respect to the location of the air diffusion header, sloping tank bottom, grit hopper and fitting of the grit collector mechanism into the tank structure. Consultation with Equipment Suppliers is advisable.

5.3.7.5.9 ***Multiple Units***

Multiple units are generally not required unless economically justifiable, or where the grit removal method requires bypassing of the tank (as with clam shell bucket).

5.3.7.6 ***Mechanical Grit Chambers***

Specific design parameters for mechanical grit chambers will be evaluated on a case-by-case basis.

5.3.7.7 ***Grit Washing***

The need for grit washing should be determined by the method of final grit disposal.

5.3.7.8 ***Dewatering***

Provision shall be made for isolating and dewatering each unit. The design shall provide for complete draining and cleaning by means of a sloped bottom equipped with a drain sump.

5.3.7.9 ***Water***

An adequate supply of water under pressure shall be provided for cleanup.

5.3.8 **Grit Removal**

Grit facilities located in deep pits should be provided with mechanical equipment for pumping or hoisting grit to ground level. Such pits should have a stairway, approved-type elevator or manlift, adequate ventilation and adequate lighting.

5.3.9 **Grit Handling**

Grit removal facilities located in deep pits should be provided with mechanical equipment for hoisting or transporting grit to ground level. Impervious, non-slip, working surfaces with adequate drainage shall be provided for grit handling areas. Grit transporting facilities shall be provided with protection against freezing and loss of material.

5.3.10 **Grit Disposal**

Disposal of grit in sanitary landfills or lagoons, as well as grit incineration shall be considered acceptable disposal methods. Whatever method of disposal is employed, the full spectrum of environmental considerations must be embodied in the final design.

5.4 PRE-AERATION AND FLOCCULATION

5.4.1 General

Pre-aeration of raw wastewater, may be used to achieve one or more of the following objectives:

a. Odor control;
b. Grease separation and increased grit removal;
c. Prevention of septicity;
d. Grit separation;
e. Flocculation of solids;
f. Maintenance of DO in primary treatment tanks at low flows;
g. Increased removals of BOD and SS in primary units; and
h. Minimizes solids deposits on side walls and bottom of wetwells.

Flocculation of sewage with or without coagulating aids, is worthy of consideration when it is desired to reduce the strength of sewage prior to subsequent treatment. Also, flocculation may be beneficial in pre-treating sewage containing certain industrial wastes.

5.4.2 Arrangement

The units should be designed so that removal from service will not interfere with normal operation of the remainder of the plant.

5.4.3 Pre-aeration

5.4.3.1 *Air Flow Measurements*

Figure 5.1 represents air flow requirements for different periods of pre-aeration.

Pre-aeration periods should be 10 to 15 minutes if odor control and prevention of septicity are the prime objectives.

5.4.4 Flocculation

5.4.4.1 *Detention Period*

When air or mechanical agitation is used in conjunction with chemicals to coagulate or flocculate the sewage, the detention period should be about 30 minutes at the design flow. However, if polymers are used this may be varied.

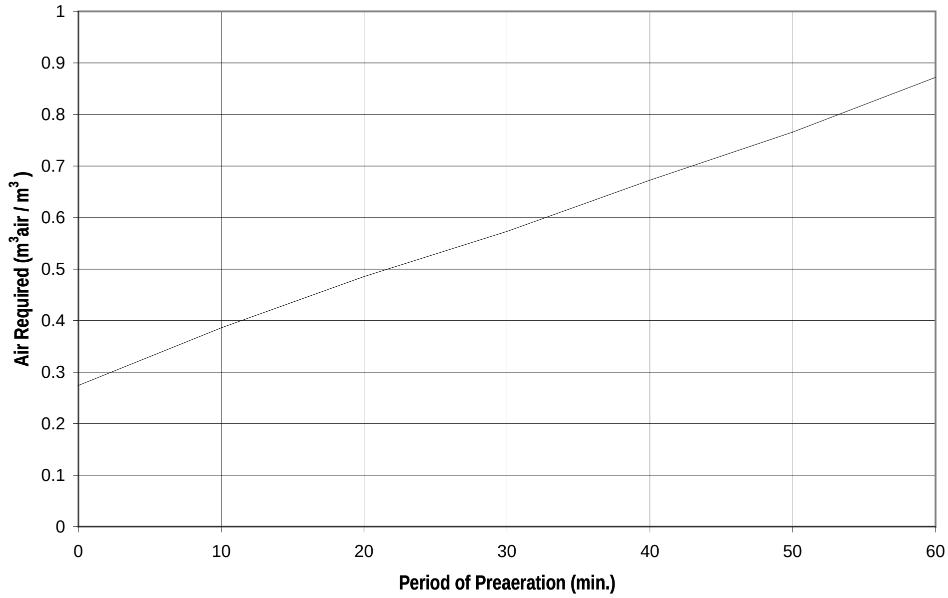
5.4.4.2 *Stirring Devices*

5.4.4.2.1 *Paddles*

Paddles should have a peripheral speed of 0.50 to 0.75 meters per second to prevent deposition of solids.

Figure 5.1

Air Flow Required for Different Periods of Preaeration



5.4.4.2.2 *Aerators*

Any of the types of equipment used for aerating activated sludge may be utilized. It shall be possible to control agitation, to obtain good mixing and maintain self-cleaning velocities across the tank floor.

5.4.4.3 *Details*

Inlet and outlet devices should be designed to insure proper distribution and to prevent short-circuiting. Convenient means should be provided for removing grit.

5.4.4.4 *Rapid Mix*

At plants where there are two or more flocculation basins utilizing chemicals, provision shall be made for a rapid mix of the sewage with the chemical so that the sewage passing to the flocculation basins will be of uniform composition. The detention period provided in the rapid mixing chamber should be very short, one-half to three minutes.

5.5 FLOW EQUALIZATION

5.5.1 General

Flow equalization can reduce the dry-weather variations in organic and hydraulic loadings at any wastewater treatment plant. It should be provided where large diurnal variations are expected.

5.5.2 Location

Equalization basins should be located downstream of pre-treatment facilities such as bar screens, comminutors and grit chambers.

5.5.3 Type

Flow equalization can be provided by using separate basins or on-line treatment units, such as aeration tanks. Equalization basins may be designed as either in-line or side-line units. Unused treatment units, such as sedimentation or aeration tanks, may be utilized as equalization basins during the early period of design life.

5.5.4 Size

Equalization basin capacity should be sufficient to effectively reduce expected flow and load variations to the extent deemed to be economically advantageous. With a diurnal flow pattern, the volume required to achieve the desired degree of equalization can be determined from a cumulative flow plot, or mass diagram, over a representative 24-hour period. To obtain the volume required to equalize the 24-hour flow:

1. Draw a line between the points representing the accumulated volume at the beginning and end of the 24-hr period. The slope of this line represents the average rate of flow.

2. Draw parallel lines to the first line through the points on the curve farthest from the first line.

3. Draw a vertical line between the lines drawn in No. 2. The length of this line represents the minimum required volume.

5.5.5 Operation

5.5.5.1 *Mixing*

Where applicable, aeration or mechanical equipment shall be provided to maintain adequate mixing. Corner fillets and hopper bottoms with draw-offs should be provided to alleviate the accumulation of sludge and grit.

5.5.5.2 *Aeration*

Where applicable, aeration equipment shall be sufficient to maintain a minimum of 1.0 mg/l of dissolved oxygen in the mixed basin contents at all times. Air supply rates should be a minimum of 0.15 litres per second per cubic meter storage capacity. The air supply should be isolated from other treatment plant aeration requirements to facilitate process aeration control, although process air supply equipment may be utilized as a source of standby aeration.

5.5.5.3 *Controls*

Inlets and outlets for all basin compartments shall be suitably equipped with accessible external valves, stop plates, weirs or other devices to permit flow control and the removal of an individual unit from service. Facilities shall also be provided to measure and indicate liquid levels and flow rates.

5.5.6 Electrical

All electrical work in housed equalization basins shall comply with the Canadian Electrical Code and the regulations under applicable Provincial Power Standards.

5.5.7 Access

Suitable access shall be provided to facilitate cleaning and the maintenance of equipment.

Chapter 6 CLARIFICATION

6.1 SEDIMENTATION TANKS

6.1.1 General Design Requirements

The need for and the design of primary sedimentation tanks will be influenced by various factors, including the following:

- the characteristics of the raw wastewater; the type of sludge digestion systems, either available or proposed (aerobic digestion should not be used with raw primary sludges):
- the presence, or absence, of secondary treatment following primary treatment;
- the need for handling of waste activated sludge in the primary settling tank:
- the need for, or possible economic benefits through, phosphorus removal in the primary settling tank(s).

6.1.1.1 *Number of Units*

Multiple units capable of independent operation are desirable and shall be provided in all plants where design flows exceed 500 cubic meters per day. Plants not having multiple units shall include other provisions to assure continuity of treatment.

6.1.1.2 *Arrangement of Units*

Settling tanks shall be arranged in accordance with Section 4.4.10.

6.1.1.3 *Interaction with Other Processes*

a. Pumping directly to any clarifier is prohibited, unless special provision is included in the design of pump controls. Attention should be focused so that pumps deliver smooth flow transmissions at all times, with a minimal energy gradient.

b. For activated sludge plants employing high energy aeration, provisions should be made for floc to be reformed before settling.

c. For primary clarifiers, tanks and equipment must be sized to not only accommodate raw waste solids but also those solids introduced by thickener overflows, anaerobic digester overflow and sometimes waste activated sludge.

6.1.1.4 *Flow Distribution and Control*

Effective flow measurement devices and control appurtenances (i,e., valves, gates, splitter boxes, etc.) shall be provided to permit proper proportion of flow to each unit. Parallel basins should be of the same size, otherwise flow shall be distributed in proportion to surface area.

6.1.1.5 *Tank Configuration and Proportions*

Consideration should be given to the probable flow pattern in the selection of tank size and shape, and inlet and outlet type and location. Generally rectangular clarifiers are designed with length-to-width ratios of at least 4:1, and width to depth ratios of 1:1 and 2.25:1.

6.1.1.6 *Site Constraints*

The selection of feasible clarifier alternatives should include the following site considerations:

a. Wind direction;
b. Proximity to residents;
c. Soil conditions;
d. Groundwater conditions; and
e. Available space.

6.1.1.7 *Size Limitations*

Rectangular clarifiers shall have a maximum length of 90 m. Circular clarifiers shall have a maximum diameter of 60 m. The minimum length of flow from inlet to outlet shall be 3 m, unless special provisions are made to prevent short circuiting. The vertical sidewater depth shall be designed to provide an adequate separation zone between the sludge blanket and the overflow weirs.

6.1.1.8 *Inlet Structures*

Inlet structures should be designed to dissipate the inlet velocity, to distribute the flow equally and to prevent short-circuiting. Channels should be designed to maintain a velocity of at least 0.3 meters per second at one-half design average flow. Corner pockets and dead ends should be eliminated and corner fillets or channelling used where necessary. Provisions shall be made for elimination or removal of floating materials in inlet structures.

6.1.1.9 *Outlet Arrangements*

6.1.1.9.1 General

Overflow weirs shall be adjustable for levelling, and sufficiently long to avoid high heads which result in updraft currents.

6.1.1.9.2 Location

Overflow weirs shall be located to optimize actual hydraulic detention time and minimize short circuiting. Peripheral weirs shall be placed at least 0.3 m from the wall.

6.1.1.9.3 Weir Troughs

Weir troughs shall be designed to prevent submergence at maximum design flow and to maintain a velocity of at least 0.3 meters per second at one-half design average flow.

6.1.1.10 *Submerged Surfaces*

The tops of troughs, beams and similar submerged construction elements shall have a minimum slope of 1.4 vertical to 1 horizontal; the underside of such elements should have a slope of 1 to 1 to prevent the accumulation of scum and solids.

6.1.1.11 *Unit Dewatering*

Unit dewatering features shall conform to the provisions outlined in Section 4.4.3.6. The bypass design should also provide for redistribution of the plant flow to the remaining units.

6.1.1.12 *Freeboard*

Walls of settling tanks shall extend at least 150 mm above the surrounding ground surface and shall provide not less than 300 mm freeboard. Additional freeboard or the use of wind screens is recommended where larger settling tanks are subject to high velocity wind currents that would cause tank surface waves and inhibit effective scum removal.

6.1.1.13 *Clarifier Covers*

Clarifiers may be required to be covered for winter operation. The structure should be constructed with adequate head room for easy access. The structure must include adequate lighting, ventilation and heating. Humidity andcondensation shall be controlled inside the structure.

6.1.2 Types Of Settling

6.1.2.1 *Type I Settling (Discrete Settling)*

Type I settling is assumed to occur in gravity grit chambers handling wastewater and in basins used for preliminary settling (silt removal) of surface waters. A determination of the settling velocity of the smallest particle to be 100% removed is fundamental to the design of Type I clarifiers. Because each particle is assumed to settle independently and with a constant velocity, a mathematical development is possible, based on Newton's Law and Stoke's Law.

6.1.2.2 *Type II Settling (Flocculant Settling)*

Type II settling occurs when particles initially settle independently but flocculate as they proceed the depth of the tank. As a result of flocculation, the settling velocities of the aggregates formed change with time, and a strict mathematical solution is not possible. Laboratory testing is required to determine appropriate values for design parameters. Type II settling can occur during clarification following fixed-film processes, primary clarification of wastewater, and clarification of potable water treated with coagulants.

Type II settling can also occur above the sludge blanket in clarifiers following activated sludge treatment; however design procedures based on Type III settling are normally used to design these units.

6.1.2.3 *Type III Settling (Hindered or Zone Settling)*

Type III settling occurs in clarifiers following activated sludge processes and gravity thickeners. While Type II processes may occur to a limited extent in such units, it is Type III that governs design. In suspensions undergoing hindered settling, the solids concentration is usually much higher than in discrete or flocculant processes. As a result, the contacting particles tend to settle as a zone or blanket, and maintain the same position relative to each other.

6.1.3 Design Criteria

Table 6.1 outlines design parameters for sedimentation tanks based upon their associated settling type.

6.1.4 Sludge And Scum Removal

6.1.4.1 *Scum Removal*

Effective scum collection and removal facilities, including baffling, shall be provided for all settling tanks. Scum baffles are to be placed ahead of the outlet weirs and extend 300 mm below the water surface. The unusual characteristics of scum which may adversely affect pumping, piping, sludge handling and disposal, should be recognized in design. Provisions may be made for the discharge of scum with the sludge; however, other special provisions for disposal may be necessary.

6.1.4.2 *Sludge Removal*

6.1.4.2.1 *Sludge Removal*

Sludge collection and withdrawal facilities shall be designed to assure rapid removal of the sludge and minimization of density currents. Suction withdrawal should be provided for activated sludge plants designed for reduction of the nitrogenous oxygen demand and is encouraged for those plants designed for carbonaceous oxygen demand reduction. Each settling tank shall have its own sludge withdrawal lines to insure adequate control of the sludge wasting rate for each tank.

6.1.4.2.2 *Sludge Collection*

Sludge collection mechanisms shall remain in operation during sludge withdrawal. Mechanism speeds shall be such as to avoid undue agitation while still producing desired collection results.

6.1.4.2.3 *Sludge Hopper*

The minimum slope of the side walls shall be 1.7 vertical to 1 horizontal. Hopper wall surfaces should be made smooth with rounded corners to aid in sludge removal. Hopper bottoms shall have a maximum dimension of 0.6 meters. Extra depth sludge hoppers for sludge thickening are not acceptable. The hoppers are to be accessible for sounding and cleaning.

6.1.4.2.4 *Cross-Collectors*

Cross-collectors serving one or more settling tanks may be useful in place of multiple sludge hoppers.

TABLE 6.1 SEDIMENTATION BASINS DESIGN PARAMETERS

TYPE OF SETTLING	SIDEWATER DEPTH (m)	SURFACE OVERFLOW (1) RATE ($m^3/m^2.d$)	WEIR LOADING RATE $m^3/m.d$ (2) (3)	SOLIDS LOADING (kg/m^2d)
Type I and Type II	3.0 - 4.6	≤ 40 at design average flow ≤ 60 at peak hourly flow	125 - 370	N/A
Type III	3.5 - 4.6	1) Settling following activated sludge: ≤ 30 at design average flow ≤ 50 at peak hourly flow 2) Settling following extended aeration: ≤ 15 at design average flow ≤ 35 at peak hourly flow 3) Settling following fixed film processes:: ≤ 25 at design average flow ≤ 45 at peak hourly flow 4) Settling following separate nitrification: ≤ 35 at peak hourly flow	< 250 (4)	49 (at an SVI of 300) - 290 (at an SVI of 100)

1. When several different overflow criteria are given (design average flow, peak hourly flow) the clarifier area to be used in the design is the larger of those computed in each case.
2. At design average flow.
3. If pumping is required, weir loading rate should be related to pump delivery rates to avoid short-circuiting.
4. Where weirs are located so that density currents upturn below them, the rate should not exceed 186 $m^3/m.d$.

6.1.4.2.5 *Sludge Removal Piping*

Each hopper shall have an individually valved sludge withdrawal line at least 150 mm in diameter. The static head available for withdrawal of sludge shall be 750 mm or greater, as necessary to maintain a 1.0 meter per second velocity in the withdrawal line. Clearance between the end of the withdrawal line and the hopper walls shall be sufficient to prevent ‘bridging” of the sludge. Adequate provisions shall be made for rodding or back-flushing individual pipe runs. Piping shall also be provided to return waste sludge to primary clarifiers.

6.1.4.2.6 *Sludge Removal Control*

Sludge wells equipped with telescoping valves or other appropriate equipment shall be provided for viewing, sampling and controlling the rate of sludge withdrawal from each tank hopper. The use of easily maintained sight glass and sampling valves may be appropriate. A means of measuring the sludge removal rate from each hopper shall be provided. Air lift type of sludge removal will not be approved for removal of primary sludges. Sludge pump motor control systems shall include time clocks and valve activators for regulating the duration and sequencing of sludge removal.

6.2 ENHANCED PRIMARY CLARIFICATION

6.2.1 Chemical Enhancement

Chemical coagulation of raw wastewater before sedimentation promotes flocculation of finely divided solids into more readily settleable flocs, thereby increasing SS, BOD, and phosphorus removal efficiencies. Sedimentation with coagulation may remove 60 to 90% of the total suspended solids (TSS), 40 to 70% of the BOD, 30 to 60% of the chemical oxygen demand (COD), 70 to 90% of the phosphorus, and 80 to 90% of the bacteria loadings. In comparison, sedimentation without coagulation may remove only 40 to 70% of the TSS, 25 to 40% of the BOD, 5 to 10% of the phosphorus loadings, and 50 to 60% of the bacteria loading. Chapter 9 of this manual contains additional information on the selection and application of chemicals for phosphorus removal.

Advantages of coagulation include greater removal efficiencies, the ability to use higher overflow rates, and more consistent performance. Disadvantages of coagulation include an increased mass of primary sludge, production of solids that are often more difficult to thicken and dewater, and an increase in operational cost and operator attention. The designer of chemical coagulation facilities should consider the effect of enhanced primary sedimentation on downstream solids-processing facilities.

6.2.1.1 *Chemical Coagulants.*

Historically, iron salts, aluminum salts, and lime have been the chemical coagulants used for wastewater treatment. Iron salts have typically been the most common of the coagulants used for primary treatment. Only a few plants use lime as a coagulant for primary treatment since lime addition produces more primary sludge because of the chemical solids than do metals salts and lime is more difficult to store, handle, and feed. Coagulant selection for enhanced sedimentation should be based on performance, reliability, and cost. Performance evaluation should use jar tests of the actual wastewater to determine dosages and effectiveness. Operating experience, cost, and other relevant information drawn from other plants should be considered during selection. Organic polymers are sometimes used as flocculation aids.

6.2.1.2 *Rapid Mix*

During rapid mix, the first step of the coagulation process, chemical coagulants are mixed with the raw wastewater. The coagulants destabilize the colloidal particles by reducing the forces (zeta potential), keeping the particles apart, which allows their agglomeration. The destabilization process occurs within seconds of coagulant addition. At the point of chemical addition, intense mixing will ensure uniform dispersion of the coagulant throughout the raw wastewater. The intensity and duration of mixing must be controlled, however, to avoid overmixing or undermixing. Overmixing may reduce the removal efficiency by breaking up existing wastewater solids and newly formed floc. Undermixing inadequately disperses the chemical, increases chemical use, and reduces the removal efficiency.

The velocity gradient, G, is a measure of mixing intensity. Velocity gradients of 300s’ are typically sufficient for rapid mix, but some designers have recommended velocity gradients as high as 1,000 sQ Mechanical mixers, in-line blenders, pumps, baffled compartments, baffled pipes, or air mixers can accomplish rapid mix. The mixing intensity of mechanical mixers and in-line blenders is independent of flow rate, but these mixers cost considerably more than other types and might become clogged or entangled with debris. Air mixing eliminates the problem of debris and can offer advantages for primary sedimentation, especially if aerated channels or grit chambers already exist. Pumps, Parshall flumes, flow distribution structures, baffled compartments, or baffled pipes—methods often used for upgrading existing facilities—offer a lower-cost but less-efficient alternative to separate mixers for new construction. Methods listed above are less efficient than separate mixers because, unlike separate mixing, the mix intensity depends on the flow rate.

6.2.1.3 *Flocculation*

During the flocculation step of the coagulation process, destabilized particles grow and agglomerate to form large, settleable flocs. Through gentle prolonged mixing, chemical bridging and/or physical enmeshment of particles occurs. Flocculation is slower and more dependent on time and agitation than is the rapid-mix step. Typical detention times for flocculation range between 20 and 30 minutes. Aerated and mechanical grit chambers, flow distribution structures, and influent wells are areas that promote flocculation upstream of primary sedimentation. Advantages and disadvantages of different configurations resemble those for rapid-mix facilities.

Like rapid mix, the velocity gradient, G, achieved with each configuration should be checked. Velocity gradients should be maintained from 50 to 80 5’. Polymers are sometimes added during the flocculation step to promote floc formation. Polymers should enter as dilute solution to ensure thorough dispersion of polymers throughout the wastewater. Polymers may provide a good floc with only turbulence and detention in the sedimentation tank inlet distribution.

6.2.1.4 *Coagulant Addition*

Supplementing conventional primary sedimentation with chemical coagulation requires minimal additional construction. The optimal point for coagulant addition is as far upstream as possible from primary sedimentation tanks. The optimum feed point for coagulant addition often varies from plant to plant. If possible, several different feed points should be considered for additional flexibility. Dispersing the coagulant throughout the wastewater is essential to minimize coagulant dosage and concrete and metal corrosion associated with coagulant addition. Flow-metering devices should be installed on chemical feed lines for dosage control.

6.2.2 Plate and Tube Settlers

Plate and tube settlers are utilized to increase the effective settling area within the clarifier or settling basin. They can be used with or without chemical enhancement but typically are utilized in advanced primary applications. These types of settlers operate on the principle that by increasing the area where particles can settle within the settling unit through the use of inclined tubes or plates will result in reduced footprint units accomplishing equivalent overflow rates to conventional settling basins with a much greater water surface area.

6.2.2.1 Calculation of Settling Area

The settling area within a plate clarifier is equal to the horizontally projected area of the vertically inclined plates. Therefore a settling basin equipped with (n) plates of overall surface area (A) inclined at an angle (∅ from the horizontal will have an equivalent settling area which can be calculated utilizing the equation:

$$\text{Total Settling Area} = nA(\cos\Theta)$$

Overflow rates can then be calculated utilizing the total settling area rather than the water surface area of the unit. Similar principles can be utilized for the calculation of total surface area and surface overflow rates for tube settlers.

6.2.2.2 Configuration

Typical settling plates are 0.6 - ~ 0.2 m wide and 3 m long with 50 mm spacing between multiple plates. Plate settlers are designed to operate in the laminar flow regime. Plate spacing must be large enough to prevent scouring of settled solids by the upward flowing liquid, to transport solids in a downward direction to the sludge hoppers, and to avoid plugging between the plates. In some instances plate vibrators or mechanical scrapers can be utilized to prevent plugging. Flash mixers and flocculation chambers may be required ahead of the plate clarifier (as with all clarifiers) to mix inchemicals to promote floc growth and enhance the clarification process. Care must be taken to transport flocculated feed to the settling unit at less than 0.3 m/s to prevent floc breakup.

6.2.3 Ballasted Floc Clanfiers

The ballasted flocculation and settling process is a precipitative process which utilizes micro-sand combined with polymer for improved floc attachment and thus improved settling. The process involves: 1) coagulation; 2) injection; 3) maturation;and 4) sedimentation. During the coagulation process, metal-salt coagulants (typically alum or ferric sulfate) are added and thoroughly mixed into solution. The water then enters the injection chamber where polymer addition is followed by micro-sand injection and subsequent flash mixing. The maturation process acts like a typical flocculation chamber, utilizing an optimum mixing energy for optimized floc agglomeration onto the micro-sand.

In the settling process, water enters the lower region of the basin and travels through lamella plates. Solids collection with tube settlers in the bottom of the settling chamber is followed by cyclonic separation of micro-sand and sludge.

The micro-sand exiting the hydrocyclone is then re-injected into the treatment process. The micro-sand used typically has a diameter of 50 to 100 microns. The typical detention times for coagulation, injection, and maturation are 1 to 2 minutes, 1 to 2 minutes, and 4 to 6 minutes, respectively. The detention time of the settling basin depends on the rise rate, which is typically between 50 to 100 m/d.

6.3 DISSOLVED AIR FLOTATION

Dissolved air flotation (DAF) refers to the process of solids-liquids separation caused by the introduction of fine gas (usually air) bubbles to the liquid phase. The bubbles attach to the solids, and the resultant buoyancy of the combined solids-gas matrix causes the matrix to rise to the surface of the liquid where it is collected by a skimming mechanism.

Flotation can be employed in both liquid clarification and solids concentration applications. Flotator liquid effluent (known as subnatant) quality is the primary performance factor in clarification applications. These applications include flotation of refinery, meat-packing, meat-rendering, and other “oily” wastewaters. Float-solids concentrations are the main performance criteria in solids concentration flotation applications. Concentration applications include the flotation of waste solids of biological, mining, and metallurgical processes.

6.3.1 Process Design Considerations and Criteria

The feed solids to a DAF clarifier are typically mixed with a pressurized recycle flow before tank entry. The recycle flow is typically DAF tank effluent, although providing water from another source as a backup is often advisable if poor DAF performance causes an effluent high in SS. The recycle flow is pumped to an air saturation tank where compressed air enters and dissolves into the recycle. As the pressurized recycle containing dissolved air is admitted back into the DAF tank (its surface is at atmospheric pressure), the pressure release from the recycle forms the air bubbles for flotation. A typical bubble-size distribution contains bubbles diameters ranging from 10 to 100 1am. Solids and air particles float and form a blanket on the DAF tank surface while the clarified effluent flows under the tank baffle and over the effluent weir. In general, the blanket on top of the DAF tank will be 150 to 300 mm thick.

Chemical conditioning with polymers is frequently used to enhance DAF performace. Polymer use significantly increases applicable solids-loading rates and solids capture but less effectively increases float-solids concentrations. If a polymer is used, it generally is introduced at the point where the recycle flow and the solids feed are mixed. Introducing the polymer solution into the recycle just as the bubbles are being formed are mixed with the solids produces the best results. Good mixing to ensure chemical dispersion while minimizing shearing forces will provide the best solids-air bubble aggregates.

Numerous factors affect DAF process performance, including:

- Type and characteristics of feed solids;
- Hydraulic loading rate;
- Solids-loading rate;
- Air-to-solids ratio;
- Chemical conditioning;
- Operating policy;

- Float-solids concentration; and
- Effluent clarity.

6.3.1.1 *Types of Solids*

A variety of solids can be effectively removed by flotation. Among these are conventional activated sludge, solids from extended aeration and aerobic digestion, pure-oxygen activated sludge, and dual biological (trickling filter plus activated-sludge) processes.

Effects of the DAF process factors listed in the previous section make it difficult to document the specific performance characteristics of each of these types of solids. In other words, the specific conditions at each plant (for example, types of process, SRT, and SVI in the aeration basin) dictate DAF performance to a greater extent than can be compensated for by flotation equipment adjustments such as air-to-solids ratio.

6.3.1.2 *Hydraulic Loading Rate*

Hydraulic loading rate refers to the sum of the feed and recycle flow rates divided by the net available flotation area. Dissolved air flotation clarifiers typically are designed for hydraulic loading rates of 60 to 120 m/d, assuming no use of conditioning chemicals. The additional turbulence in flotators when the hourly hydraulic loading rate exceeds 5 in/h may hinder the establishment of a stable float blanket and reduce the attainable float-solids turbulence forces the flow regime away from plug flow and more toward mixed flow. The addition of a polymer flotation aid generally is required to maintain satisfactory performance at hourly hydraulic loading rates greater than 5 in/h.

6.3.1.3 *Solids-Loading Rate*

The solids-loading rate of a DAF clarifier is generally denoted in terms of weight of solids per effective flotation area. With the addition of polymer, the solids-loading rate to a DAF thickener generally can be increased 50 to 100%, with up to a 0.5 to 1% increase in the thickened-solids concentration.

Operational difficulties may arise when the solids-loading rate exceeds approximately 10 kg/in2 .h. The difficulties generally are caused by coincidental operation of excessive hydraulic loading rates and by float-removal difficulties. Even in those instances when the hydraulic-loading rate can be maintained at less than 120 m/d, operation at solids-loading rates more than 10 kg/in2 .h can cause float-removal difficulties. The increased amount of float created at high solids-loading rates necessitates continuous skimming, often at high skimming speeds.

Increased skimming speed, however, can cause float blanket disturbance and increase the amount of solids in the subnatant to unacceptable levels. In these circumstances, the addition of polymer flotation aid to increase the rise rate of the solids and the rate of float-blanket consolidation can alleviate some of the operating difficulties. Although stressed conditions, such as mechanical breakdown, excessive solids wastage, or adverse solids characteristics, may make it necessary to periodically operate in this manner, the flotation system should not be designed on this basis.

6.3.1.4 *Feed-Solids Concentration*

Changes in feed-solids concentration indirectly affect flotation in connection with the resultant changes in operating conditions. If the fed flow rate, recycle flow, pressure, and skimmer operations remain constant, an increase in feed-solids concentration results in a decrease in the air-to-solids ratio. Changes in feed-solids concentration aiso result in changes to the float-blanket inventory and depth. Adjustments to the float skimmer speed many be required when operating strategy includes maintenance of a specific float-blanket depth or range of depths.

6.3.1.5 *Air-to-Solids Ratio*

The air-to-solids ratio is perhaps the single most important factor affecting DAF performance. It refers to the weight ratio of air available for flotation to the solids to be floated in the feed stream. Reported ratios range from 0.01:1 to 0.4:1; adequate flotation is achieved in most municipal wastewater clarification applications at ratios of 0.02:1 to 0.06:1. Pressurization system sizing depends on many variables, including design solids loading, pressurization system efficiency, system pressure, liquid temperature, and concentration of dissolved solids. Pressurization system efficiencies differ among manufacturers and system configurations and can range from as low as 50% to more than 90%. Detailed information is available regarding the design, specification, and testing of pressurization systems.

Because the float from a DAF clarifier contains a considerable amount of entrained air, this pumping application requires positive-displacement or centrifugal pumps that do not air bind, and special consideration of suction conditions. Initial density of the skimmed solids is approximately 700 kg/in3. After the solids are held for a few hours, the air escapes and the solids return to normal densities. Float-solids content increases with increasing air-to-solids ratios up to a point where further increases in air-to-solids ratios result in only a nominal or no increase in float solids. The typical air-to-solids ratio at which float solids are maximized varies from 2 to 4%.

6.3.1.6 *Float-Blanket Depth*

The float produced during the flotation process must be removed from the flotation tank. The float-removal system usually consists of a variable-speed float skimmer and a beach arrangement. The volume of float that must be removed during each skimmer pass depends on the solids-loading rate, the chemical dosage rate, and the consistency of the float material.

Float-removal system skimmers are designed and operated to maximize float drainage time by incrementally removing only the top (driest) portion of the float and preventing the float blanket from expanding to the point where float exits the system in the subnatant. The optimal float depth varies from installation to installation. A float depth of 0.3 to 0.6 in is typically sufficient to maximize float-solids content.

6.3.1.7 *Polymer Addition*

Chemical conditioning can enhance the performance of a DAF unit. Conditioning agents can be used to improve clarification and/or increase the float-solids concentration attainable with the unit. The amount of conditioning agent required, the point of addition (in the feed stream or recycle stream), and the method for intermixing should be specifically determined for each installation. Bench-scale flotation tests or pilot-unit tests provide the most effective method of determining the optimal chemical conditioning scheme for a particular installation. Typical polymer doses range from 2 to 5 g dry polymer/kg dry feed solids.

The addition of polymer usually affects solids capture to a greater extent than float-solids content. The float-solids content generally is increased up to 0.5% by the addition of dry polymer at a dosage of 2 to 5 g/kg dry solids.

If the lower ranges of hydraulic and solids loadings are used, the addition of polymer flotation aid typically is unnecessary for well-designed and —operated DAF clarifiers. Maintenance of proper design and operating conditions as described in the preceding sections results in stable operation and satisfactory performance in terms of solids capture and float-solids concentration.

Solids recovery without polymer addition generally will be much greater than 90% when the DAF unit is sized as previously discussed. High loadings or adverse solids conditions can reduce solids recovery to 75 to 90%. Polymer-aided recovery can exceed 95%.

Under normal operations, the solids recycled from the DAF unit will not be damaging to the treatment system but will have the effect of increasing the WAS to be processed. In cases where the solids or hydraulic loading already are excessive, the recycled solids pose an additional burden on the system. Polymers should be employed under these conditions to maximize solids capture from the DAF unit.

6.4 PROTECTIVE AND SERVICE FACILITIES

6.4.1 Operator Protection

All clarification tankage shall be equipped to enhance safety for operators. Such features shall appropriately include machinery covers, life lines, stairways, walkways, handrails and slip-resistant surfaces.

6.4.2 Mechanical Maintenance Access

The design shall provide for convenient and safe access to routine maintenance items such as gear boxes, scum removal mechanisms, baffles, weirs, inlet stilling baffle area and effluent channels.

6.4.3 Electrical Fixtures and Controls

Electrical fixtures and controls in enclosed settling basins shall comply with the Canadian Electrical Code and the applicable Provincial Power Standards. The fixtures and controls shall be located so as to provide convenient and safe access for operation and maintenance. Adequate area lighting shall be provided.

7.1 ACTIVATED SLUDGE

7.1.1 General

7.1.1.1 Applicability

The activated sludge process and its various modifications, may be used where sewage 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.

7.1.1.2 Specific Process Selection

The activated sludge process and its several modifications may be employed to accomplish varied degrees of removal of suspended solids and reduction of five day BOD. Choice of the process most applicable will be influenced by the proposed plant size, type of waste to be treated, degree and consistency of treatment required, anticipated degree of operation and maintenance and operating and capital costs. All designs shall provide for flexibility in operation.

7.1.1.3 Aeration Equipment Selection

Evaluation of aeration equipment alternatives should include the following considerations:

The size of the aeration tank for any particular adaptation of the process shall be determined by full scale experience, pilot plant studies, or rational calculations based mainly on food to microorganism ratio and mixed liquor suspended solids levels. Other factors, such as size of treatment plant, diurnal load variations, and degree of treatment required, shall also be considered. In addition, temperature, pH, and reactor dissolved oxygen shall be considered when designing for nitrification.

- Costs - capital, maintenance and operating;
- Oxygen transfer efficiency;
- Mixing capabilities;
- Diffuser clogging problems;
- Air pre-treatment requirements;
- Total power requirements;
- Aerator tip speed of mechanical aerators used with activated sludge systems;
- Icing problems;
- Misting problems; and
- Cooling effects on aeration tank contents.

7.1.1.4 Energy Requirements

This process requires major energy usage to meet aeration demands. Energy costs in relation to critical water quality conditions must be carefully evaluated. Capability of energy usage phase-down while still maintaining process viability, both under normal and emergency energy availability conditions, must be included in the activated sludge design.

7.1.1.5 Winter Protection

Protection against freezing shall be provided to ensure continuity of operation and performance.

7.1.1.6 Pretreatment

Where primary settling tanks are not used, effective removal or exclusion of grit, debris, excessive oil or grease, and comminution or screening of solids shall be accomplished prior to the activated sludge process.

Where primary settling is used, provision shall be made for discharging raw sewage directly to the aeration tanks to facilitate plant start-up and operation during the initial stages of the plant's design life.

7.1.1.7 Waste Activated Sludge Concentration

In the absence of primary settling tanks, other effective means of waste sludge concentration shall be provided.

7.1.2 Process Definitions

The following are brief descriptions of a number of modifications of the activated sludge process.

7.1.2.1 Conventional Activated Sludge

The plug flow activated sludge process is a biological mechanism capable of removing 85 to 95% BOD from typical municipal wastewater. The flow pattern is plug-flow-type. The process is characterized by 20 to 45% sludge return. This is the original activated sludge process and was later modified to suit various applications, situations and treatment requirements. One characteristic of the plug flow configuration is a very high organic loading on the mixed liquor suspended solids (MLSS) in the initial part of the task. Plug flow configurations are often preferred when high effluent DO's are sought.

7.1.2.2 Complete Mix Activated Sludge

In a complete mix activated sludge process, the characteristics of the mixed liquor are similar throughout the aeration tank. That is, the influent waste is rapidly distributed throughout the tank and the operating characteristics measured in terms of solids, oxygen uptake rate (OUR), MLSS, and soluble BOD_5 concentration are identical throughout the tank. Because the entire tank contents are the same quality as the tank effluent, there is a very low level of food available at any time to a large mass of microorganisms. This is the major reason why the complete mix modification can handle surges in the organic loading without producing a change in effluent quality.

7.1.2.3 Step Aeration

Step feed is a modification of the plug flow configuration in which the secondary influent is fed at two or more points along the length of the aeration tank. With this arrangement, oxygen uptake requirements are relatively even, resulting in better utilization of the oxygen supplied. Step feed configurations generally use diffused aeration equipment. Secondary influent flow is usually added in the first 50 to 75% of the aeration tank's length.

7.1.2.4 *Contact Stabilization*

Contact stabilization activated sludge is both a process and a specific tankage configuration. Contact stabilization encompasses a short-term contact tank, secondary clarifier, and a sludge stabilization tank with about six times the detention time used in the contact tank.

This unit operation was developed to take advantage of the fact that BOD removal occurs in two stages. The first is the absorptive phase and the second is the stabilization of the absorbed organics.

Contact stabilization is best for smaller flows in which the mean cell residence time (MCRT) desired is quite long. Therefore, aerating return sludge can reduce tank requirements by as much as 30 to 40% versus that required in an extended aeration system.

7.1.2.5 *Extended Aeration*

The extended aeration process used the same flow scheme as the complete mix or plug flow processes but retains the wastewater in the aeration tank for long periods of time. This process operates at a high MCRT (low F/M) resulting in a condition where there is not enough food in the system to support all the microorganisms present. The microorganisms therefore compete very actively for the remaining food and even use their own cell structure for food. This highly competitive situation results in a highly treated effluent with low sludge production. However, extended aeration plant effluents generally have significant concentrations of "pin floc" resulting in BOD_5 and SS removals of about 85%. Many extended aeration systems do not have primary clarifiers. Also, many are package plants used by small communities.

The main disadvantages of this system are the large oxygen requirements per unit of waste entering the plant and the large tank volume needed to hold the wastes for the extended period.

7.1.2.6 *Oxidation Ditch*

The oxidation ditch is a variation of the extended aeration process. The wastewater is pumped around a circular or oval pathway by a mechanical aerator/pumping device at one or more points along the flow pathway. In the aeration tank, the mixed liquor velocity is maintained between 0.2 to 0.37 m/s in the channel to prevent solids from settling.

Oxidation ditches use mechanical brush disk aerators, surface aerators, and jet aerator devices to aerate and pump the liquid flow.

7.1.2.7 *High Rate Aeration*

This is a type of short-term aeration process in which relatively high concentrations of MLSS are maintained, by utilizing high sludge recirculation rates (100 to 500%), and low hydraulic retention times. Depending on the excess sludge wasting procedure, 60 to 90% BOD removal is achieved for normal domestic wastes. This process is usually (but not necessarily) accomplished in "combined-tank" units.

7.1.2.8 High Purity Oxygen

The most common high purity oxygen activated sludge process uses a covered and staged aeration tank configuration. The wastewater, return sludge, and oxygen feed gas enter the first stage of this system and flow concurrently through the tank. The tanks in this system are covered to retain the oxygen gas and permit a high degree of oxygen use. A prime advantage of the staged reactor configuration of the oxygenation system is the system's ability to match approximately the biological uptake rate with the available oxygen gas purity.

7.1.3 Return Sludge Equipment

7.1.3.1 Return Sludge Rate

The minimum permissible return sludge rate of withdrawal from the final settling tank is a function of the concentration of suspended solids in the mixed liquor entering it, the sludge volume index of these solids and the length of time these solids are retained in the settling tank. Since undue retention of solids in the final settling tank may be deleterious to both the aeration and sedimentation phases of the activated sludge process, the rate of sludge return expressed as a percentage of the average design flow of sewage should generally be variable between the limits set forth as follows:

PERCENTAGE OF AVERAGE DESIGN FLOW			
Type of Process	**Minimum**	**Normal**	**Maximum**
Plug Flow	25	30	100
Complete Mix	25	30	100
Carbonaceous Stage of Separate Stage	25		75
Nitrification	25	50	75
Step Aeration	50	100	150
Contact Stabilization	50	100	150
Extended Aeration	50	100	150
Oxidation Ditch	50	50	200
High Rate			
Nitrification Stage of Separate Stage Nitrification	50		200

The rate of sludge return shall be varied by means of variable speed motors, drives or timers (small plants) to pump sludge at the above rates.

7.1.3.2 Return Sludge Pumps

If motor driven return sludge pumps are used, the maximum return sludge capacity shall be obtained with the largest pump out of service. A positive head should be provided on pump suctions. Pumps should have at least 100 mm suction and discharge openings.

If air lifts are used for returning sludge from each settling tank hopper, no standby unit will be required, provided the design of the air lifts are such as to facilitate their rapid and easy cleaning and provided other suitable standby measures are provided. Air lifts should be at least 100 mm in diameter.

7.1.3.3 *Return Sludge Piping*

Suction and discharge piping should be at least 100 mm in diameter and should be designed to maintain a velocity of not less than 0.6 m/s and not more than 2 m/s, when return sludge facilities are operating at normal return sludge rates. Suitable devices for observing, sampling and controlling return activated sludge flow from each settling tank shall be provided.

7.1.3.4 *Waste Sludge Facilities*

Waste sludge control facilities should be designed for the maximum sludge production of the process. Means for observing, measuring, sampling and controlling waste activated sludge flow shall be provided. Waste sludge may be discharged to the primary settling tank, concentration or thickening tank, sludge digestion tank, mechanical dewatering facilities or any practical combination of these units.

7.1.3.5 *Froth Control Units*

It is essential to include some means of controlling froth formation in all aeration tanks. A series of spray nozzles may be fixed on top of the aeration tank. Screened effluent or tap water may be sprayed through these nozzles (either continuously or on a time clock on-off cycle) to physically break up the foam. Provision may be made to use antifoaming chemical agents into the inlet of the aeration tank or preferably into the spray water.

7.2 SEQUENCING BATCH REACTOR (SBR)

The Sequencing Batch Reactor (SBR) is a fill-and-draw activated sludge treatment system. All SBR systems utilize five steps that occur sequentially within the same tank as follows: (1) fill, (2) react (aeration), (3) settle (clarification), (4) decant, and (5) idle. Process modifications can be made by varying the times associated with each step in order to achieve specific treatment objectives. When designing or evaluating SBR systems care must be taken with the processes that are unique to the SBR. These include:

a. Fill Method
b. Hydraulic Control Systems
c. Aeration Control Systems
d. Method of Decant
e. Sizing of Disinfection Equipment for Decant Flows
f. Sludge Wasting Methods

One of the main strengths of the SBR process is the process flexibility that can be achieved. Therefore, the above processes can be performed using a variety of methods. Designers of SBR systems must be prepared to supply sufficient detailed information at the request of regulatory authorities.

7.2.1 Process Configurations

One classification of SBR systems distinguishes those that operate with continuous feed and intermittent discharge (CFID) from those that operate with intermittent feed and intermittent discharge (IFID).

7.2.2 Continuous Influent Systems

Continuous feed—intermittent discharge reactors receive influent wastewater during all phases of the treatment cycle. When there is more than one reactor, as is typically the case for municipal systems, the influent flow is split equally to the various reactors on a continuous basis. For two-reactor systems, it is normal to have the reactor cycle operations displaced so that one SBR is aerating while the second SBR is in the settling and decant phases. This makes it possible to aerate both reactors with one blower continuously in operation and also spreads the decant periods so that there is no overlap. The dry weather flow cycle time for most CFIC systems is generally 3 to 4 hours. Each cycle typically devotes 50% of the cycle time to aeration, 25% to settling, and 25% to decant. Stormwater flows are accommodated by reducing cycle time. Under extreme flow condition, the reactor may operate as a primary clarifier (no aeration phase) with the decanters set at top water level (TWL).

With a CFID system, TWL occurs at the start of the decant phase. Because CFID systems generally operate on the basis of preset time cycles, TWL varies for each cycle as a function of the influent flow for that particular cycle. The actual effluent flow rate during the discharge event depends on the number of reactors and the percentage of each cycle devoted to decant.

A key design consideration with CFID systems is to minimize short-circuiting between influent and effluent. Influent and effluent discharge are typically located at opposite ends of rectangular reactors, with length-to-width ratios of 2:1 to 4:1 being common. Installation of a prereaction chamber separated by a baffle wall from the main reaction chamber is also a standard feature of some systems.

7.2.3 Intermittent Influent Systems

IFID types of systems are sometimes referred to as the conventional, or "true," SBR systems. The one common characteristic of all IFID systems is that the influent flow to the reactor is discontinued for some portion of each cycle.

In IFID systems each reactor operates with five discrete phases during a cycle. During the period of reactor fill, any combination of aeration, mixing, and quiescent filling may be practices. Mixing independent of aeration can be accomplished by using jet aeration pumps or separate mixers. Some systems distribute the influent over a portion of the reactor bottom so that it will contact settled solids during unaerated and unmixed fill. The end of the fill cycle is controlled either by time (that is, fill for a preset length of time) or by volume (that is, fill until the water level rises a fixed amount). Flow information from the WWTP influent flow measurement or from the rise rate in the reactor determined by a series of floats may be used to control the time allocated to aeration, mixing, or filling in accordance with previously programmed instructions.

At the end of the fill cycle, all influent flow to the first reactor is stopped, and flow is diverted to the second reactor. Continuous aeration occurs during the react phase for a predetermined time period (typically 1 to 3 hours). Again, the time devoted to reaction in any given cycle may automatically be changed as a function of influent flow rate. At the completion of the reaction phase, aeration and any supplemental mixing is stopped, and the mixed liquor is allowed to settle under quiescent conditions (typically 30 to 60 minutes). Next, clarified effluent is decanted until the bottom water level (BWL) is reached. The idle period represents that time period between the end of decant and the time when influent flow is again redirected to a given reactor. During high-flow periods, the time in idle will typically be minimal.

The actual flow rate during discharge has the potential to be several times higher than the influent flow rate. Discharge flow rates are critical design parameters for the downstream hydraulic capacity of sewers (in the case of industrial treatment facilities) or processes such as disinfection or filtration.

Another variation of the IFID approach dispenses with a dedicated reaction phase and initiates the settling cycle at the end of aerated fill. Yet another IFID approach allows influent to enter the reactor at all times except for the decant phase so that normal system operation consists of the following phases: (1) fill-aeration, (2) fill-settling, (3) no fill-decant, and (4) fill-idle; these systems also include an initial selector compartment that operates either at constant or variable volume and serves as a flow splitter in multiple-basin systems. Biomass is directed from the main aeration zone to the selector.

Sequencing batch reactor systems can also be designed for nitrification-denitrification and enhanced biological phosphorus removal. In these cases, the cycle times devoted to such processes as anaerobic fill, anoxic fill, mixed/unmixed fill, aerobic fill, and dedicated reaction depend on the treatment objectives. Mineral addition may also be practiced to achieve effluent objectives more stringent than typical secondary effluent requirements. Systems can also be configured to switch from IFID operation to CFID operation when necessary to accommodate stormwater flows or to allow a basin to be removed from service while still treating the entire WWTP flow in a remaining basin. The one common factor behind all SBRs is that aeration, settling, and decant occur within the same reactor.

7.2.4 Sequencing Batch Reactor Equipment

7.2.4.1 Process Control

The programmable logic controller (PLC) is the optimum tool for SBR control and all present-day vendors use this approach. Sequencing batch reactor manufacturers supply both the PLC and required software. Typically, programs are developed and modified by the SBR vendor using a desk-top computer and software supplied by the PLC vendor. Vendor-developed programs are proprietary and may not be modified by the design engineer or the WWTP operator. Depending on the proprietary software design and type of system, the operator may independently select such variables as solids waste rates; storm cycle times; and aeration, mixing, and idle times. In addition, the design engineer may develop additional software to interface to PLC to a desk-top computer for graphic presentation of process operation to the operator and generation of archive date and compliance reports.

Programmable logic controller hardware is of modular construction. Troubleshooting procedures are well defined, and replacement of a faulty module is not difficult. An internal battery protects the software in the event of power failure. The software is backed up by a memory chip (EPROM) and can be easily reloaded if the battery fails. The PLC expertise required of the owner is limited to maintenance and repair functions that are well within the capability of a competent electrician.

7.2.4.2 *Reactors*

Reactor shapes include rectangular, oval, circular, sloped sidewall, and other unique approaches. Design TWLs and BWLs often allow decanting from 20 to 30% of the reactor contents per cycle.

7.2.4.3 *Decanters*

Some decanters are mechanically actuated surface skimmers that typically rest above the TWL. The decanter is attached to the discharge pipe by smaller pipes that both support and drain the decanter. The discharge pipe is coupled at each end through seals that allow it to rotate. A screw-type jack attached to a worm tear, sprocket, and chain to an electric motor rotates the decanter from above the TWL to BWL. The speed of rotation is adjustable.

Other decanters are floated on the reactor surface. These decanters may approximate a large-diameter plug valve, whereby the top portion acts as the valve seat (and provides flotation). The bottom is the plug that is connected to a hydraulic operator that moves it away from the seat to allow discharge, or back to the seat to stop discharge. Other floating decanters consist of a length of pipe suspended on floats, with the pipe having a number of orifices bored in the bottom. The number of orifices (and length of pipe) is flow dependent. Each orifice is blocked by a flapper or plugs to prevent solids entry during aeration. There are also decanter configurations that float an effluent discharge pump.

Other decanters are typically fixed-position siphons located on the reactor wall. The bottom of the decanter (collection end of the siphon) is positioned at the BWL. Flow into the decanter is under a front lip (scum baffle), over an internal dam, and out through a valve. When the water level in the reactor falls below the front lip, air enters the decanter, breaking the siphon and stopping flow. The trapped air prevents mixed liquor from entering during the reaction and settling modes. At the end of settling, the trapped air is released through a solenoid valve and the siphon is started.

7.2.4.4 *Solids Wasting*

The wasting of both aerated mixed liquor suspended solids (MLSS) and settled MLSS is practiced. The wasting systems frequently consist of a submersible pump with a single point for withdrawal. Gravity flow waste systems are also used. Another approach uses influent distribution piping for multiple-point with withdrawal of the settled solids.

7.2.4.5 *Aeration/Mixing Systems*

A variety of aeration and mixing systems are in use with SBRs. These include jet aeration, fine- and course-bubble aeration, and turbine mechanical aeration. Some systems use a floating mixer to provide mixing independent of aeration. Other diffused aeration facilities do not have any mixing capability independent of aeration. Independent mixing is readily obtained with a jet aeration system.

7.3 ACTIVATED SLUDGE DESIGN PARAMETERS

Table 7.1 shows typical design parameters and efficiencies for various activated sludge process modifications.

TABLE 7.1 - TYPICAL DESIGN PARAMETERS FOR ACTIVATED SLUDGE PROCESS MODIFICATIONS

MODIFICATION	PROCESS LOADING RATE	MCRT (days)	F/M (kg BOD / kg MLVSS)	ORGANIC LOADING RATE (kg BOD/cu. m/d)	HYDRAULIC RETENTION TIME (hrs)	MLSS (mg/l)	BOD REMOVAL EFFICIENCY (%)
PLUG FLOW	CONVENTIONAL - LOW RATE	4 - 15	0.20 - 0.50	0.30 - 0.60	4 - 8	1500 - 3000	85 - 95
COMPLETE MIX	CONVENTIONAL - LOW RATE	4 - 15	0.20 - 0.60	0.80 - 2.0	3 - 5	3000 - 6000	85 - 95
STEP AERATION	CONVENTIONAL - LOW RATE	4 - 15	0.20 - 0.50	0.30 - 0.60	3 - 5	2000 - 3500	85 - 95
CONTACT STABILIATION	CONVENTIONAL - LOW RATE	3 - 10	0.20 - 0.60	1.0 - 1.2	CONTACT TANK - 0.5 - 1.0 STABILIATION TANK - 3.0 - 6.0	CONTACT TANK - 1000 - 3000 STABLIZATION TANK - 4000 - 10000	80 - 90
EXTENDED AERATION	LOW RATE	20 - 30	0.05 - 0.15	0.16 - 0.40	18 - 36	3000 - 6000	75 - 90
OXIDATION DITCH	LOW RATE	20 - 30	0.05 - 0.15	0.16 - 0.40	18 - 36	3000 - 5000	75 - 90
HIGH RATE AERATION	HIGH RATE	2 - 6	0.40 - 1.0	1.0 - 6.0	2 - 4	3000 - 8000	75 - 90
HIGH PURITY OXYGEN	HIGH - CONVENTIONAL RATE	8 - 20	0.20 - 1.0	1.6 - 4.0	1 - 3	3000 - 6000	85 – 95
SEQUENCING BATCH REACTOR (SBR)	CONVENTIONAL – LOW RATE	N/A	0.05 – 3.0	0.08-0.25	12 - 48	1500 - 5000	85 - 95

The size of the aeration tank for any particular adaptation of the process shall be determined by full scale experience, pilot plant studies, or rational calculations based mainly on food to microorganism ratio and mixed liquor suspended solids levels. Other factors, such as size of treatment plant, diurnal load variations, and degree of treatment required, shall also be considered. In addition, temperature, pH, and reactor dissolved oxygen shall be considered when designing for nitrification.

7.4 AERATION

7.4.1 Arrangement of Aeration Tanks

7.4.1.1 General Tank Configuration

a. Dimensions

The dimensions of each independent mixed liquor aeration tank or return sludge re-aeration tank shall be such as to maintain effective mixing and utilization of air.

Aeration basin depth is an important consideration in the design of aeration systems because of the effect that depth has on the aeration efficiency and air pressure requirements of diffused aeration devices and mixing capabilities of mechanical aerators. A minimum aeration basin depth of 3.0 to 4.6 m is recommended for typical sewage treatment plants. Oxidation ditches should have minimum depth of 1.6 m.

b. Short-circuiting

For very small tanks or tanks with special configuration, the shape of the tank, the location of the influent and sludge return and the installation of aeration equipment should provide for positive control of short-circuiting through the tank.

c. Number of Units

Total aeration tank volume shall be divided among two or more units, capable of independent operation, when the total aeration tank volume required exceeds 140 m^3.

7.4.1.1.1 Inlets and Outlets

a. Controls

Inlets and outlets for each aeration tank unit shall be suitably equipped with valves, gates, stop plates, weirs or other devices to permit controlling the flow to any unit and to maintain a reasonably constant liquid level. The hydraulic properties of the system shall permit the design peak instantaneous hydraulic load to be carried with any single aeration tank unit out of service. The effluent weir for an oxidation ditch must be easily adjustable by mechanical means.

b. Conduits

Channels and pipes carrying liquids with solids in suspension shall be designed to maintain self-cleansing velocities or shall be agitated to keep such solids in suspension at all rates of flow within the design limits. Adequate provisions should be made to drain segments of channels which are not being used due to alternate flow patterns.

7.4.1.1.2 Measuring Devices

Devices should be installed for indicating flow rates of raw sewage or primary effluent, return sludge and air to each tank unit. For plants designed for sewage flows of 5000 m^3/d or more, these devices should totalize and record, as well as indicate flows. Where the design provides for all return sludge to be mixed with the raw sewage (or primary effluent) at one location, then the mixed liquor flow rate to each aeration unit should be measured.

7.4.1.1.3 Freeboard

All aeration tanks should have a freeboard of not less than 450 mm. Additional freeboard or windbreak may be necessary to protect against freezing or windblown spray. If a mechanical surface aerator is used, the freeboard should not be less than 900 mm.

7.4.1.2 Aeration Equipment

7.4.1.2.1 General

Oxygen requirements generally depend on maximum BOD loading, degree of treatment and level of suspended solids concentration to be maintained in the aeration tank mixed liquor. Aeration equipment shall be capable of maintaining a minimum of 2.0 mg/L of dissolved oxygen in the mixed liquor at all times and providing thorough mixing of the mixed liquor. In the absence of experimentally determined values, the design oxygen requirements for all activated sludge processes shall be 1.1 kg O_2/kg peak BOD_5 applied to the aeration tanks with the exception of the extended aeration process, for which the value shall be 1.5. In the case of nitrification, the oxygen requirement for oxidizing ammonia must be added to the above requirement for carbonaceous BOD removal. The nitrogen oxygen demand (NOD) shall be taken as 4.6 times the diurnal peak TKN content of the influent. In addition, the oxygen demands due to recycle flows - heat treatment supernatant, vacuum filtrate, elutriates, etc. - must be considered due to the high concentrations of BOD and TKN associated with such flows.

Careful consideration should be given to maximizing oxygen utilization per unit power input. Unless flow equalization is provided, the aeration system should be designed to match diurnal organic load variation while economizing on power input.

7.4.1.2.2 Variable Oxygenation Capacity

Consideration should be given to reducing power requirements of aeration systems by varying oxygenation capacity to match oxygen demands within the system. Such a system would utilize automatic D.O. probes in each aeration basin to measure dissolved oxygen levels.

7.4.1.2.3 Mixing Requirements

The aeration system which is selected must not only satisfy the oxygen requirements of the mixed liquor, but must also provide sufficient mixing to ensure that the mixed liquor remains in suspension. The power levels necessary to achieve uniform dissolved oxygen and mixed liquor suspended solids concentrations are shown in Table 7.2.

TABLE 7.2 - AERATION MIXING REQUIREMENTS

AERATION SYSTEM	FOR UNIFORM D.O. LEVELS	FOR UNIFORM MLSS LEVELS
MECHANICAL	1.6 TO 2.5 W/cu. m	16 TO 25 W/cu. m
DIFFUSED (COARSE BUBBLE, SPIRAL ROLL)	---	0.33 L/cu. m/s
DIFFUSED (FINE BUBBLE DOMES, FULL FLOOR COVERAGE)	---	0.6 L/sq. m/s

NOTES:

1. Mixing requirements vary with basin geometry, MLSS concentrations, placement of aeration devices, pumping efficiency of aerators, etc. Wherever possible, refer to full-scale testing results for the particular aerator being considered.
2. L/cu. m/s refers to volume of air per second per volume of aeration tank.
3. L/sq. m/s refers to volume of air per second per horizontal cross-sectional area of aeration tank.

7.4.1.2.4 Back-up Requirements

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 or two blowers will have a greater effect on capacity than the breakdown of one or 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; and
- means other than duplicate equipment to provide the necessary capacity in the event of a breakdown (for instance, using over-sized mechanical aerators with adjustable weirs to control power draw and oxygenation capacity, or using two speed mechanical aerators, etc.).

Generally considerations such as the above will mean that diffused aeration systems will require a standby blower but mechanical aeration systems may not require standby units, depending upon the number of duty units, availability of replacement parts, etc.

7.4.1.2.5 Oxygen Transfer and Oxygen Transfer Efficiency

Aeration equipment must be designed to carry out its functions under conditions much different than those under which it may be tested by the equipment supplier. The bulk of oxygen transfer tests are conducted under conditions commonly referred to as standard or are corrected to standard conditions.

The designer, therefore, must test the unit under standard conditions and project its efficiency to the mixed liquor, or conduct the test in the mixed liquor. In either case, there are intricacies (related to aerator testing) involved in making this conversion from one condition to another. It is good practice to work with the suppliers when selecting aerators to discuss and agree on, at the time of design, the planned test procedure and interpretation of results.

It is most common to express the oxygenation rate of a particular activated sludge aeration device either as standard oxygen rate, SOR or actual oxygen rate, AOR, both in Kg of oxygen transferred per hour. Either value is considered to be determinable, given the other and its transfer environment. Methods used to calculate oxygen transfer for conditions other than standard, or to correct to standard conditions the results obtained by mixed liquor testing are as follows:

Mechanical surface aerators: Diffused air and submerged turbine aerators:

$AOR = \alpha(SOR)\,(ß(C_{sw}-C_0)/C_s)\,\Theta^{(T-20)}$ (1) $AOR = \alpha(SOR)\,(ß(C_{SC*}-C_0)/C_s)\,\Theta^{(T-20)}$ (2)

α = relative rate of oxygen transfer as compared to clean water, dimensionless [equal to 1.0 under standard conditions. Mixed liquor range is 0.6 (near basin influent) to 0.94, with the higher value representative of well-treated wastes; values generally range from 0.8 to 0.94; initial stages of plug flow systems may have very low α; industrial wastes may reduce α];

ß = relative oxygen saturation value as compared to clean water, dimensionless [equal to 1.0 under standard conditions. Mixed liquor range to 0.9 to 0.97, with upper level seen in well-treated wastes];

Θ = temperature correction constant, 1.024;

C_s = oxygen saturation value of clean water at standard conditions, C_s = 9.17 mg/L,

C_{sw} = saturation value of clean water at the surface, at site conditions of temperature, T, and actual barometric pressure, P_a,

$C_{sc}*$ = corrected C_s value for water depth, D, and oxygen content of gaseous phase, mg/L,

C_0 = initial (or steady state) DO level mg/L [equal to 0.0 mg/L under standard conditions. Mixed liquor values range from 1.5 to 2.0 mg/L at average oxygen uptake conditions], and

T = Temperature of bulk liquid, °C [equal to 20°C under standard conditions. Mixed liquor values range from 5° to 30°C; highest operating temperature is most conservative in terms of design (C_s and $\Theta^{(20-T)}$ to self-correct)].

In the absence of experimentally determined alpha and beta factors, wastewater transfer efficiency shall be assumed to be 50% of clean water efficiency for plants treating primarily (90% or greater) domestic sewage. Treatment plants where the waste contains higher percentages of industrial wastes shall use a correspondingly lower percentage of clean water efficiency and shall have calculations submitted to justify such a percentage.

Oxygen transfer efficiencies are generally represented in terms of kg of O_2 transferred per MJ. Manufacturers of aeration equipment will generally designate the specific equipment O_2 transfer rate as Ns (the standard transfer efficiency or rated capacity). The rated capacity can be expressed as:

$$Ns = \frac{SOR}{\text{Total Power Consumed (by mixer/aerator, pump, blower)}} \qquad (3)$$

The designer must therefore use equation (3) to determine the AOR as described in equation (1) or (2).

7.4.1.2.6 *Characteristics of Aeration Equipment*

Table 7.3 outlines various characteristics of some typical aeration equipment.

7.4.1.2.7 *Diffused Air Systems*

Typical air requirements for all activated sludge processes except extended aeration (assuming equipment capable of transmitting to the mixed liquor the amount of oxygen required in Section 7.1.5.2.1) is 100 cu. meters per kg of BOD_5 peak aeration tank loading. For the extended aeration process the value is 125 m^3.

Air requirements for diffused air systems should be augmented as required by consideration of the following items:

a. To the air requirements calculated shall be added air required for channels, pumps, aerobic digesters or other air-use demand.

b. The specified capacity of blowers or air compressors, particularly centrifugal blowers, should take into account that the air intake temperature may reach 40°C or higher and the pressure may be less than normal. The specified capacity of the motor drive should also take into account that the intake air may be -30°C or less and may require oversizing of the motor or a means of reducing the rate of air delivery to prevent overheating or damage to the motor.

c. The blowers shall be provided in multiple units, so arranged and in such capacities as to meet the maximum air demand with the single largest unit out of service. The design shall also provide for varying the volume of air delivered in proportion to the load demand of the plant. Aeration equipment shall be easily adjustable in increments and shall maintain solids suspension within these limits.

TABLE 7.3 - CHARACTERISTICS OF SOME AERATION EQUIPMENT

EQUIPMENT TYPE	EQUIPMENT CHARACTERISTICS	PROCESS WHERE USED	ADVANTAGES	DISADVANTAGES	REPORTED TRANSFER EFFICIENCY* (kg/MJ) FOR STD. CONDITIONS, 0 D0, 20°c, 101 KPa, AND CLEAN WATER
DIFFUSED AIR: A.BUBBLER POROUS DIFFUSERS	PRODUCE FINE-TO-MEDIUM BUBBLES. MADE OF CERAMIC DOMES, PLATES, TUBES, OR PLASTIC-CLOTH TUBE OR BAG.	HIGH RATE, CONVENTIONAL EXTENDED, STEP, MODIFIED, CONTACT-STABILIZATION ACTIVATED SLUDGE PROCESS.	GOOD MIXING; MAINTAINS LIQUID TEMPERATURE. VARYING AIR FLOW PROVIDES GOOD OPERATIONAL FLEXIBILITY.	HIGH INITIAL AND MAINTENANCE COSTS; AIR FILTERS NEEDED; SPIRAL CONFIGURATION LIMITS TANK GEOMETRY.	0.31 - 0.42
NONPOROUS DIFFUSERS	MADE IN BUBBLE CAP, NOZZLE, VALVE, ORIFICE, OR SHEAR TYPES. THEY PRODUCE COARSE OR LARGE BUBBLES. SOME MADE OF PLASTIC WITH CHECK VALVE DESIGN.	SAME AS FOR POROUS DIFFUSERS.	NON-CLOGGING, MAINTAINS LIQUID TEMPERATURE; LOW MAINTENANCE COST.	HIGH INITIAL COST; LOW OXYGEN TRANSFER EFFICIENCY; HIGH POWER COST. FOULING MAY OCCUR.	0.2 - 0.31
B.TUBULAR	PRODUCES HIGH SHEAR AND ENTRAINMENT AS WATER-AIR MIXTURE IS FORCED THROUGH VERTICAL CYLINDER CONTAINING STATIC MIXING ELEMENTS. CYLINDER CONSTRUCTION IS METAL OR PLASTIC.	PRIMARILY AERATED LAGOON APPLICATIONS.	ECONOMICALLY ATTRACTIVE; LOW MAINTENANCE; HIGH TRANSFER EFFICIENCIES FOR DIFFUSED AIR SYSTEMS. WELL SUITED FOR AERATED LAGOON APPLICATIONS.	ABILITY TO ADEQUATELY MIX REACTOR BASIN CONTENTS IS QUESTIONABLE. APPLICATION FOR USE IN HIGH RATE BIOLOGICAL SYSTEMS UNCONFIRMED.	0.31 - 0.44
C. JET	COMPRESSED AIR AND PUMPED LIQUID ARE VIOLENTLY INTERMIXED IN NOZZLE AND AT DISCHARGE INTO VESSEL.	SAME AS FOR BUBBLER DIFFUSER.	SUITED FOR DEEP TANKS; MODERATE COST.	TANK GEOMETRY LIMITED. CLOGGING OF NOZZLE REQUIRES BLOWER AND PUMP. PRIMARY TREATMENT REQUIRED.	0.43 - 0.60
MECHANICAL SURFACE: A.RADIAL FLOW, LOW SPEED 20 - 60 RPM	LOW OUTPUT SPEED; LARGE DIAMETER TURBINE, FLOATING, FIXED-BRIDGE OR PLATFORM MOUNTED. USED WITH GEAR REDUCER.	SAME AS FOR BUBBLER DIFFUSER.	TANK DESIGN FLEXIBILITY; HIGH PUMPING CAPACITY.	SOME ICING IN COLD CLIMATES. INITIAL COST HIGHER THAN AXIAL FLOW AERATORS. GEAR REDUCER MAY CAUSE MAINTENANCE PROBLEMS.	0.34 - 0.76
B.AXIAL FLOW 300 - 1200 RPM	HIGH OUTPUT SPEED. SMALL DIAMETER PROPELLER. THEY ARE DIRECT, MOTOR DRIVEN UNITS MOUNTED ON FLOATING STRUCTURE.	AERATED LAGOONS AND REAERATION.	LOW INITIAL COST; EASY TO ADJUST TO VARYING WATER LEVEL FLEXIBLE OPERATION.	SOME ICING IN COLD CLIMATES; POOR MAINTENANCE ACCESSIBILITY, MIXING CAPACITY MAY BE INADEQUATE.	0.34 - 0.43
C. BRUSH ROTOR	LOW OUTPUT SPEED; USED WITH GEAR REDUCER.	OXIDATION DITCH, APPLIED EITHER AS AN AERATED LAGOON OR AS AN ACTIVATED SLUDGE.	MODERATE INITIAL COST. GOOD MAINTENANCE ACCESSIBILITY.	SUBJECT TO OPERATIONAL VARIABLES WHICH MAY AFFECT EFFICIENCY; TANK GEOMETRY IS LIMITED.	0.43 - 0.60
SUBMERGED TURBINE	UNITS CONTAIN A LOW SPEED TURBINE AND PROVIDE COMPRESSED AIR TO DIFFUSER RINGS OR OPEN PIPE. FIXED-BRIDGE APPLICATION.	SAME AS FOR BUBBLER DIFFUSER.	GOOD MIXING; HIGH CAPACITY INPUT PER UNIT VOLUME; DEEP TANK APPLICATION; OPERATIONAL FLEXIBILITY. NO ICING OR SPLASH.	REQUIRE BOTH GEAR REDUCER AND BLOWER; HIGH TOTAL POWER REQUIREMENTS; HIGH COST.	0.29 - 0.43

* REPORTED EFFICIENCY VARIES BECAUSE OF TANK GEOMETRY, DESIGN AND OTHER FACTORS.

d. Diffuser systems shall be capable of providing for the diurnal peak oxygen demand or 200% of the design average day oxygen demand, whichever is larger. The air diffusion piping and diffuser system shall be capable of delivering normal air requirements with minimal friction losses.

Air piping systems should be designed such that total head loss from the blower outlet (or silencer outlet where used) to the diffuser inlet does not exceed 3.4 KPa at average operating conditions.

The spacing of diffusers should be in accordance with the oxygen requirements through the length of the channel or tank and should be designed to facilitate adjustment of their spacing without major revision to air header piping. Fifty per cent blanks should be provided in at least the first half of the aeration system, for possible addition of more diffusers if found necessary.

All plants shall be designed to incorporate removable diffusers that can be serviced and/or replaced without dewatering the tank.

e. Individual assembly units of diffusers shall be equipped with control valves, preferably with indicator markings for throttling, or for complete shutoff. Diffusers in any single assembly shall have substantially uniform pressure loss.

f. Air filters shall be provided in numbers, arrangements and capacities to furnish at all times an air supply sufficiently free from dust to prevent damage to blowers and clogging of the diffuser system used. Blowers must have silencers, flexible connections and gauges.

g. Blowers which require internal lubrication are not desirable because of the danger of diffuser clogging from oil being carried in the air stream. Water-piston-type compressors are not desirable because they increase the condensation in the air system, resulting in a more severe corrosion problem in the piping and greater pressure loss required to pass the condensate through the diffusers.

7.4.1.2.8 Mechanical Aeration Systems

a. Oxygen Transfer Performance

The mechanism and drive unit shall be designed for the expected conditions in the aeration tank in terms of the power performance. Certified testing shall verify mechanical aerator performance. In the absence of specific design information, the oxygen requirements shall be calculated using a transfer rate not to exceed 1.22 kg 0_2/kW·h in clean water under standard conditions.

b. Design Requirements

1. maintain a minimum of 2.0 mg/L dissolved oxygen in the mixed liquor at all times throughout the tank or basin;
2. maintain all biological solids in suspension;

3. meet maximum oxygen demand and maintain process performance with the largest unit of service; and

4. provide for varying the amount of oxygen transferred in proportion to the load demand on the plant.

5. provide that motors, gear housing, bearings, grease fittings, etc., be easily accessible and protected from inundation and spray as necessary for the proper functioning of the unit.

c. Winter Protection

Due to high heat loss, the mechanism, as well as subsequent treatment units, shall be protected from freezing where extended cold weather conditions occur.

7.5 ROTATING BIOLOGICAL CONTACTORS

7.5.1 General

7.5.1.1 Applicability

The Rotating Biological Contactor (RBC) process may be used where sewage is amenable to biological treatment. The process may be used to accomplish carbonaceous and/or nitrogenous oxygen demand reductions.

Considerations for the rotating biological contactor (RBC) process should include:

- Raw sewage amenability to biological treatment;
- Pretreatment effectiveness including scum and grease removal;
- Expected organic loadings, including variations;
- Expected hydraulic loadings, including variations;
- Treatment requirements, including necessary reduction of carbonaceous and/or nitrogenous oxygen demand;
- Sewage characteristics, including pH, temperature, toxicity, nutrients;
- Maximum organic loading rate of active disc surface area;
- Minimum detention time at maximum design flow.

7.5.1.2 Winter Protection (Enclosures)

Wastewater temperature affects rotating contactor performance. Year-round operation requires that rotating contactors be covered to protect the biological growth from cold temperatures and the excessive loss of heat from the wastewater with the resulting loss of performance.

Enclosures shall be constructed of a suitable corrosion resistant material. Windows or simple louvred mechanisms which can be opened in the summer and closed in the winter shall be installed to provide adequate ventilation. To minimize condensation, the enclosure should be adequately insulated and/or heated. Mechanical ventilation should be supplied when the RBC's are contained within a building provided with interior access for personnel.

7.5.1.3 ***Required Pretreatment***

RBC's must be preceded by effective settling tanks equipped with scum and grease collecting devices, unless substantial justification is submitted for other pretreatment devices which provide for effective removal or grit, debris and excessive oil or grease prior to the RBC units. Bar screening or comminution are not sole means of pretreatment.

7.5.1.4 ***Flow Equalization***

For economy of scale, the peaking factor of maximum flow to average daily flow should not exceed 3. Flow equalization should be considered in any instance where the peaking factor exceeds 2.5.

7.5.1.5 ***Operating Temperature***

The temperature of wastewater entering any RBC should not drop below 13°C unless there is sufficient flexibility to decrease the hydraulic loading rate or the units have been increased in size to accommodate the lower temperature. Otherwise, insulation or additional heating must be provided to the plant.

7.5.1.6 ***Design Flexibility***

Adequate flexibility in process operation should be provided by considering one or more of the following:

a) Variable rotational speeds in first and second stages;
b) Multiple treatment trains;
c) Removable baffles between all stages;
d) Positive influent flow control to each unit or flow train;
e) Positively controlled alternate flow distribution systems;
f) Positive airflow metering and control to each shaft when supplemental operation or air drive units are used;
g) Recirculation of secondary clarifier effluent.

7.5.1.7 ***Hydrogen Sulphide***

When higher than normal influent or sidestream hydrogen sulphide concentrations are anticipated, appropriate modifications in the design should be made.

7.5.2 **Unit Sizing**

The Designer of an RBC system shall conform to the following design criteria, unless it can be shown by thorough documentation that other values or procedures are appropriate. This documentation may include detailed design calculations, pilot test results, and/or manufacturer's empirical design procedures. It should be noted that use of manufacturer's design procedures should be tempered with the realization that they are not always accurate and in some cases can substantially overestimate attainable removals.

7.5.2.1 Unit Sizing Considerations

Unit sizing shall be based on experience at similar full-scale installations or thoroughly documented pilot testing with the particular wastewater. In determining design loading rates, expressed in units of volume per day per unit area of media covered by biological growth, the following parameters must be considered:

a) design flow rate and influent waste strength;

b) percentage of BOD to be removed;

c) media arrangement, including number of stages and unit area in each stage;

d) rotational velocity of the media;

e) retention time within the tank containing the media;

f) wastewater temperature; and

g) percentage of influent BOD which is soluble.

In addition to the above parameters, loading rates for nitrification will depend upon influent total kjeldahl nitrogen (TKN), pH and allowable effluent ammonia nitrogen concentration.

7.5.2.2 Hydraulic Loading

Hydraulic loading to the RBC's should range between 75 to 155 L/m^{2}d of media surface area without nitrification, and 30 to 80 L/s with nitrification.

7.5.2.3 Organic Loading

The RBC process is approximately first order with respect to BOD removal; ie., for a given hydraulic loading (or retention time) a specific percent BOD reduction will occur, regardless of the influent BOD concentration. However, BOD concentration does have a moderate effect on the degree of treatment, and thus the possibility of organic overloading in the first stage. With this in mind, organic loading to the first stage of an RBC train should not exceed 0.03 to 0.04 kg BOD/m^{2}d or 0.012 to 0.02 kg BOD soluble/m^{2}d.

Loadings in the higher end of these ranges will increase the likelihood of developing problems such as heavier than normal biofilm thickness, depletion of dissolved oxygen, nuisance organisms, and deterioration of overall process performance. The structural capacity of the shaft; provisions for stripping biomass; consistently low influent levels of sulfur compounds to the RBC units; the media surface area required in the remaining stages; and the ability to vary the operational mode of the facility may justify choosing a loading in the high end of the range, but the operator must carefully monitor process operations.

7.5.2.4 Tank Volume

For purposes of plant design, the optimum tank volume is measured as wastewater volume held within a tank containing a shaft of media per unit of growth covered surface on the shaft, or litres per square metre (L/m^2). The optimum tank volume determined when treating domestic wastewater up to 300 mg/L BOD is 0.042 L/m^2, which takes into account wastewater displaced by the media and attached biomass. The use of tank volumes in excess of 0.042 L/m^2 does not yield corresponding increases in treatment capacity when treating wastewater in this concentration range.

7.5.2.5 Detention Time

Based on a tank volume of 0.042 L/m², the detention time in each RBC stage should range between 40 to 120 minutes without nitrification, and 90 to 250 minutes with nitrification.

7.5.2.6 Media Submergence and Clearance

RBC's should operate at a submergence of approximately 40 percent based on total media surface area. To avoid possible shaft overstressing and inadequate media wetting, the liquid operating level should never drop below 35 percent submergence. Media submergence of up to 95 percent may be allowed if supplemental air is provided. A clearance of 10 to 23 cm. between the tank floor and the bottom of the rotating media should be provided so as to maintain sufficient bottom velocities to prevent solids deposition in the tank.

7.5.3 Design Considerations

7.5.3.1 Unit Staging

The arrangement of media in a series of stages has been shown to significantly increase treatment efficiency. It is therefore recommended that an RBC plant be constructed in at least four stages for each flow path (or four zones of media area).

Four stages may be provided on a single unit by providing baffles within the tank. For small installations where the total area requirements dictate two units per flow path, two units may be placed in series with a single baffle in each tank, thus providing the minimum of four stages. For larger installations requiring four or more units per flow path, the units may be placed in a series within the flow path, with each unit itself serving as a single stage. Generally, though, plants requiring more than four stages should be constructed in a series of parallel floor trains, each comprised of four separate stages.

Wastewater flow to RBC units may be either perpendicular or parallel to the media shafts.

7.5.3.2 Tankage

RBC units may be placed in either steel or concrete tankage with baffles when required, and constructed of a variety of materials. The design of the tankage must include:

1. Adequate structural support for the RBC and drive unit;
2. Elimination of the "dead" areas;
3. Satisfactory hydraulic transfer capacity between stages of units; and
4. Considerations for operator safety.

The structure should be designed to withstand the increased loads which could result if the tank were to be suddenly dewatered with a full biological growth on the RBC units. The sudden loss of buoyancy resulting from unexpected tank dewatering could increase the bearing support loadings by as much as 40%.

Provisions for operator protection can be included in the tankage design by setting the top of the RBC tankage about one foot above the surrounding floor and walkways, with handrails placed along the top of the tankage, to provide an effective barrier between the operator and exposed moving equipment. The high tank walls will also prevent loss or damage by any material accidentally dropped in the vicinity of the units and entering the tankage.

7.5.3.3 *High Density Media*

Except under special circumstances, high density media should not be used in the first stage. Its use in subsequent stages should be based on appropriate loading criteria, structural limitations of the shaft and media, and media configuration.

7.5.3.4 *Shaft Rotational Velocity*

The peripheral velocity of a rotating shaft should be approximately 18 m/min for mechanically driven shaft, and between 9 and 18 m/min for an air driven shaft. Provision should also be made for rotational speed control and reversal.

7.5.3.5 *Biomass Removal*

A means for removing excess biofilm growth should be provided, such as air or water stripping, chemical additives, rotational speed control/reversal, etc.

7.5.3.6 *Dissolved Oxygen Monitoring*

First-stage dissolved oxygen (DO) monitoring should be provided. The RBC should be able to maintain a positive DO level in all stages.

7.5.3.7 *Supplemental Air*

Periodic high organic loadings may require supplemental aeration in the first stage to promote sloughing of biomass.

7.5.3.8 *Side Stream Inflows*

The type and nature of side stream discharges to an RBC must be evaluated, and the resulting loads must be added to the total facility influent loads. Anaerobic digesters increase ammonia nitrogen loadings, and sludge conditioning processes such as heat treatment contribute increased organic and ammonia nitrogen loadings. Whenever septic tank discharges comprise part of the influent wastewater or any unit processes are employed that may produce sulfide ahead of the RBC units, the additional oxygen demand associated with sulfide must be considered in system design.

7.5.3.9 *Recirculation*

Consideration should be given to providing recirculation of RBC effluent flow. This may be necessary during initial start-up and when the inflow rate is reduced to extremes.

For small installations, such as those serving an industrial park or school, the inflow over weekends or at holiday periods may drop to zero. During such periods, the lack of incoming organic load will cause the media biogrowth to enter the endogenous respiration phase where portions of the biogrowth become the food source or substrate for other portions of the biogrowth. If this condition lasts long enough, all of the biogrowth will eventually be destroyed. When this condition is allowed to exist, the RBC process does not have adequate biogrowth to provide the desired treatment when the inflow restarts.

If flow can be recycled through the sludge holding/treatment units and then to the RBC process, an organic load from the sludge units can be imposed on the RBC process. This imposed load will help to maintain the biogrowth and, as a secondary benefit, help stabilize and reduce the sludge.

When any new facility is first started, the biogrowth is slow to establish. If it is desired to build up the biogrowth before directing all of the inflow to the RBC process (as when the RBC is replacing an older existing process) some inflow may be directed to the RBC process and recycled.

In the first few days, minimal biogrowth will develop with only minimal removal of the organic load. By recycling, the unused organic load again becomes available to the biogrowth. As the biogrowth develops, the recycle rate should be reduced, with new inflow added to increase the organic load. As the biogrowth develops further, the recycle is eventually reduced to zero with all of the inflow being the normal RBC influent.

7.5.3.10 Load Cells

Load cells, especially in the first stage(s), can provide useful operating and shaft load data. Where parallel trains are in operation, they can pinpoint overloaded or underloaded trains. Stop motion detectors, rpm indicators and clamp-on ammeters are also potentially useful monitoring instruments.

Therefore, load cells shall be provided for all first and second stage shafts. Load cells for all other shafts in an installation are desirable.

7.5.3.11 Shaft Access

In all RBC designs, access to individual shafts for repair or possible removal must be considered. Bearings should also be accessible for easy removal and replacement if necessary. Where all units in a large installation are physically located very close together, it may be necessary to utilize large off-the-road cranes for shaft removal. Crane reach, crane size, and the impact of being able to drain RBC tankage and dry a unit prior to shaft removal should all be considered when designing the RBC layout.

7.5.3.12 Structural Design

The designer should require the manufacturer to provide adequate assurance that the shaft and media support structures are protected from structural failure for the design life of the facility. Structural designs should be based on appropriate American Welding Society (AWS) stress category curves modified as necessary to account for the expected corrosive environment. All fabrication during construction should conform to AWS welding and quality control standards.

7.5.3.13 Energy Requirements

Energy estimates used for planning and design should be based on expected operating conditions such as temperature, biofilm thickness, rotational speed, type of unit (either mechanical or air driven), and media surface area instead of normalized energy data sometimes supplied by equipment manufacturers. Care should be taken to assure that manufacturers' data are current and reflect actual field-validated energy usage.

Only high efficiency motors and drive equipment should be specified. The designer should also carefully consider providing power factor correction for all RBC units.

7.5.3.14 Nitrification Consideration

Effluent concentrations of ammonia nitrogen from the RBC process designed for nitrification are affected by diurnal load variations. Therefore, it may be necessary to increase the design surface area proportional to the ammonia nitrogen diurnal peaking rates to meet effluent limitations. An alternative is to provide flow equalization sufficient to insure process performance within the required effluent limitations.

7.6 WASTE STABILIZATION PONDS

7.6.1 Supplement to Pre-Design Report

7.6.1.1 General

The Pre-Design report shall contain pertinent information on location, geology, soil conditions, area for expansion and any other factors that will affect the feasibility and acceptability of the proposed project.

The following information must be submitted in addition to that required in Section 1.3.

7.6.1.2 Location in Relation to Nearby Facilities

The location and direction of all residences, commercial developments, recreational areas and water supplies within two kilometres of the proposed pond shall be included in the Pre-Design report.

7.6.1.3 Land Use Zoning

Land use zoning adjacent to the proposed pond site shall be included.

7.6.1.4 Soil Borings

Data from soil borings, conducted by an independent soil testing laboratory to determine subsurface soil characteristics and groundwater characteristics (including elevation and flow) of the proposed site and their effect on the construction and operation of a pond, shall also be provided. At least one boring shall be a minimum of 7.5 m in depth or into bedrock, whichever is shallower. If bedrock is encountered, rock type, structure and corresponding geological formation data should be provided. The boring shall be filled and sealed. The permeability characteristics of the pond bottom and pond seal materials shall also be studied.

7.6.1.5 Percolation Rates

Data demonstrating anticipated percolation rates at the elevation of the proposed pond bottom shall be included.

7.6.1.6 Site Description

A description, including maps showing elevations and contours of the site and adjacent area suitable for expansion shall be identified. Due consideration shall be given to additional treatment units and/or increased waste loadings or determining load requirements.

7.6.1.7 *Location of Field Tile*

The location, depth, and discharge point of any field tile in the immediate area of the proposed site shall be identified.

7.6.1.8 *Sulfate Content of Water Supply*

Sulfate content of the basic water supply shall be determined.

7.6.1.9 *Well Survey*

A pre-construction survey of all nearby wells (water level and water quality) is mandatory.

7.6.2 Location

7.6.2.1 *Distance From Habitation*

A stabilization basin site should be located as far as practicable, with a minimum of 150 m from isolated habitation and 300 m from built up areas or areas which may be built up within a reasonable future period. Consideration should be given to site specifics such as topography, prevailing winds, forests, etc.

A minimum distance of 100 m from public roads and highways is also recommended. Aerated stabilization basins separation distances shall be considered the same as mechanical plants. (Section 4.2.2)

7.6.2.2 *Prevailing Winds*

If practicable, ponds should be located so that local prevailing winds will be in the direction of uninhabited areas.

7.6.2.3 *Surface Runoff*

Location of ponds in watersheds receiving significant amounts of storm water runoff is discouraged. Adequate provision must be made to divert storm water runoff around the ponds and protect pond embankments from erosion.

7.6.2.4 *Groundwater Pollution*

Existing wells which serve as drinking water sources shall be protected from health hazards. Possible travel of pollutants through porous soils and fissured rocks should be objectively evaluated to safeguard the wells. A pond shall be located as far as practicable, with a minimum of 300 m from any well used as a drinking water source.

A minimum separation of 1.2 m between the bottom of the pond and the maximum groundwater elevation shall be maintained.

7.6.2.5 *Protection of Surface Water Supplies*

Stabilization basins shall be located downhill, downstream and remote from all sources of surface water supplies (lakes and rivers). The following minimum distances shall be employed as the criteria:

Minimum Distance from a Lake or River to the Centre of a Dyke of a Proposed Stabilization Basin	Remarks
120 m	Lined stabilization basin, pervious soil
75 m	Lined stabilization basin, impervious soil

7.6.2.6 *Geology*

Ponds shall not be located in areas which may be subjected to karstification (i.e. sink holes or underground streams generally occurring in areas underlain by limestone or dolomite).

A minimum separation of 3 m between the pond bottom and any bedrock formation is recommended.

7.6.2.7 *Floodplains*

A pond shall not be located within the 100 year floodplain.

7.6.3 Definitions

7.6.3.1 *Aerobic StabilizationBasin*

Aerobic ponds are shallow basins used for Wastewater Treatment. The organic contaminants in the wastewater are degraded by aerobic and facultative bacteria. The basins characteristically receive a light organic loading. They are used primarily to achieve additional organic removal following conventional wastewater treatment. Dissolved oxygen is furnished by oxygen transfer between the air and water surface, and by photosynthetic algae. The amount of oxygen supplied by natural surface re-aeration depends largely on wind-induced turbulence.

7.6.3.2 *Facultative Stabilization Basins*

The facultative pond is divided into an aerobic layer at the top and an anaerobic layer on the bottom. The aerobic layer is generated by algae which produce oxygen by photosynthesis. Settleable solids are permitted to accumulate on the pond bottom, and are broken down anaerobically. Waste stabilization is accomplished by a combination of anaerobic, aerobic, and a preponderance of facultative organics interacting with the wastewater.

7.6.3.3 *Aerated Stabilization Basins*

An aerated stabilization basin may be aerated aerobic (completely mixed) or aerated facultative. It does not depend on algae and sunlight to furnish DO or bacterial respiration, but instead uses diffusers or other mechanical aeration devices to transfer the major portion of oxygen and to create some degree of mixing. Because of the mixing, removal of suspended solids in the stabilization basin effluent is an important consideration.

Aerated stabilization basins can be described as heavily loaded oxidation basins, or very lightly loaded activated sludge systems. The micro-organisms responsible for the organic breakdown tend to be similar to those found in activated sludge systems.

Aerated stabilization basins may be used in series with aerobic stabilization basins. In such cases, the primary purpose of the stabilization basin without aeration is for solids removal.

7.6.4 Application, Advantages and Disadvantages of Different Stabilization Basin Types

Table 7.4 presents advantages and disadvantages of the different pond and stabilization basin types for various applications.

TABLE 7.4 - APPLICATION, ADVANTAGES AND DISADVANTAGES OF THE DIFFERENT STABILIZATION BASIN TYPES

PARAMETER	UNAERATED AEROBIC	FACULTATIVE	AERATED	
			AEROBIC	FACULTATIVE
APPLICATION	NUTRIENT REMOVAL; TREATMENT OF SOLUBLE ORGANIC WASTES; SECONDARY EFFLUENTS	TREATMENT OF RAW DOMESTIC AND INDUSTRIAL WASTES	TREATMENT OF RAW DOMESTIC AND INDUSTRIAL WASTES	TREATMENT OF RAW DOMESTIC AND INDUSTRIAL WASTES
ADVANTAGES	LOW OPERATING AND MAINTENANCE COSTS	LOW OPERATING AND MAINTENANCE COSTS;	SMALL VOLUME AND AREA; RESISTANCE TO UPSETS	SMALL VOLUME AND AREA; RESISTANCE TO UPSETS
DISADVANTAGES	LARGE VOLUME AND AREA; POSSIBLE ODOURS	LARGE VOLUME AND AREA; POSSIBLE ODOURS	SIGNIFICANT MAINTENANCE AND OPERATING COSTS; HIGH SOLIDS IN EFFLUENT; FOAMING	MAINTENANCE AND OPERATION COSTS; FOAMING

7.6.5 Basis of Design

7.6.5.1 Waste Stabilization Basins

7.6.5.1.1 Holding Capacity Requirements

Before the design of a waste stabilization pond system can be initiated, the designer shall determine the following:

- whether the stabilization basin can be continuously discharged or must operate on a fill-and-draw basis;
- the period of the year if any, when discharge will not be permitted;
- what discharge rates will be permitted with fill-and-draw stabilization basin and what, if any, provision must be made for controlling effluent discharge rates in proportion to receiving stream flow rates;
- what the minimum time for discharge of stabilization basin cell contents should be for fill-and-draw systems.

The holding capacity of ponds shall be based upon average daily sewage flow rates, making a special allowance for net precipitation entering the cells.

7.6.5.1.2 Area and Loadings

One hectare of water surface should be provided for each 250 design population or population equivalent. In terms of BOD, a loading of 22 kg BOD_5/ha·day should not be exceeded. Higher or lower design loadings will be judged after review of material contained in the Pre-Design report and after a field investigation of the proposed site by the regulatory authority.

Due consideration shall be given to possible future municipal expansion and/or additional sources of wastes when the original land acquisition is made. Suitable land should be available at the site for increasing the size of the original construction.

Where substantial ice cover may be expected for an extended period, it may be desirable to operate the facility to completely retain winter-time flows.

Design variables such as pond depth, multiple units, detention time and additional treatment units must be considered with respect to applicable standards for BOD_5, total suspended solids (TSS), fecal coliforms, dissolved oxygen (DO) and pH.

7.6.5.1.3 Flow Distribution

The main inlet sewer or forcemain should terminate at a chamber which permits hydraulic and organic load splitting between the stabilization basin cells. The ability to introduce raw sewage to all cells is desirable, but as a minimum, there must be a capability to divide raw sewage flows between enough cells to reduce the BOD_5 loading to 22 kg BOD_5/ha·day, or less.

The inlet chamber should be provided with a lockable aluminum cover plate or grating, divided into small enough sections to permit easy handling.

7.6.5.1.4 Typical Performance Potentials

Atlantic region environmental conditions are expected to facilitate the following performance of stabilization ponds treating typical domestic wastes:

Winter efficiency	=	70% BOD removal
Summer efficiency	=	80% BOD removal
Organic Load	=	22 kg BOD_5/ha·day
Liquid depth	=	1.5 to 1.8 m
Suspended Solids removal	=	80% but may decrease with increasing algal concentrations

7.6.5.1.5 Controlled -Discharge Stabilization Ponds

For controlled-discharge systems, the area specified as the primary ponds should be equally divided into two cells. The third or secondary cell volume should, as a minimum, be equal to the volume of each of the primary cells.

In addition, the design should permit for adequate elevation difference between primary and secondary ponds to permit gravity filling of the secondary from the primary. Where this is not feasible, pumping facilities may be provided.

7.6.5.1.6 Flow-Through Pond Systems

At a minimum, primary cells shall provide adequate detention time to maximize BOD removal. Secondary cells should then be provided for additional detention time with depths to two meters to facilitate both solids and coliform reduction.

7.6.5.1.7 Tertiary Pond

When ponds are used to provide additional treatment for effluents from existing or new secondary sewage treatment works, the reviewing authority will, upon request, establish BOD loadings for the pond after due consideration of the efficiencies of the preceding treatment units.

7.6.5.2 Aerated Stabilization Basins

7.6.5.2.1 General

Aerated ponds can be either aerobic or facultative. An aerated aerobic pond contains dissolved oxygen through the whole system with no anaerobic zones. The pond shape and the aerating power provides complete mixing. The aerated facultative pond provides a partially mixed condition which will cause an anaerobic zone to develop at the bottom as suspended solids settle due to low velocity in the system.

a. Aerated Aerobic Stabilization Basins

In general, an aerated pond can be classified as an aerobic pond (complete mixed) if the mechanical aeration power level is above six watts per cubic meters of maximum storage.

Aerated aerobic ponds should be designed to maintain complete mixing with bottom velocities of at least 0.15 m/s. It is important that sufficient mixing power be provided.

Quiescent settling areas adjacent to the aerated cell outlets or the addition of suspended solids removal processes such as a clarifier must follow aerated aerobic treatment, to insure compliance with suspended solids discharge requirements. In most cases, a minimum detention time of one day is required to achieve solids separation. Algae growth should be limited by controlling the hydraulic detention time to two days or less. Water depth of not less than one meter shall be maintained to control odours arising from anaerobic decomposition. Adequate provision must be made for sludge storage so that the accumulated solids will not reduce the actual detention time.

b. Aerated Facultative Stabilization Basins

Aerated facultative ponds should be designed to maintain a minimum of two mg/L of dissolved oxygen (DO) in the upper zone of the liquid.

The aeration system must be able to transfer up to 1.0 kilogram of oxygen per kilogram of BOD_5 applied uniformly throughout the pond when the water temperature is $20^{o}C$. The organic loading rate should be maintained between 0.031 and 0.048 $kg/m^3 \cdot day$.

The escape of algae into the effluent should be controlled by providing a quiescent area adjacent to each cell outlet with an overflow rate of 32 $m^3/m^2 \cdot d$. If multiple aerated facultative cells are used, all cells following the first one shall have diminished aeration capacity to permit additional settling.

Whenever possible, provisions should be provided for recirculating part (5-10%) of the final aeration cell effluent back into the influent in order to maintain a satisfactory mix of active micro-organisms.

7.6.5.2.2 Design Parameters

7.6.5.2.2.1 Detention Time

The mean cell residence time of an aerated pond should ensure that the suspended micro-organisms have adequate detention time to transform non-settling and dissolved solids into settleable solids, an adequate factor of safety is provided for periods of high hydraulic loading and that the detention time in the aerated pond is controlled by the rate of metabolism during the coldest period of the year.

As a minimum, the detention time should reflect 85% BOD_5 removal from November to April, being based on good and efficient operation of the aeration equipment. For the development of final design parameters, it is recommended that actual experimental data be developed; however, the aerated pond system design for minimum detention time may be estimated using the following formula:

$$t = \frac{E}{2.3\,K_1 \times (100-E)}$$

Where:

t = detention time, days

E = percent of BOD_5 to be removed in an aerated pond

K_1 = reaction coefficient, aerated stabilization basin, base 10. For normal domestic sewage, the K_1 value may be assumed to be 0.12/d at 20°C and 0.06/d at 1°C.

The reaction rate coefficient for domestic sewage which includes some industrial wastes, other wastes and partially treated sewage must be determined experimentally for various conditions which might be encountered in the aerated ponds. Conversion of the reaction rate coefficient at other temperatures shall be made based on experimental data. Additional storage volume should be considered for sludge and ice cover.

7.6.5.2.2.2 Oxygen Requirement

Oxygen requirements generally will depend on the BOD loading, the degree of treatment and the concentration of suspended solids to be maintained. Aeration equipment shall be capable of maintaining a minimum dissolved oxygen level of two mg/L in the ponds at all times.

The oxygen requirements should meet or exceed the peak 24 hours summer loadings. A safety factor of up to two should be considered in designing oxygen supply equipment based on average BOD_5 loadings. The amount of oxygen requirement has been found to vary from 0.7 to 1.5 times the amount of BOD_5 removed. Suitable protection from weather shall be provided for electrical control.

7.6.5.3 Industrial Wastes

Due consideration shall be given to the type and effects of industrial wastes on the treatment process. In some cases, it may be necessary to pretreat industrial or other discharges.

7.6.5.4 Multiple Units

At a minimum, a waste stabilization pond system shall consist of two cells designed to facilitate both series and parallel operations. The maximum size of a pond cell should be five hectares. A one cell system may be utilized in small installations. Larger cells may be permitted for bigger installations.

All systems should be designed with piping flexibility to permit isolation of any cell without affecting the transfer and discharge capabilities of the total system.

Requirements for multiple units in an aerated stabilization basin system shall be similar to those in an activated sludge system, including requirements for back-up aeration equipment.

7.6.5.5 Design Depth

The minimum operating depth should be sufficient to prevent growth of aquatic plants and damage to the dykes, control structures, aeration equipment and other appurtenances. In no case should pond depths be less than 0.6 metres.

7.6.5.5.1 Controlled-Discharge Stabilization Ponds

The maximum water depth shall be 1.8 meters in primary cells. Greater depths in subsequent cells are permissible although supplemental aeration or mixing may be necessary.

7.6.5.5.2 Flow-Through Stabilization Ponds

Maximum normal liquid depth should be 1.5 meters.

7.6.5.5.3 Aerated Pond Systems

In general, normal water depths vary from 1.2 to 3.6 meters when using surface aerators, however, consideration should be given to depths of up to five meters to minimize surface heat losses.

7.6.5.6 Pond Shape

Acute angles within any wastewater stabilization pond or aerated stabilization basin should be avoided. Square cells are preferred to long narrow rectangular cells. The long dimension of any pond should not align with the prevailing wind direction.

7.6.6 Pond Construction Details

7.6.6.1 Embankments and Dykes

7.6.6.1.1 Materials

Embankments and dykes shall be constructed of relatively impervious materials and compacted to at least 90 percent Standard Proctor Density to form a stable structure. Vegetation and other unsuitable materials should be removed from the area where the embankment is to be placed.

A soils consultant's report shall be required for all earthen berm construction to demonstrate the suitability of the soils. In certain instances a hydrogeologist's report may be required to assess possible impact on the water table. All topsoil must be stripped from the area on which the berms are to be constructed.

7.6.6.1.2 Top Width

The minimum embankment top width should be three meters to permit access of maintenance vehicles.

7.6.6.1.3 Maximum Slopes

Unless otherwise specified by a soil consultant's report, embankment slopes should not be steeper than:

a. Inner

Three horizontal to one vertical.

b. Outer

Three horizontal to one vertical.

7.6.6.1.4 Minimum Slopes

Embankment slopes should not be flatter than:

a. Inner

Four horizontal to one vertical. Flatter slopes are sometimes specified for larger installations because of wave action but have the disadvantage of added shallow areas conducive to emergent vegetation. Other methods of controlling wave action may be considered.

b. Outer

Outer slopes shall be sufficient to prevent surface water runoff from entering the ponds.

7.6.6.1.5 Freeboard

Minimum freeboard shall be one meter.

7.6.6.2 Erosion Control

a. Outer Dykes

The outer dykes shall have a cover layer of at least 100 mm of fertile topsoil to promote establishment of an adequate vegetative cover wherever rip rap is not utilized. Adequate vegetation shall be established on dykes from the outside toe to

0.5 m below the top of the embankment as measured on the slope. Perennial-type, low-growing, spreading grasses that minimize erosion and can be mowed are most satisfactory for seeding on dykes. Additional erosion control may also be necessary on the exterior dyke slope to protect the embankment from erosion due to severe flooding of a watercourse.

b. Inner Dykes

Alternate erosion control on the interior dyke slopes has become necessary for ponds because of problems associated with mowing equipment not designed to run on slopes as well as a lack of maintenance by the plant owner. The inner dykes shall have a cover of at least 200 mm of pit run gravel or other material graded in a manner to discourage the establishment of any vegetation. The material should be spread on dykes from the inside toe to the top of the embankment. Clean and sound rip rap or an acceptable equal shall be placed from 0.3 m above the high water mark to 0.6 m below the low water mark (measured on the vertical). Maximum size of rock used should not exceed 150 mm.

c. Top of Embankment

The top of the embankment used for access around the perimeter of the dykes shall have a cover layer of at least 300 mm of cover material similar to the one described in Section 7.4.6.2. b.

d. Additional Erosion Protection

Rip rap or some other acceptable method of erosion control is required as a minimum around all piping entrances and exits. For aerated cells the design should ensure erosion protection on the slopes and bottoms in the areas where turbulence will occur.

e. Erosion Control During Construction

Effective site erosion control shall be provided during construction according to applicable provincial documents such as “Erosion and Sedimentation Control Handbooks for Construction Sites”, if available in the province of jurisdiction. An approved erosion control plan is required before construction begins.

7.6.6.3 Vegetation Control

A method shall be specified which will prevent vegetation growth over the surface of the inner slope and top of the embankment.

7.6.6.4 Pond Bottom

7.6.6.4.1 Location

A minimum separation of 3.0 m between the cell bottom and bedrock is recommended. Cell bottoms should be located at least 1.2 m above the high groundwater level, in order to prevent inflow and/or liner damage.

7.6.6.5 Uniformity

The pond bottom should be as level as possible at all points. Finished elevations should not be more than 75 mm from the average elevation of the bottom.

7.6.6.5.1 *Vegetation*

The bottom shall be cleared of vegetation and debris. Organic material thus removed shall not be used in the dyke core construction. However, suitable topsoil relatively free of debris may be used as cover material on the outer slopes of the embankment as described in Design Section 7.4.6.2.a.

7.6.6.5.2 *Permeability Tests*

Permeability tests shall be carried out on the soil material at each proposed stabilization basin site except in cases where the soil is unmistakably impervious. The permeability tests may take either of two forms:

1. Laboratory tests on samples from below the proposed bottom of the stabilization basin and from the material to be used in the dykes.

2. Field seepage tests. These may be conducted in the following way. A pit shall be dug to the level of the proposed stabilization basin bottom and the bottom of the dug hole carefully cleaned. At least one test shall be conducted for every two hectares of stabilization basin area. A pipe with an internal diameter of at least 0.2 m and length of at least 1.2 m shall be carefully placed in a vertical position resting on the bottom of the hole. The hole shall be backfilled around the outside of the pipe to a height of one meter with carefully tamped soil. Particular care should be given to tamping the soil near the bottom.

The pipe shall be filled with water to a depth of 1.2 m. The water must be placed in the pipe gently so as not to disturb the soil at the bottom.

The drop in water level from a head of 1.2 m shall be recorded for each of at least 3 twenty-four hour periods, or until the readings become consistent. (Level shall be re-adjusted to 1.2 m at the beginning of each 24 hour period).

7.6.6.5.3 *Interpretation of Hydraulic Conductivity Measurements*

There can be major differences between laboratory and field hydraulic conductivity measurements. These differences are likely to occur because of complex geological and hydrogeological conditions, in situ and errors in measurement methods. The ratio of K (in situ) to K (laboratory) may be in the range of 0.38 to 64. The major reasons for higher field values are: 1) laboratory tests are generally run on homogeneous, clayey samples; 2) sand seams, fissures and other macrostructures in the field are not present in laboratory samples; 3) measurement of vertical K in the laboratory and horizontal K in the field; and 4) changes in soil structure, chemical characteristics of the permeant, air entrapment in laboratory samples and other errors associated with laboratory tests.

The value of K from a field test as described above may be obtained from the following equation:

$$K = (A/FDt) \times [\ln(h_1/h_2)]$$

Where,

A = area of standpipe

t = time for head change from h_1 to h_2

D = diameter of hole

h = head water above water table

F = 2.0 for a borehole with a flat bottom at an upper impervious boundary,

or

= 2.75 for a cased borehole with a flat bottom in the middle of a deep soil layer.

* Olson and Daniel (1981)

Alternatively, if the laboratory tests show a permeability greater than 1 x 10^{-6} cm per second, or if the drop in head of the field test exceeds 10 mm per 24 hours, then provision should be made to make the soil more impermeable, as indicated in Design Section 7.4.6.5.8.

7.6.6.5.4 Soil

Soil used in constructing the pond bottom (not including liner) and dyke cores shall be relatively incompressible and tight and compacted at or up to four percent above the optimum water content to at least 90 percent Standard Proctor Density. Soft pockets that would prevent sufficient compaction of the liner must be sub-excavated and replaced with suitable, compacted fill.

7.6.6.5.5 Liner

Stabilization Ponds shall be sealed such that seepage loss through the seal is as low as practicably possible. Liners consisting of soils or bentonite as well as synthetic liners may be considered, provided the permeability, durability and integrity of the proposed material can be satisfactorily demonstrated for anticipated conditions. Results of a testing program which substantiates the adequacy of the proposed liner must be incorporated into and/or accompany the Pre-Design report. Standard ASTM procedures or acceptable similar methods shall be used for all tests. Where clay liners are used, precautions should be taken to avoid erosion and desiccation cracking prior to placing the system in operation.

7.6.6.5.6 Seepage Control Criterion for Clay Liners

The seepage control criterion for municipal wastewater stabilization ponds and aerated stabilization basins utilizing clay liners specifies a maximum hydraulic conductivity, K, for the pond liner as a function of the liner thickness, L, and water depth, D, by the equation:

$$\text{Maximum K (m/s)} = \frac{4.6 \times 10^{-8}\,\text{m/s} \times \text{L (m)}}{\text{D (m)} + \text{L (m)}}$$

where all units are in meters and seconds.

For example, a compacted clay liner that is 0.5 m thick must have a hydraulic conductivity of about 1.3 x 10^{-8} m/s (1.3 x 10^{-6} cm/s) or less. The "K" obtained by the above expression corresponds to a percolation rate of pond water of less than 40 cubic meters per day per hectare at a water depth of 1.2 metres.

7.6.6.6 Seepage Control Criterion for Synthetic Liners

For synthetic liners, seepage loss through the liner shall not exceed the quantity equivalent through an adequate soil liner. For liner durability the minimum liner thickness for a HDPE liner shall be 1.5 mm (60 mil). The liner shall be underlain by a sand layer with a minimum thickness of 150 mm.

7.6.6.7 Site Drainage

Surface drainage must be routed around and away from cells. Field tiles within the area enclosed by the berms must be located and blocked so as to prevent cell content leakage. Measures must be taken, where necessary, to avoid disruption of field tile and surface drainage of adjacent lands, by constructing drainage works to carry water around the site.

7.6.7 Design and Construction Procedures for Clay Liners

7.6.7.1 Delineation of Borrow Deposit

The first step in designing a compacted clay liner is delineating a relatively uniform deposit of suitable borrow material, preferably from the pond cut or from a nearby borrow area. The required volume of clayey soil is equal to the surface area of the pond interior times the liner thickness (measured perpendicular to the bottom and side slope surfaces). A large reserve volume is recommended to ensure that there is indeed sufficient clay volume after removing silt and sand pockets and other unsuitable materials.

7.6.7.2 Liner Thickness

Recommended minimum compacted clay liner thicknesses are 0.5 m on the pond bottom and 0.7 m on the side slopes, to allow for weathering, variations in actual thickness, pockets of poor quality material that escape detection, etc. If a clay core in the dyke is preferred over an upstream clay blanket liner, then the core should be well keyed into the bottom liner. A minimum core width of three meters is suggested to allow economic and proper placement and compaction of the clay using large earth-moving equipment.

7.6.7.3 Hydraulic Conductivity of Compacted Clay

The in situ hydraulic conductivity of the compacted clay liner should be predicted from laboratory tests on the proposed clay borrow material. Several samples should be selected representing the range of material within the designated borrow zone, not just the better material. Permeability tests should be performed on the samples compacted to the required density (i.e. 95% of standard Proctor maximum dry density) at a moisture content anticipated in the field. It is recommended that the sensitivity of the compacted clay hydraulic conductivity to variations in density and moisture content be determined. The designer must be prepared to ensure that the soil is brought to the specified moisture content (i.e. by wetting), unless the natural moisture content is already suitable.

A laboratory value for K should be calculated from the weighted average of the individual tests. The weighting of each test value should be according to the estimated percent of the borrow volume that the individual sample represents.

It is recommended that the liner design be based on a K in situ that is one order of magnitude larger than the average K (lab), i.e.:

K (design) = K (in situ) = 10 x average K (lab)

The increase in the K value is a factor of safety to allow for the effects of macro-structure, poor quality borrow, etc., in the field. The K (design) and liner thickness values should meet the seepage criteria outlined in Section 7.4.6.5.6. If K (design) is too high, the more selective borrowing or adjustment of compaction moisture content could be investigated. Otherwise, an alternative liner material will be required.

7.6.7.4 *Subgrade Preparation*

Clay should not be placed directly over gravel or other materials that do not provide an adequate filter to prevent piping erosion of the liner.

7.6.7.5 *Liner Material Placement and Compaction*

The clay should be placed in uniform, horizontal lifts of about 150 mm maximum loose thickness. The liner should be constructed in at least three lifts. Thin lifts ensure more uniform density, better bonding between lifts and reduces the likelihood of continuous seepage channels existing in the liner. Large lumps, cobbles and other undesirable materials are more easily identified in thin lifts. Lumps of soil greater than 100 mm in maximum dimension should be broken up prior to compaction. As far as practical, the liner should be built up in a uniform fashion over the pond area, in order to avoid sections of butted fill where seepage paths may develop.

Each lift should be compacted within the specified moisture content range to the required density using heavy, self-propelled sheepsfoot compactors. Lift surfaces that have been allowed to dry out should be scarified prior to placing of the next lift. Lift surfaces that have degraded due to precipitation etc., should either be removed or allowed to dry to the required moisture content and then be re-compacted.

The completed liner should be smoothed out with a smooth-barrel compactor to reduce the liner surface area exposed to water absorption and swelling. The liner base should not be allowed to dry out or be exposed to freezing temperatures. Ideally, the liner should be flooded as soon as possible after construction and acceptance.

7.6.7.6 *Construction Control*

The most important form of quality control during construction of compacted clay liners will be observation and direction by the engineer. The characteristics of the desired liner material should be established in as much detail as possible (i.e. by color, texture, moisture content, plasticity or characteristic features such as the mineralogy of pebbles in till). Quick visual or index test identification by experienced field personnel is probably the best way to detect poor quality material. An indirect but simple way of controlling liner quality is to perform frequent in situ density and moisture content tests. The density and moisture content may then be related to hydraulic conductivity by the relationships established during the laboratory test program (see Section 7.4.7.3). The frequency of tests should be increased when soil conditions are variable. The tests may be used to statistically evaluate the overall liner properties and to assess suspect zones in the liner.

In situ density and moisture content tests should be carried out on a routine basis for each lift. Tests should be conducted on a grid pattern (say 30 x 30 m to 60 x 60 m grids for large ponds and at closer spacing for small ponds) and in suspect areas.

The completed liner may be assessed by performing in situ infiltration tests, which may be theoretically related to hydraulic conductivity values (see Section 7.4.6.5.3). It should be noted that the compacted clay liner is most likely to be partially saturated at the end of construction. The presence of five to ten percent air voids will result in an unsaturated K value that is somewhat higher than the saturated K value.

The completed liner may also be cored and the hydraulic conductivity of a trimmed sample can be tested in a suitable permeameter, i.e., odometer falling head tests or triaxial constant head tests. All holes created in the liner due to tests, stakes or other circumstances should be backfilled with well-compacted liner material.

7.6.7.7 Planning

The most important aspect of constructing a compacted clay liner may be the planning stage when the inspection engineer's role is defined, contract specifications are prepared and construction strategies are worked out. The engineer must have an adequate degree of control over material selection and methods of placement. The work procedure must be flexible with respect to earth movement.

Ideally, the borrow for a compacted clay liner would be the cut material just below the eventual pond invert. Thus, material may be cut and placed in a single operation for much of the pond liner area, although some stockpiling of borrow may be inevitable.

The lower lift of the liner might consist of reworked native soil broken up by tilling and re-compacted to eliminate fissures, etc. Nevertheless, the contract should allow for selective borrowing of cut material for liner use, for stockpiling, removal of undesirable materials and possible additional borrowing outside of the cut area.

7.6.8 Prefilling

Prefilling the pond should be considered in order to protect the liner, to prevent weed growth, to reduce odour and to maintain moisture content of the seal. However, the dykes must be completely prepared as described in Design Section 7.4.6.2.b before the introduction of water.

7.6.9 Influent Lines

7.6.9.1 Material

Any generally accepted material for underground sewer construction will be given consideration for the influent line to the pond. Unlined corrugated metal pipe should be avoided however, due to corrosion problems. In material selection, consideration must be given to the quality of the wastes, exceptionally heavy external loadings, abrasion, soft foundations and similar problems.

7.6.9.2 *Surcharging*

The design and construction of influent piping shall insure that where surcharging exists, due to the head of the pond, no adverse effects will result. These effects shall include basement flooding and overtopping of manholes.

7.6.9.3 *Forcemains*

Forcemains terminating in a sewage stabilization basin should be fitted with a valve immediately upstream of the stabilization basin.

7.6.9.4 *Location*

Influent lines should be located along the bottom of the pond so that the top of the pipe is below the average elevation of the pond seal. However, the pipe shall have adequate seal below it. The use of an exposed dyke to carry the influent line to the discharge points is prohibited.

7.6.9.5 *Point of Discharge*

The influent line to a square single celled pond should be essentially center discharging. Each square cell of a multiple celled pond operated in parallel shall have its own near center inlet but this does not apply to those cells following the primary cell, when series operation alone is used. Influent lines to single celled rectangular ponds should terminate at approximately the third point farthest from the outlet structure. Influent and effluent piping should be located to minimize short-circuiting within the pond. Consideration should be given to multi-influent discharge points for primary cells of 5 ha or larger.

All aerated cells shall have influent lines which distribute the load within the mixing zone of the aeration equipment. Consideration of multiple inlets should be closely evaluated for any diffused aeration system. For aerated stabilization basins the inlet pipe may go directly through the dyke and end at the toe of the inner slope.

7.6.9.6 *Influent Discharge Apron*

Inlet pipes should terminate with an upturned elbow, with the pipe extending 450 mm above the cell bottom.

The end of the discharge line shall rest on a suitable concrete apron large enough to prevent the terminal influent velocity at the end of the apron from causing soil erosion. A minimum size apron of one meter square shall be provided.

7.6.9.7 *Pipe Size*

The influent system shall be sized to permit peak raw sewage flow to be directed to any one of the primary cells. Influent piping should provide a minimum scouring velocity of 0.6 m/s.

7.6.10 Control Structure and Interconnecting Piping

7.6.10.1 *Structure*

Facility design shall consider the use of multi-purpose control structures to facilitate normal operational functions such as drawdown and flow distribution, flow and depth measurement, sampling, pumps for re-circulation, chemical additions and mixing and minimization of the number of construction sites within the dykes.

Control structures shall

(a) be accessible for maintenance and adjustment of controls;

(b) be adequately ventilated for safety and to minimize corrosion;

(c) be locked to discourage vandalism;

(d) contain controls to permit water level and flow rate control, complete shutoff and complete draining;

(e) be constructed of non-corrodible materials (metal-on-metal contact in controls should be of similar alloys to discourage electro-chemical reactions); and

(f) be located to minimize short-circuiting within the cell and avoid freezing and ice damage.

Recommended devices to regulate water level are valves, slide tubes, dual slide gates, or effluent chambers complete with a water level regulating weir. Regulators should be designed so that they can be preset to stop flows at any pond elevation.

7.6.10.2 Piping

All piping shall be of ductile iron or other acceptable material. The piping shall not be located within or below the liner. Pipes should be anchored with adequate erosion control.

7.6.10.2.1 Drawdown Structure Piping

a. Submerged Takeoffs

For ponds designed for shallow or variable depth operations, submerged takeoffs are recommended. Intakes shall be located a minimum of three meters from the toe of the dyke and 0.6 meters from the top of the liner and shall employ vertical withdrawal.

b. Multi-level Takeoffs

For ponds that are designed deep enough to permit stratification of pond content, multiple takeoffs are recommended. There shall be a minimum of 3 withdrawal pipes at different elevations. The bottom pipe shall conform to a submerged takeoff. The others should utilize horizontal entrance. Adequate structural support shall be provided.

c. Surface Takeoffs

For use under constant discharge conditions and/or relatively shallow ponds under warm weather conditions, surface overflow-type withdrawal is recommended. Design should evaluate floating weir box or slide tube entrance with baffles for scum control.

d. Maintenance Drawdown

All ponds shall have a pond drain to allow complete emptying, either by gravity or pumping, for maintenance. These should be incorporated into the above-described structures.

In aerated stabilization basins where a diffused air aeration system and submerged air headers are used, provision should be made to drain each stabilization basin (independently of others) below the level of the air header.

e. Emergency Overflow

All cells shall be provided with an emergency overflow system which overflows when the liquid reaches within 0.6 m of the top of the berms.

7.6.10.2.2 Hydraulic Capacity

The hydraulic capacity for continuous discharge structures and piping shall allow for at least the expected future peak sewage pumping rate.

The hydraulic capacity for controlled discharge systems shall permit transfer of water at a rate of 150 mm of pond water depth per day at the available head.

7.6.10.2.3 Interconnecting Piping

Interconnecting piping for multiple unit installations operated in series should be valved or provided with other arrangements to regulate flow between structures and permit flexible depth control. The interconnecting pipe to the secondary cell should discharge horizontally near the stabilization basin bottom to minimize need for erosion control measures and should be located as near the dividing dyke as construction permits. Interconnection piping shall enable parallel or series flow patterns between cells.

7.6.10.3 Location

The outlet structure and the inter-connecting pipes should be located i) away from the corners where floating solids accumulate and ii) on the windward side to prevent short-circuiting.

7.6.11 Miscellaneous

7.6.11.1 Fencing

The pond area shall be enclosed with a suitable fence to preclude livestock and discourage trespassing. The fence should be located on the outer dyke at a distance of 500 mm from the top outside edge of the embankment. Fencing should not obstruct vehicle traffic on top of the dyke. A vehicle access gate of sufficient width to accommodate equipment should be provided. All access gates should be provided with locks.

7.6.11.2 Access

An all-weather access road shall be provided to the pond site to allow year-round maintenance of the facility.

7.6.11.3 Warning Signs

Appropriate signs should be provided along the fence around the pond to designate the nature of the facility and warn against trespassing. At least one sign shall be provided on each side of the site and one for every 150 m of its perimeter.

7.6.11.4 Flow Measurement

Provisions for flow measurement shall be provided on the outlet. Safe access to the device should be made to permit safe measurement.

7.6.11.5 Groundwater Monitoring

An approved system of wells or lysimeters may be required around the perimeter of the pond site to facilitate groundwater monitoring. The need for such monitoring will be determined on a case-by-case basis.

7.6.11.6 Pond Level Gauges

Pond level gauges shall be provided.

7.6.11.7 Service Building

A service building for laboratory and maintenance equipment shall be provided, if required.

7.6.11.8 Liquid Depth Operation

Optimum liquid depth is influenced to some extent by stabilization basin area since circulation in larger installations permits greater liquid depth. The basic plan of operation may also influence depth. Facilities to permit operation at selected depths between 0.6 to 1.5 meters are recommended for operational flexibility. Where winter operation is desirable, the operating level can be lowered before ice formation and gradually increased to 1.5 meters by the retention of winter flows. In the spring, the level can be lowered to any desired depth at the time surface runoff and dilution water are generally at a maximum. Shallow operation can be maintained during the spring with gradual increased depths to discourage emergent vegetation in the summer months. In the fall, the levels can be lowered and again be ready for retention of winter storage.

7.6.11.9 Pre-Treatment and Post-Treatment

The wastewater shall be treated by bar screens before entering the stabilization basin.

The treated effluent shall be disinfected prior to discharging into the receiving water.

7.7 OTHER BIOLOGICAL SYSTEMS

New biological treatment schemes with promising applicability in wastewater treatment may be considered if the required engineering data for new process evaluation is provided in accordance with Section 4.4.2. A number of new biological systems are described below. These systems typically are manufactured by companies who hold proprietary designs and as proprietary information cannot be included in this manual the design data presented is fairly general in nature. A description of these systems mainly describing their application and typical loading rates is provided here. New treatment schemes may be added to the main section of this chapter when sufficient and adequate design data becomes available. These additions will be noted in the revision record.

7.7.1 Biological Aerated Filters

Biological aerated filters (BAFs) are submerged, granular media upflow filters, which treat wastewater by biologically converting carbonaceous and nitrogenous matter using biomass fixed to the media and physically capturing suspended solids within the media. The filters are aerated to remove carbonaceous matter and convert ammonia-nitrogen to nitrates via nitrification. Non-aerated filters in the presence of supplemental carbonaceous organic matter can convert nitrates to nitrogen gas through denitrification.

BAFs are designed either as co-current backwash or countercurrent backwash systems. the co-current backwash design has a nozzle deck supporting a granular media that has a specific gravity greater than 1.0. Pre-treated wastewater is introduced under the nozzle deck and flows up through a slightly expanded media bed, and effluent leaves the filter from above the media. Process air is introduced just above the nozzle deck (the bed is not aerated for denitrification). During backwash, wash water and air scour are introduced below the nozzle deck and flow up through the bed. Wash water is pumped to the head of the plant or directly to solids handling.

The countercurrent backwash BAF operates under the same general principles, except that the granular media has a specific gravity less than 1.0. Therefore, the media float and are retained from above by the nozzle deck. During backwash, wash water flows by gravity through the media. Process air is introduced below the media; therefore, scour air moves countercurrent to the wash water flow.

7.7.1.1 Design Features

The granular media bed for both designs typically is 3 to 4 m deep and the media are 3 to 6 mm in diameter. The media-specific surface area ranges from 500 to 2,000 m^2/m^3. The contact time in the media typically is 0.5 to 1.0 hour. The media bed is backwashed every 24 to 48 hours for 20 to 40 minutes using a wash water volume about three times the media volume. Backwash water from a single event is collected in a storage tank and returned to the head of the plant or directly to solids processing over a 1- to 2-hour period. Backwash water typically contains from 400 to 1,200 mg/L of suspended solids. The backwash water recycle flow can represent up to 20% of the raw influent wastewater flow. Most manufacturers have estimated that solids production from the BAF system is comparable to that of a conventional activated sludge system. Effluent pollutant concentrations from a single BAF cell increases for approximately 30 minutes following a backwash event, so a minimum of four cells should be included in any design to dampen these spikes.

The nozzle deck features polyethylene nozzles that prevent media loss and assist in evenly distributing flow across the bed. The reported media loss from the BAF system is less than 2% per year. The nozzle openings are slightly smaller than the media and require that influent be pre-treated with a fine screen to prevent plugging. Headloss across the media bed can be more than 2 m prior to backwash. In existing installations, the filters are constructed above grade. The combination of the tall structure (6 m) and headloss across the bed requires pumping influent flow to the BAF in most situations. In addition, the co-current designs require pumping of wash water, which is a significant, but intermittent, energy demand.

Process air is required in BAF cells that are removing carbonaceous organic matter (biochemical oxygen demand or BOD) and are nitrifying ammonia-nitrogen. The process aeration system consists of coarse- to medium-bubble diffusers on a stainless steel piping grid. Because of the difficulty in accessing the aeration grid, the diffusers are constructed as simply and reliably as possible. The amount of air that must be added to the system is determined by the oxygen demand of the biomass. Energy for process air can represent more than 80% of the energy demand in a BAF system.

7.7.1.2 Configurations

BAFs can operate in different process configurations, depending on the facilities, effluent goals, and wastewater characteristics. The process can follow either chemically assisted primary sedimentation or an activated sludge system. This level of treatment is required because of a BAF system's sensitivity to high influent BOD and suspended solids loadings. Following primary sedimentation, BAF cells can be operated for carbonaceous BOD removal or, under lower loading rates (less than 1.5 kg BOD/$m^3 \cdot d$), for both carbonaceous BOD and ammonia-nitrogen removal. A cell can operate in a nitrification mode following an activated sludge system or another BAF cell removing carbonaceous BOD. A denitrification BAF process can follow either an activated sludge or BAF system that is nitrifying.

7.7.1.3 Performance

The performance of BAFs in terms of allowable loading rates and effluent quality depends on influent wastewater quality and temperature. In general, higher organic or suspended solids influent loadings result in higher effluent concentrations. Adequate water velocity is necessary to provide scouring of the biomass and even flow distribution across the media bed. Inadequate water velocity can result in premature bed plugging; this is especially true for denitrification reactors in which the effects of air scouring are not present.

Factors that positively affect complete nitrification include:

- warm water temperature,
- adequate aeration and good air distribution, and
- low carbonaceous BOD and suspended solids loading.

Denitrification usually requires methanol addition, and water velocities must be greater than 10 m/h.

7.7.2 Moving Bed Biofilm Reactors

The patented MBBR process was developed by the Norwegian company Kaldnes Miløteknologi (KMT). The basic concept of the MBBR is to have continuously operating, noncloggable biofilm reactors with no need for backwashing or return sludge flows, low head-loss and high specific biofilm surface area. This is achieved by having the biomass grow on small carrier elements that move along with the water in the reactor. The movement is normally caused by coarse-bubble aeration in the aeration zone and mechanical mixing in an anoxic/anaerobic zone and mechanical mixing in an anoxic/anaerobic zone. However, for small plants, mechanical mixers are omitted for simplicity reasons and pulse aeration for a few seconds a few times per day can be used to move the biofilm carriers in anoxic reactors.

The biofilm carrier elements are made of 0.96 specific gravity polyethylene and shaped like small cylinders, with a cross in the inside of the cylinder and longitudinal fins on the outside. To keep the biofilm elements in the reactor, a screen of perforated places is placed at the outlet of the reactor. Agitation constantly moves the carrier elements over the surface of the screen; the scrubbing action prevents clogging.

Almost any size or shape tank can be retrofitted with the MBBR process. The filling of carrier elements in the reactor may be decided for each case, based on degree of treatment desired, organic and hydraulic loading, temperature and oxygen transfer capability. The reactor volume is totally mixed and consequently there is no "dead" space or unused space in the reactor. Organic loading rates for these reactors are typically in the order of 3.5 – 7.0 g BOD/m^2 of media surface area/d for BOD removal and less than 3.5 g BOD/m^2 of media surface area/d for nitrification.

7.7.3 Membrane Bioreactors

Membrane Bioreactors consist of a suspended growth biological reactor (activated sludge system variation) integrated with a microfiltration membrane system. The key to the technology is the membrane separator which allows elevated levels of biomass to degrade or remove the soluble form of the organic pollutants from the waste stream. These systems typically operate in the nanofiltration or microfiltration range which results in removal of particles greater than 0.01 and 0.1 µm, respectively.

7.7.3.1 Configuration

Membrane bioreactors can be configured in a number of different ways, however, the two main configuration differ by those in which the membranes are submersed directly in the bioreactor and those which contain external membrane process tankage. When membrane modules are submersed into the bioreactor, they are in direct contact with the wastewater and sludge. A vacuum is created within the hollow fibers by the suction of a permeate pump. The treated water passes through the membrane, enters the hollow fibers and is pumped out by the permeate pump. An air flow may be introduced to the bottom of the membrane module to create turbulence which scrubs and cleans the membrane fibers keeping them functioning at a high flux rate. The filtrate or permeate is then collected for reuse or discharge.

Outboard membrane processes operate in a similar manner however, the membranes are contained in a separate tank through which the wastewater requiring filtration constantly flows. Again air is often added for both treatment and membrane scouring purposes. The main difference between the two configurations lies in the membrane cleaning processes where membranes submersed within the aeration tanks must be removed for cleaning while outboard membranes are cleaned by evacuating the membrane tankage and providing for equalization during the cleaning procedures within the main aeration tank.

7.7.3.2 Process Description

The benefits of these processes are consistent effluent quality, reduced footprint, increased expansion capabilities within the same tankage, and ease of operation. Tertiary quality effluent is the normal output of a membrane bioreactor. Typical effluent quality is presented in Table 7.6.

Virtually no solids are lost via the permeate stream and the wasting of solids is reduced. As a result, the sludge age can be very accurately determined. Nitrification for ammonia removal is easily achieved by optimizing reactor and sludge age to specific wastewater characteristics and effluent requirements. Absolute control of the nitrifiers results in high nitrification rates even in winter periods and under adverse and unstable conditions.

If required, denitrification can be achieved with for membrane processes as when operating at a MLSS of 15,000 mg/L and higher, the mixed liquor rapidly becomes anoxic in the absence of a continuous stream of air. Furthermore, the high levels of biomass ensures that in the anoxic zone, at all times there is enough denitrifiers to efficiently convert the nitrates into nitrogen gas.

TABLE 7.6 – MEMBRANE BIOREACTOR EFFLUENT QUALITY

PARAMETERS	SECONDARY TREATMENT	TERTIARY TREATMENT	MEMBRANE BIOREACTOR
BOD (mg/L)	10-12	< 5	< 2
TSS (mg/L)	10-15	<1	<1
NH_3 (mg/L)	1-10	1-10	< 0.3
Total P (mg/L)	> 1	0.1-0.5	< 0.1
Total Coliforms (MPN/100mL)	> 1000	> 1000	< 100
Fecal Coliforms (MPN/100mL)	> 100	> 100	< 10
Sludge Yield (kg/kg BOD_5 removed)	0.3-0.6	0.3 -0.6	0.1-0.3

Chapter 8 EFFLUENT DISINFECTION

8.1 BASIS FOR DISINFECTION OF SEWAGE TREATMENT PLANT EFFLUENT

Disinfection of sewage treatment plant effluent **shall** be required in all cases, unless confirmed otherwise by the regulatory agencies.

The design shall consider meeting both the bacterial standards and the disinfectant residual limit in the effluent. The disinfection process should be selected after due consideration of waste characteristics, type of treatment process provided prior to disinfection, waste flow rates, pH of waste, disinfectant demand rates, current technology application, cost of equipment and chemicals, power cost, and maintenance requirements. The designer shall consider the provisions of the Federal "Transportation of Dangerous Goods Act" and the applicable Provincial Dangerous Goods Legislation when designing a disinfection system.

8.2 FORMS OF DISINFECTION

Chlorine is the most commonly used chemical for wastewater effluent disinfection. The forms most often used are liquid chlorine and calcium or sodium hypochlorite. Other disinfectants, including chlorine dioxide, ozone, bromine, or ultraviolet disinfection, may be accepted by the approving authority in individual cases. If chlorination is utilized, it may be necessary to dechlorinate if the chlorine level in the effluent would impair the natural aquatic habitat of the receiving stream. The use of chlorine capsules may be considered for small systems.

8.3 CHLORINATION

8.3.1 Design Guidelines

8.3.1.1 *Mixing*

The disinfectant shall be positively mixed as rapidly as possible, with a complete mix being effected in three seconds. This may be accomplished by either the use of a turbulent flow regime or a mechanical flash mixer.

8.3.1.2 *Diffusers*

A chlorine solution diffuser shall be placed ahead of the contact tank and near the vicinity of the mixing area.

8.3.1.3 *Contact Time and Residual*

A total chlorine residual of 0.5 mg/L is generally required. The required detention time shall be based upon the more stringent of either 30 minutes at design average daily flow or 15 minutes at the design peak hourly flow. The criteria to be used in the design shall be that which provides the largest volume for the contact tank.

8.3.1.4 *Coliform Levels*

Acceptable effluent coliform levels shall be based upon the results of the receiving water study and the receiving water quality guidelines.

8.3.1.5 *Contact Tank*

In order that the chlorine contact tank can provide the required detention, dead zones within the tank must be avoided and the flow through the tank must approach plug flow as closely as possible. Back-mixing within the contact tank must be avoided to prevent short-circuiting and the resulting poor disinfection results. Covered tanks are discouraged.

To approach a plug-flow regime, flow channels with length-to-width ratios of greater than 40:1 are required. Length-to-width ratios of 10:1 produce detention times of approximately 70 percent of the theoretical residence times. In rectangular tanks, longitudinal baffling to produce long, narrow flow channels with a serpentine flow pattern and with guide vanes at changes in direction is a preferred method.

Since some sedimentation occurs in chlorine contact tanks, provision should be made for periodic sludge removal from the chlorine contact tank(s). The drain should be valved. If it is necessary to take the contact tank out of operation for cleaning, and if short-term discontinuation of disinfection cannot be tolerated due to other critical uses made of the receiving waters, two contact basins shall be provided. In less critical situations, one contact basin will suffice provided that the bypass facilities are equipped with a chlorine application point for emergency disinfection.

8.3.2 Chlorination Facilities Design

8.3.2.1 *Feed Equipment*

8.3.2.1.1 *Capacity*

The chlorinator shall be sized according to the design flow of the treatment plant and its capacity may vary, depending on the uses and points of application of the chlorine. For disinfection, the capacity should be adequate to produce a concentration of residual chlorine in the plant effluent as measured by the standard DPD test such as to dependably and consistently obtain 0.5 mg/L chlorine residual after a contact period of 20-30 minutes under average flow conditions.

For normal domestic sewage, the following may be used as a guide in sizing chlorination facilities.

Table 8.1 Chlorine Dosage Requirements	
TYPE OF TREATMENT	**DOSAGE (mg/L)**
Raw Wastewater (Fresh)	6-15
Raw Wastewater (Septic)	12-25
Primary Effluent	5-20
Activated Sludge Plant Effluent	2-8
Trickling Filter Plant Effluent	3-10
RBC Plant Effluent	3-10
Tertiary Filtration Effluent	2-6
Nitrified Effluent	2-6

In order that effective disinfection can be maintained at all times, without the need to overdose excessively at low flow periods, the chlorine feed equipment should be paced by the effluent flow rate.

8.3.2.1.2 Standby Equipment and Spare Parts

Standby equipment of sufficient capacity should be available to replace the largest unit during shutdowns. Spare parts shall be available for all chlorinators to replace parts which are subject to wear and breakage.

8.3.2.1.3 Water Supply

An ample supply of water shall be available for operating the chlorinator. Where a booster pump is required, duplicate equipment should be provided and, when necessary, standby power. Protection of a potable water supply shall conform to the requirements of Section 3.2.8.

The use of a potable water supply for solution-feed chlorinators is the preferred method. However, in the case of small plants, plant effluent may be used for operating the chlorinator.

8.3.2.2 ***Odor Control***

Should odor control be a critical factor, additional capacity of a prechlorination system to the extent of about 80 percent of the raw wastewater chlorine demand shall be required during the warm summer days. It is not desirable to split the functions of the chlorinators, especially for large plants. One group shall be designed for prechlorination and another for disinfection. In the case of large plants, each group shall be interchangeable to facilitate a standby feature.

Prechlorination must be accomplished ahead of the first open structure in the plant and thereby reduce the escape of hydrogen sulphide gas into the atmosphere.

8.3.3 Chlorine Supply

8.3.3.1 *Type*

Chlorine is available for disinfection in gas, liquid (hypochlorite solution), and pellet (Hypochlorite tables) form. The type of chlorine should be carefully evaluated during the facility planning process. The use of chlorine gas or liquid will be most dependent on the size of the facility and the chlorine does required. Large quantities of chlorine, such as are contained in ton cylinders and tank cars, can present a considerable hazard to plant personnel and to the surrounding area, should such containers develop leaks. Both monetary cost and the potential public exposure to chlorine should be considered when making the final determination.

8.3.3.2 *Cylinders*

Seventy kilogram cylinders should be used when chlorine demand is less than 44 kilograms per day. Cylinders should be stored in an upright position with adequate support brackets and chains at 0.67 cylinder height for each cylinder.

8.3.3.3 *Tonne Containers*

The use of 900 kg containers should be considered where the average daily chlorine consumption is over 44 kilograms. A hoist or crane with a capacity of at least 2000 kg hall be provided for the handling of the ton containers.

8.3.3.4 *Tank Cars*

At large installations (chlorine consumption greater than 900 kg/day), consideration should be given to the use of tank cars, generally accompanied by liquid chlorine evaporators. Liquid chlorine lines from tank cars to evaporators shall be buried and installed in a conduit and shall not enter below grade spaces. Systems shall be designed for the shortest possible pipe transportation of liquid chlorine.

The tank car being used for the chlorine supply shall be located on a dead end, level track that is a private siding. The tank car shall be protected from accidental bumping by other railway cars by a locked derail device or a closed locked switch or both. The area shall be clearly posted "DANGER-CHLORINE". The tank car shall be secured by adequate fencing with gates provided with locks for personnel and rail access.

The tank car site shall be provided with a suitable operating platform at the unloading point for easy access to the protective housing or the tank car for connection of flexible feedlines and valve operation. Adequate area lighting shall be provided for night time operation and maintenance.

8.3.3.5 *Scales*

Scales of proper size shall be provided at all plants using chlorine gas. At large plants, scales of the indicating and recording type are recommended. At the least a platform scale shall be provided. Scales shall be of corrosion-resistant material. Scales shall be set on grade, or a ramp shall be built to facilitate the moving of cylinders on and off the scale platform. Scales should be sized so as to accommodate the maximum number of containers required to serve the maximum chlorine feed rate.

8.3.3.6 *Evaporators*

Where manifolding of several cylinders or 900 kg containers will be required to supply sufficient chlorine, consideration should be given to the installation of a chlorine evaporator to produce the quantity of gas required.

8.3.3.7 *Hoists*

Handling of 900 kg containers requires hoisting equipment. It is desirable to use a power-operated hoist and travel particularly when it is necessary to change containers frequently. Hoists should have a minimum capacity of 200 kg and be equipped with an approved type of lifting-beam container grab.

8.3.3.8 *Leak Detection and Controls*

A bottle of ammonium hydroxide solution of industrial strength (56%) should be available for detecting chlorine leaks. Where ton containers or tank cars are used, a leak repair kit approved by the Chlorine Institute shall be provided. Consideration should also be given to the provision of caustic soda solution reaction tanks for absorbing the contents of leaking 900 kg cylinders where such cylinders are in use. At large installations, consideration should be given to the installation of automatic gas detection and related alarm equipment.

8.3.4 Methods of Dosage Control

8.3.4.1 *Open Loop Flow Proportional Control*

Automatic proportioned-to-flow control consists of varying the rate of chlorine feed in proportion to the sewage flow as determined by a metering device. The dosage rate is manually set, and the control device varies the rate in relation to flow rate. The chlorinator may be either automatic or manual start and stop.

Usually over-chlorination is practised to insure results. Invariably, some chlorine is wasted by such a device.

8.3.4.2 *Closed-Loop Flow Proportional Control (Compound-Loop Arrangement with One Chlorine Analyzer):*

Chlorine residual analyzer provides feedback to the chlorinator. Flow signal and dosage signal each separately control the added chlorine feed with a compound-loop arrangement. If the residual is above the pre-determined level, the chlorine feed rate is reduced, and vice versa. In some designs, chlorine residual is measured at one point in the system and in other designs at 2 or 3 points.

8.3.4.3 *Close-Loop Flow Proportional Control (Compound-Loop Arrangement with Two Chlorine Analyzers*

This ideal system employs quantitative as well and qualitative feed control as in the previous case. However, the qualitative control is accomplished at two points in the flow stream. One sample is automatically collected immediately downstream from the point of chlorination (diffuser) and analyzed by another chlorine analyzer which (i) monitors the combined residual after a given contact time and (ii) adjusts the control point on the analyzer which controls the chlorine metering equipment. When the residual chlorine is more than the desired (pre-set) level, the chlorine feed rate is reduced and vice versa.

8.3.4.4 *Required Chlorine Control Systems*

Plants with proper qualitative and quantitative control systems are known to chlorinate effectively and efficiently. However, the plants without such controls show either inadequate performance (due to under dosage) or waste chlorine unnecessarily (by undue overdose). Higher than needed chlorine residuals may result in ecological damage to the receiving waters.

The following table summarizes chlorine control guidelines:

TABLE 8.2 - CHLORINE CONTROL GUIDELINES

SIZE OF PLANT	TYPE OF RECEIVING WATER	RECOMMENDED CONTROL	METHOD OF CHLORINE RESIDUAL DETERMINATION
LARGE	ALL TYPES	CLOSED-LOOP, FLOW PROPORTIONAL, TWO CHLORINE ANALYZERS	AMPEROMETRIC TITRATOR; CONTINUOUS DETERMINATION AND RECORDING
MEDIUM	ECOLOGICALLY SENSITIVE WATERS WITH FISHING POTENTIAL	SAME AS LARGE PLANTS	SAME AS LARGE PLANTS
	WATERS OF PUBLIC HEALTH IMPORTANCE	CLOSED-LOOP, FLOW PROPORTIONAL, ONE CHLORINE ANALYZER	AMPEROMETRIC TITRATOR; CONTINUOUS DETERMINATION (OPTIONAL RECORDING)
SMALL	ECOLOGICALLY SENSITIVE RECEIVING WATER	CLOSED-LOOP, FLOW PROPORTIONAL, ONE CHLORINE ANALYZER	AMPEROMETRIC TITRATOR; CONTINUOUS DETERMINATION
	RECEIVING WATER OF PUBLIC HEALTH IMPORTANCE	OPEN-LOOP, FLOW PROPORTIONAL	STARCH-IODIDE METHOD ORRTHOTOLIDINE METHOD (INTERMITTENT - MANUAL)

8.3.5 Storage and Handling

8.3.5.1 *Building Layout and Material Handling*

If gas chlorination equipment and chlorine cylinders are to be in a building used for other purposes, a gas-tight partition shall separate this room from any other portion of the building. Doors to the chlorinator room shall open only to the outside of the building and shall be equipped with panic hardware. Such rooms shall be at ground level and should permit easy access to all equipment.

The storage area should be separated from the feed area. A bilingual "DANGER" sign shall be placed on the door and safety precaution instructions to startup and shutdown shall be placed at a visible location on the wall. Full and empty chlorine cylinders shall be stored separately and shall be chained to the wall in the vertical position. Cylinders should not be stored near flammable materials, heating or ventilation units, elevator shafts and on uneven or subsurface floors.

Chlorine cylinders shall be conveyed by a wheeled cart. Suitable lifting bars or hoist shall be used to unload and place the 900 kg containers on their side on level rails.

Storage and handling procedures shall conform to the guidelines of the latest edition of the "Chlorine Manual" prepared by "The Chlorine Institute, Inc."

8.3.5.2 *Special Construction Details*

A clear glass, gas-tight window shall be installed in an exterior door or interior wall of the chlorinator room to permit the chlorinator to be viewed without entering the room. The building shall be of fireproof material.

The distance from any point in the room and the outside door shall not exceed five meters. The chlorinator shall be placed 1.0 m from the outside wall. Twenty-five mm piping shall be used and pipes and valves shall be color coded.

8.3.5.3 *Heat*

The chlorinator room shall be provided with a means of heating so that a temperature of at least 15°C can be maintained, but the room should be protected from excess heat. Cylinders shall be kept at essentially room temperature.

8.3.5.4 *Ventilation*

With chlorination systems, forced, mechanical ventilation shall be installed which will provide one complete air change per minute when the room is occupied. The entrance to the air exhaust duct from the room shall be 300 mm above the floor and the point of discharge shall be so located as not to contaminate the air inlet to any buildings or inhabited areas. The air inlet shall be located near the ceiling on the opposite side of the room so as to provide cross ventilation with air and at such temperatures that will not adversely affect the chlorination equipment. The vent hose from the chlorinator should discharge to an air scrubber before venting above grade to the atmosphere.

8.3.5.5 *Electrical Controls*

The controls for the fans and lights shall be such that they will automatically operate when the door is open. The controls shall also be such that they can be manually operated from the outside without opening the door. All electrical equipment shall be vapor-proof. Fans and lights should be on the same off and on switch whenever possible.

8.3.5.6 *Respiratory Protection*

A self-contained air-supply breathing apparatus in good operating condition shall be available at all installations where chlorine gas is handled. This equipment shall be stored outside of any room where chlorine is used or stored. Instructions for using, testing and replacing parts and air tanks shall be posted. The units shall use compressed air, have at least 30-minute capacity and be compatible with the units used by the fire department responsible for the plant. Other safety equipment required includes a first-aid kit, a fire extinguisher, goggles and gloves, a chlorine container repair kit and a shower.

8.3.6 Piping and Connections

Piping systems should be as simple as possible, specially selected and manufactured to be suitable for chlorine, with a minimum number of joints; piping should be well supported and protected against temperature extremes.

The correct weight or thickness of steel shall be suitable for use with DRY chlorine liquid or gas. Even minute traces of water added to chlorine results in a corrosive attack that can only be resisted by pressure piping utilizing materials such as silver, gold, platinum or Hasteloy "C". Low pressure lines made of hard rubber, saran-lined, rubber-lined, polyethylene, polyvinylchloride (PVC) or Uscolite materials are satisfactory for wet chlorine or aqueous solutions of chlorine.

Due to the corrosiveness of wet chlorine, all lines designed to handle dry chlorine should be protected from the entrance of water or air containing water.

8.3.7 Miscellaneous

8.3.7.1 *Drains*

Floor drainage should be provided near all injector assemblies, all chlorine residual analyzers, all evaporators and certain chlorinators which specifically require floor drains. Floor drains from the chlorine room should not be connected to floor drains from other rooms.

8.3.7.2 *Vents*

All chlorinators shall have a pressure vacuum relief vent system, which should be carried to an air scrubber then to the outside atmosphere, without traps, to a safe area, one vent for each chlorinator. The ends of the vent lines should point down, be covered with a copper wire screen to exclude insects, and should not be more than 7.5 m above the chlorinator. The line should have a slight downward pitch from the high point (directly above the chlorinator) to drain any condensate away from the chlorinator. It is acceptable to run the vent vertically (but no more than 7.5 m) above the chlorinator to the roof, with a 180° return bend at the exit.

Each external chlorine pressure-reducing valve should be checked to see if it is provided with a vent; some are not vented, depending on the chlorine capacity. When supplied, these vents should drain away from the valves. These valves should be located high enough so that the individual drains will have a continuous downgrade to the outside atmosphere.

Evaporators have a steam vapor vent which can be manifolded together and discharged to the atmosphere without traps.

8.3.7.3 *Chlorinator Alarms*

Each chlorinator in large plants shall be equipped with a vacuum switch that should close or open a contact (and start an alarm) when there is an unusually high or low vacuum in the line from the chlorinator to the injector.

Medium size plants are encouraged to include such vacuum switch-alarm systems.

8.3.8 Evaluation of Effectiveness

8.3.8.1 *Sampling*

Facilities shall be included for sampling the disinfected effluent after contact. Either grab or continuous monitoring, as conditions warrant, should be made for effluent chlorine residual of the disinfected effluent.

8.3.8.2 *Testing and Control*

Equipment shall be provided for measuring chlorine residual, employing the standard DPD test. The equipment should enable residual measurement to the nearest 0.1 mg/L in the range below 0.5 mg/L and to an accuracy of approximately 25 percent above 0.5 mg/L. Where the discharge occurs at points requiring rigid bacteriological controls such as on public water supply watersheds, recreational watersheds or shellfish waters or waters tributary thereto, the installation of demonstrated effective facilities for automatic chlorine residual analysis, recording and proportioning systems should be considered. In sensitive areas, dechlorination may be required. Chemicals such as sulphur dioxide, hydrogen peroxide, sodium metabisulfite or granular activated carbon may be used for such purposes.

8.3.9 Hypochlorination

8.3.9.1 *Supply*

Hypochlorite for the purpose of wastewater disinfection is usually in a liquid or solid form. Sodium hypochlorite is available in the liquid form with 12 to 15 percent available chlorine. Calcium hypochlorite is a solid with 70 percent available chlorine and comes in either the 22.5 or 45 kg plastic or plastic-lined containers and in pellet form. The choice of chemical is dependent on quantity used and distance of travel from the supplier.

8.3.9.2 *Storage and Handling*

Special storage facilities are required for handling hypochlorination equipment. Chemicals containing chlorine compound should be stored in a separate room used for that purpose only. No other materials should be stored in the same room. The room should be of fire-resistant construction and at or above grade. As heat and light affect the shelf life of sodium hypochlorite, the storage area should be kept cool and be protected from direct sunlight.

Calcium hypochlorite (HTH) shall be kept dry and covered. The storage area must not be serviced by automatic sprinkler systems. When heated above 170°C, HTH releases oxygen. For this reason, HTH must be kept away from flammable materials. Calcium hypochlorite storage areas should be provided with an exhaust system for the purpose of dust removal.

8.3.9.3 *Metering*

Application of hypochlorite for the purpose of disinfection should be by metering pumps specifically designed for this purpose. The system should be capable of maintaining a chlorine residual of at least 0.5 mg/L after a retention time of 20-30 minutes. Calcium hypochlorite should be initially mixed in a make-up tank prior to any chlorination purpose.

8.4 DECHLORINATION

8.4.1 General

Dechlorination of effluent shall be considered when the receiving water is:

a. considered to be highly important for the fishing industry; or

b. ecologically sensitive to chlorine toxicity and susceptible to the adverse effects of chlorine residuals; or

c. of public health importance.

The decisions regarding use of dechlorination shall be made on a case-by-case basis.

Dechlorination is especially recommended for situations where low coliform densities, as well as chlorine residuals, are jointly required. The most common dechlorination chemicals are sulphur compounds, particularly sulphur dioxide gas or aqueous solutions of sulphite or bisulphate. Pellet dechlorination systems are also available for small facilities. The type of dechlorination system should be carefully selected considering criteria including the following: type of chemical storage required, amount of chemical needed, ease of operation, compatibility with existing equipment, and safety.

8.4.2 Dosage

The dosage of dechlorination chemicals should depend on the residual chlorine in the effluent, the final residual chlorine limit, and the particular form of the dechlorinating chemical used. The most common dechlorinating agent is sulphite. The following forms of the compound are commonly used and yield sulphite (SO_2) when dissolved in water.

Dechlorination Chemical	Theoretical mg/L Required to Neutralize 1 mg/L Cl2
Sulphur dioxide (gas)	0.9
Sodium meta bisulphate (solution)	1.34
Sodium bisulphate (solution)	1.46

Theoretical values may be used for initial approximations, to size feed equipment with the consideration that under good mixing conditions 10% excess dechlorinating chemical is required above theoretical values. Excess sulphur dioxide may consume oxygen at a maximum of 1.0 mg dissolved oxygen for every 4 mg SO_2.

The liquid solutions come in various strengths. The solutions may need to be further diluted to provide the proper dose of sulfite.

8.4.3 Containers

Depending on the chemical selected for dechlorination, the storage containers will vary from gas cylinders, liquid in 190 litre drums or dry compounds. Dilution tanks and mixing tanks will be necessary when using dry compounds and may be necessary when using liquid compounds to deliver the proper dosage. Solution containers should be covered to prevent evaporation and spills.

8.4.4 Feed Equipment, Mixing, and Contact Requirements

8.4.4.1 *Equipment*

In general, the same type of feeding equipment used for chlorine gas may be used with minor modifications for sulphur dioxide gas. However, the manufacturer should be contacted for specific equipment recommendations. No equipment should be alternately used for the two gases. The common type of dechlorination feed equipment utilizing sulphur compounds include vacuum solution feed of sulphur dioxide gas and a positive displacement pump for aqueous solutions of sulphite or bisulphate.

The selection of the type of feed equipment utilizing sulphur compounds shall include consideration of the operator safety and overall public safety relative to the wastewater treatment plant's proximity to populated areas and the security of gas cylinder storage. The selection and design of sulphur dioxide feeding equipment shall take into account that the gas reliquifies quite easily. Special precautions must be taken when using ton containers to prevent reliquefaction.

Where necessary to meet the operating ranges, multiple units shall be provided for adequate peak capacity and to provide a sufficiently low feed rate on turn down to avoid depletion of the dissolved oxygen concentrations in the receiving waters.

8.4.4.2 *Mixing Requirements*

The dechlorination reaction with free or combined chlorine will generally occur within 15-20 seconds. Mechanical mixers are required unless the mixing facility will provide the required hydraulic turbulence to assure thorough and complete mixing. The high solubility of SO_2 prevents it from escaping during turbulence.

8.4.4.3 *Contact Time*

A minimum of 30 seconds for mixing and contact time shall be provided at the design peak hourly flow or maximum pumping rate. A suitable sampling point shall be provided downstream of the contract zone. Consideration shall be given to a means of reaeration to assure maintenance of an acceptable dissolved oxygen concentration in the stream following sulfonation.

8.4.4.4 *Standby Equipment and Spare Parts*

The same requirements apply as for chlorination systems.

8.4.4.5 *Sulphonator Water Supply*

The same requirements apply as for chlorination systems.

8.4.5 Housing Requirements

8.4.5.1 *Feed and Storage Rooms*

The requirements for housing SO_2 gas equipment should follow the same guidelines as used for chlorine gas.

When using solutions of the dechlorinating compounds, the solutions may be stored in a room that meets the safety and handling requirements set forth in Section 3.8. The mixing, storage, and solution delivery areas must be designed to contain or route solution spillage or leakage away from traffic areas to an appropriate containment unit.

8.4.5.2 *Protective and Respiratory Gear*

The respiratory air-pac protection equipment is the same as for chlorine, (See Section 4.8). Leak repair kits of the type used for chlorine gas that are equipped with gasket material suitable for service with sulphur dioxide gas may be used. For additional safety considerations, See Section 4.8.

8.4.6 Sampling and Control

8.4.6.1 *Sampling*

Facilities shall be included for sampling the dechlorinated effluent for residual chlorine. Provisions shall be made to monitor for dissolved oxygen concentration after sulphonation when required by the regulatory agency.

8.4.6.2 *Testing Control*

Provision shall be made for manual or automatic control of sulphonator feed rates based on chlorine residual measurement or flow.

8.4.7 Activated Carbon

Granular activated carbon may also be used to dechlorinate wastewater effluent. The dechlorination reaction is dependent on the chemical state of the free chlorine, chlorine concentration and flow rate, physical characteristics of the carbon, and the wastewater characteristics.

Dechlorination usually is accomplished in fixed downflow beds using gravity or pressure type filters. Regular backwashing is necessary to preserve dechlorination efficiency.

Suggested design criteria for a wastewater dechlorination activated carbon system, based on potable water application, include a wastewater application rate of 2 L/m^2s, an empty bed contact time of 15 to 20 min with an influent free residual of 3 to 4 mg/L, and an effective carbon bed life of at least 3 years.

8.5 ULTRAVIOLET (UV) DISINFECTION

The following sections describe factors that affect the performance of Ultraviolet Disinfection Systems. Systems should be designed to account for these effects.

8.5.1 UV Transmission

UV light's ability to penetrate wastewater is measured with a spectrophotometer using the same wavelength (254 nm) that is produced by germicidal lamps. This measurement is called the percent Transmission or Absorbance and it is a function of all the factors which absorb or reflect UV light. As the percent transmission gets lower (higher absorbance) the ability of the UV light to penetrate the wastewater and reach target organisms decreases. The system designer must obtain samples of the wastewater during the worst conditions or carefully attempt to calculate the minimum expected UV transmission by testing wastewater from plants which have a similar influent and treatment process. The designer must also strictly define the disinfection limits since they determine the magnitude of the UV dose required.

8.5.2 Wastewater Suspended Solids

Some of the suspended solids in wastewater will absorb or reflect the UV light before it can penetrate the solids to kill any occluded organisms. UV light can penetrate into suspended solids with longer contact times and higher intensities, but there is still a limit to the ability to kill the microorganisms. UV systems must be designed based on maximum effluent SS levels.

8.5.3 Design Flow Rate and Hydraulics

The number of microorganisms that are inactivated within a UV reactor is a function of the multiplication of the average intensity and residence time. That is, the UV Dose (D) is equal to the intensity (I) times the Retention Time (t).

$$D = It$$

As the flow rate increases the number or size of the UV lamps must be proportionately increased to maintain the same disinfection requirements. An Ultraviolet Disinfection system must be designed for worst case conditions. The minimum dosage occurs at the maximum flow rate and end of lamp life.

8.5.4 Level Control

The height of the wastewater above the top row of UV lamps must be rigidly controlled by a flap gate or weir for all flow rates. The UV system must be designed for the maximum flow rate. This is especially important if the wastewater treatment plant receives storm water runoff. The UV system must also be designed to operate at the maximum flow rate. During low flow periods, the wastewater has a greater chance to warm up around the quartz sleeves and produce deposits on the sleeves. There is also the possibility of exposing the quartz sleeves to the air. Because the lamps are warm, any compounds left on the sleeves will bake onto them. Water splashing onto these exposed sleeves will also result in UV absorbing deposits. When the flow returns to normal, some of the water passing through the UV unit may not be properly disinfected. The designer must be very careful with the selection of the flow control device for the above situation. Both flow gates and weirs may be used for level control.

8.5.5 Wastewater Iron Content

Iron can affect the UV disinfection by absorbing UV light. Dissolved iron, iron precipitate on quartz sleeves, and adsorption of iron by suspended solids, bacterial floc and other organic compounds, all decrease UV transmittance. Wastewater with iron levels greater than 0.3 mg/L may require pretreatment to attain the desired disinfection level.

8.5.6 Wastewater Hardness

Calcium and magnesium salts, which are generally present in water as bicarbonates or sulfates, cause water hardness. Hard water will precipitate on any warm or hot surface. Since the optimum operating temperature of the low pressure mercury lamp is 40^0C, the surface of the protective quartz sleeve will be warm. It will create a molecular layer of warm water where calcium and magnesium salts can be precipitated. These precipitates will prevent some of the UV light from entering the wastewater.

Waters which approach or are above 300 mg/L of hardness may require pilot testing of a UV system. This is especially important if very low flows or no flow situations are expected, because they allow the water to warm up around the quartz sleeves and produce excessive coating.

8.5.7 Wastewater Sources

Periodic influxes of industrial wastewater may contain UV absorbing organic compounds, iron or hardness, any of which may affect UV performance. Industries discharging wastes that contain such materials may be required to pre-treat their wastewater.

Low concentrations of dye may be too diluted to be detected without using a spectrophotometer. Dye can readily absorb ultraviolet light thereby preventing UV disinfection.

8.5.8 UV Lamp Life

Low pressure mercury lamps are rated for 9,000 hours of continuous use. Rated average useful life is defined by the UV disinfection industry as the elapsed operating time under essentially continuous operation for the output to decline to 60 percent of the output the lamp had at 100 hours. The UV system must be designed so that the minimum required dose or intensity is available at the end of lamp life.

Power costs and lamp replacement costs are the two main factors affecting UV maintenance expenditures. Therefore, UV lamps should only be replaced if no other cause for not meeting the disinfection requirements can be found. Examples of other causes are quartz sleeve fouling, decreased levels of UV transmission, or increased levels of suspended solids in the wastewater.

8.5.9 UV System Configuration and Redundancy

Once the number of lamps required to meet the required disinfection permit levels has been determined, a system configuration must be developed. This configuration must meet operational requirements such as plant flow variations and redundancy requirements. Redundancy helps insure that the UV system can continue to operate and meet disinfection permits in spite of a subsystem or component failure. It allows regularly scheduled maintenance such as quartz cleaning to be performed at any time.

8.5.10 Detailed Design Manuals

The following sources contain detailed design information for UV Disinfection:

Water Pollution Control Federation: *Wastewater Disinfection*, Manual of Practice FD-10, Alexandria, VA, 1986.

U.S. Environmental Protection Agency: *Design Manual for Municipal Wastewater Disinfection*, EPA 625/ 1-86-021, Cincinnati, OH, 1987.

8.6 OZONATION

8.6.1 Ozone Generation

Ozone may be produced from either an air or an oxygen gas source. Generation units shall be automatically controlled to adjust ozone production to meet disinfection requirements.

8.6.2 Dosage

The ozone demand in the wastewater must be satisfied, as evidenced by the presence of an ozone residual, before significant disinfection takes place. Below this dosage there is reduction of oxygen-consuming material.

Because of the form of ozone and its short life, it is necessary that it be step-fed into the wastewater to provide the contact period needed to accomplish disinfection.

Effectiveness of ozone as a disinfectant is relatively independent of pH and temperature values, although a pH of 6.0 to 7.0 appears to be the most favourable range. A dosage of 5 to 8 mg/L is needed to accomplish disinfection of secondary effluent. The amount and characteristics of suspended solids present in the secondary effluent can be used to determine ozone dosage empirically:

$$\text{Ozone Dosage} = 1.5 + 0.38\ \text{TSS}$$

8.6.3 Design Considerations

8.6.3.1 *Feed Equipment*

Ozone dissolution is accomplished through the use of conventional gas diffusion equipment, with appropriate consideration of materials. If ozone is being produced from air, gas preparation equipment (driers, filters, compressors) is required. If ozone is being produced from oxygen, this equipment may not be needed as a clean dry pressurized gas supply will be available.

Where ozone capacities of 500 kg/d or less are required, air feed is preferred. Modification of the single-pass air feed system should be considered in determining the most economic system for application in wastewater treatment.

8.6.3.2 *Air Cleaning*

Removal of foreign matter such as dirt and dust is essential for optimum performance and life of an ozone device. For small units, cartridge-type impingement filters may be economical. For larger operations, electrical precipitator or combination filters are preferred.

8.6.3.3 *Compression*

Positive displacement rotary-type compressors are preferred for large installations. Internally lubricated units should not be used since oil vapor will permanently impair the water-adsorptive capacity of the driers. Need for standby capacity and flexibility of operation requires the installation of several blower units.

The required compressor rating will depend on the pressure drop through the entire system. Generally, a 70 kN/m^2 pressure is necessary to force the air through the coolers, driers, ozonation devices, and the 4.5 to 6 m head of water in the mixing and contact system.

8.6.3.4 *Cooling and Drying*

Pre-treatment for reducing moisture in the feed gas stream shall be required.

8.6.3.5 *Injection, Mixing and Contact*

Intimate mixing of an ozone-enriched air stream with the wastewater as well as maintaining contact for an adequate period of time are essential. The major problems to be considered are satisfying the ozone demand, the rapid rise of the gas to the liquid surface of the contact chamber and escape of ozonated air bubbles, and the relatively short half-life of ozone. Consequently, where ozone contact beyond a few minutes is needed, the ozonated feed stream is staged with the amount of ozone for each stage set at a level that can be consumed usefully.

8.6.3.6 *Controls*

The design engineer should be cognizant of the fact that ozone is a toxic gas, and that if compressed oxygen is used as the feed gas, special provisions must be met in its handling and storage. The ozonation process involves a series of mechanical and electrical units that require appropriate maintenance and repair and are susceptible to the same malfunctions as are all such pieces of equipment. Standby capacity normally is provided in all essential components. Information can be obtained from the equipment manufacturer on the metering and alarm systems needed for continuous process monitoring and warning of failure in any element of the process.

8.6.3.7 *Piping and Connections*

For ozonation systems, the selection of material should be made with due consideration for ozone's corrosive nature. Copper or aluminum alloys should be avoided. Only material at least as corrosion-resistant to ozone as Grade 304L stainless steel should be specified for piping containing ozone in non-submerged applications. Unplasticized PVC, Type I, may be used in submerged piping, provided the gas temperature is below 60°C and the gas pressure is low.

Chapter 9 NUTRIENT REMOVAL & TERTIARY TREATMENT

9.1 PHOSPHORUS REMOVAL

9.1.1 General

9.1.1.1 *Applicability*

The following factors should be considered when determining the need for phosphorus control at municipal wastewater treatment facilities:

a. the present and future phosphorus loadings from the existing municipal wastewater treatment facility to the receiving water;

b. the existing phosphorus levels in the receiving water and the effects of these levels on the rate of eutrophication along the entire length of receiving waters;

c. the predicted response of the receiving water to increased phosphorus loadings;

d. the existing and desired water quality of the receiving water along its entire length;

e. the existing and projected uses of the receiving water; and

f. consideration of the best practicable technology available to control phosphorus discharges.

9.1.1.2 *Phosphorus Removal Criteria*

A municipal wastewater treatment facility shall be required to control the discharge of phosphorus if the following conditions exist:

a. Eutrophication of the receiving water environment is either occurring or may occur at a rate which may affect the existing and potential uses of the water environment; or

b. The municipal wastewater effluent discharge is contributing or may contribute significantly to the rate of receiving water eutrophication.

9.1.1.3 *Method of Removal*

Acceptable methods for phosphorus removal shall include chemical precipitation, high rate filtration or biological processes.

9.1.1.4 *Design Basis*

9.1.1.4.1 Preliminary Testing

Laboratory, pilot or full scale studies of various chemical feed systems and treatment processes are recommended for existing plant facilities to determine the achievable performance level, cost-effective design criteria, and ranges of required chemical dosages.

The selection of a treatment process and chemical dosage for a new facility should be based on such factors as influent wastewater characteristics, effluent requirements, and anticipated treatment efficiency.

9.1.1.4.2 System Flexibility

Systems shall be designed with sufficient flexibility to allow for several operational adjustments in chemical feed location, chemical feed rates, and for feeding alternate chemical compounds.

9.1.2 Effluent Requirements

If phosphorus control is required, the maximum acceptable concentration of final effluent phosphorus and/or the maximum acceptable mass loading to the receiving stream shall be established on a site specific basis.

9.1.3 Process Requirements

9.1.3.1 *Dosage*

Typical chemical dosage requirements of various chemicals required for phosphorus removal are outlined in Table 9.1.

Dosages will vary with the phosphorus concentration in the effluent. The required chemical dosage shall include the amount needed to react with the phosphorus in the wastewater, the amount required to drive the chemical reaction to the desired state of completion, and the amount required due to inefficiencies in mixing or dispersion. Excessive chemical dosage should be avoided.

9.1.3.2 *Chemical Selection*

The choice of lime or the salts of aluminum or iron should be based on the wastewater characteristics and the economics of the total system.

When lime is used it may be necessary to neutralize the high pH prior to subsequent treatment in secondary biological systems or prior to discharge in those flow schemes where lime treatment is the final step in the treatment process.

TABLE 9.1 - TYPICAL CHEMICAL DOSAGE REQUIREMENTS FOR PHOSPHORUS REMOVAL

TYPE OF TREATMENT	ADDITION POINT	DOSAGE RATE (mg/L)		
PLANT		CHEMICAL	RANGE	AVERAGE
MECHANICAL				
PRIMARY	RAW SEWAGE	ALUM FERRIC CHLORIDE LIME	100 6-30 167-200	100 16 185
SECONDARY	RAW SEWAGE	LIME ALUM FERRIC CHLORIDE	40-100	70
	SECONDARY SECTION	LIME ALUM FERRIC CHLORIDE	30-150 2-30	65 11
WASTE STABILIZATION PONDS				
SEASONAL RETENTION LAGOONS	BATCH DOSAGE TO CELLS	ALUM FERRIC CHLORIDE LIME	100-210 17-22 250-350	163 20 300
CONTINUOUS DISCHARGE LAGOON	RAW SEWAGE	ALUM FERRIC CHLORIDE LIME	225 20 400	225 20 400

9.1.3.3 *Chemical Feed System*

In designing the chemical feed system for phosphorus removal, the following points should be considered:

a. the need to select chemical feed pumps, storage tanks and piping suitable for use with the chosen chemicals(s);

b. selection of chemical feed equipment with the required range in capacity;

c. the need for a standby chemical feed pump;

d. provision of flow pacing for chemical pumps proportional to sewage flow rates;

e. flexibility by providing a number of chemical application points;

f. the need for protection of storage and piping from the effect of low temperatures;

g. selection of the proper chemical storage volume;

h. the need for ventilation in chemical handling rooms; and

i. provision for containment of any chemical spills.

9.1.3.4 *Chemical Feed Points*

Selection of chemical feed points shall include consideration of the chemicals used in the process, necessary reaction times between chemical and polyelectrolyte additions, and the wastewater treatment processes and components utilized. Considerable flexibility in feed location should be provided, and multiple feed points are recommended.

9.1.3.5 *Flash Mixing*

Each chemical must be mixed rapidly and uniformly with the flow stream. Where separate mixing basins are provided, they should be equipped with mechanical mixing devices. The detention period should be at least 30 seconds.

9.1.3.6 *Flocculation*

The particle size of the precipitate formed by chemical treatment may be very small. Consideration should be given in the process design to the addition of synthetic polyelectrolytes to aid settling. The flocculation equipment should be adjustable in order to obtain optimum floc growth, control deposition of solids, and prevent floc destruction.

9.1.3.7 *Liquid - Solids Separation*

The velocity through pipes or conduits from flocculation basins to settling basins should not exceed 0.5 m/s in order to minimize floc destruction. Entrance works to settling basins should also be designed to minimize floc shear.

Settling basin design shall be in accordance with criteria outlined in Chapter 6. For design of the sludge handling system, special consideration should be given to the type and volume of sludge generated in the phosphorus removal process.

9.1.3.8 *Filtration*

Effluent filtration shall be considered where effluent phosphorus concentrations of less than 1 mg/L must be achieved.

9.1.4 Feed Systems

9.1.4.1 *Location*

All liquid chemical mixing and feed installations should be installed on corrosion resistant pedestals and elevated above the highest liquid level anticipated during emergency conditions.

Lime feed equipment should be located so as to minimize the length of slurry conduits. All slurry conduits shall be accessible for cleaning.

9.1.4.2 *Liquid Chemical Feed System*

Liquid chemical feed pumps should be of the positive displacement type with variable feed rate. Pumps shall be selected to feed the full range of chemical quantities required for the phosphorus mass loading conditions anticipated with the largest unit out of service.

Screens and valves shall be provided on the chemical feed pump suction lines.

An air break or anti-siphon device shall be provided where the chemical solution stream discharges to the transport water stream to prevent an induction effect resulting in overfeed.

Consideration shall be given to providing pacing equipment to optimize chemical feed rates.

9.1.4.3 ***Dry Chemical Feed System***

Each dry chemical feeder shall be equipped with a dissolver which is capable of providing a minimum 5-minute retention at the maximum feed rate.

Polyelectrolyte feed installations should be equipped with two solution vessels and transfer piping for solution make-up and daily operation.

Make-up tanks shall be provided with an educator funnel or other appropriate arrangement for wetting the polymer during the preparation of the stock feed solution. Adequate mixing should be provided by a large-diameter low-speed mixer.

9.1.5 **Storage Facilities**

9.1.5.1 ***Size***

Storage facilities shall be sufficient to insure that an adequate supply of the chemical is available at all times. The exact size required will depend on the size of the shipment, length of delivery time, and process requirements. Storage for a minimum of 10-days supply should be provided.

9.1.5.2 ***Location***

The liquid chemical storage tanks and tank fill connections shall be located within a containment structure having a capacity exceeding the total volume of all storage vessels. Valves on discharge lines shall be located adjacent to the storage tank and within the containment structure.

Auxiliary facilities, including pumps and controls, within the containment area shall be located above the highest anticipated liquid level. Containment areas shall be sloped to a sump area and shall not contain floor drains.

Bag storage should be located near the solution make-up point to avoid unnecessary transportation and housekeeping problems.

9.1.5.3 ***Accessories***

Platforms, ladders, and railings should be provided as necessary to afford convenient and safe access to all filling connections, storage tank entries, and measuring devices.

Storage tanks shall have reasonable access provided to facilitate cleaning.

9.1.6 Other Requirements

9.1.6.1 *Materials*

All chemical feed equipment and storage facilities shall be constructed of materials resistant to chemical attack by all chemicals normally used for phosphorus treatment.

9.1.6.2 *Temperature, Humidity and Dust Control*

Precautions shall be taken to prevent chemical storage tanks and feed lines from reaching temperatures likely to result in freezing or chemical crystallization at the concentrations employed. A heated enclosure or insulation may be required. Consideration should be given to temperature, humidity and dust control in all chemical feed room areas.

9.1.6.3 *Cleaning*

Consideration shall be given to the accessibility of piping. Piping should be installed with plugged wyes, tees or crosses at changes in direction to facilitate cleaning.

9.1.6.4 *Drains and Drawoff*

Above-bottom drawoff from chemical storage or feed tanks shall be provided to avoid withdrawal of settled solids into the feed system. A bottom drain shall also be installed for periodic removal of accumulated settled solids. Provisions shall be made in the fill lines to prevent back siphonage of chemical tank contents.

9.1.7 Hazardous Chemical Handling

The requirements of Section 4.8.2 Hazardous Chemical Handling shall be met.

9.1.8 Sludge Handling

9.1.8.1 *General*

Consideration shall be given to the type and additional capacity of the sludge handling facilities needed when chemicals are added.

9.1.8.2 *Dewatering*

Design of dewatering systems should be based, where possible, on an analysis of the characteristics of the sludge to be handled. Consideration should be given to the ease of operation, effect of recycle streams generated, production rate, moisture content, dewaterability, final disposal, and operating cost.

9.2 AMMONIA REMOVAL

9.2.1 Breakpoint Chlorination

9.2.1.1 *Applicability*

The breakpoint chlorination process is best suited for removing relatively small quantities of ammonia, less than 5 mg/L NH_3-N, and in situations whose low residuals of ammonia or total nitrogen are required.

9.2.1.2 *Design Considerations*

9.2.1.2.1 *Mixing*

The reaction between ammonia and chlorine occurs instantaneously, and no special design features are necessary except to provide for complete uniform mixing of the chlorine with the wastewater. Good mixing can best be accomplished with in-line mixers or backmixed reactors. A minimum contact time of 10 min is recommended.

9.2.1.2.2 *Dosage*

The sizing of the chlorine producing and/or feed device is dependent on the influent ammonia concentration to be treated as well as the degree of pretreatment the wastewater has received. As the level of wastewater pretreatment increases, the required amount of chlorine decreases and approaches the theoretical amount required to oxidize ammonia to nitrogen (7.6 mg/L C/2:1 mg/L NH_3-N). Table 9.2 shows the quantities of chlorine required, based on operating experience as well as recommended design capabilities. These ratios are applied to the maximum anticipated influent ammonia concentration.

TABLE 9.2 - QUANTITIES OF CHLORINE REQUIRED FOR THREE WASTEWATER SOURCES

WASTEWATER SOURCE	CHLORINE: NH_3 - N RATIO TO REACH BREAKPOINT	
	EXPERIENCE	RECOMMENDED DESIGN CAPABILITY
RAW	10:1	13:1
SECONDARY EFFLUENT	9:1	12:1
LIME SETTLED AND FILTERED SECONDARY EFFLUENT	8:1	10:1

9.2.1.2.3 *Monitoring*

If insufficient chlorine is available to reach the breakpoint, no nitrogen will be formed and the chloramines formed ultimately will revert back to ammonia. Provisions should be made to continuously monitor the waste, following chlorine addition, for free chlorine residual and to pace the chlorine feed device to maintain a set-point free chlorine residual.

9.2.1.2.4 *Standby Equipment*

The chemical feed assembly used for ammonia removal by breakpoint chlorination is considered in the preliminary design of the complete chlorination system, including those requirements for prechlorination, intermediate, and post-chlorination applications. Depending on the use of continuous chlorination at points within the system, some consideration is given to the use of standby chlorination equipment for the ammonia removal system. Reliability needs and maximum dosage requirements for the various application points shall also be examined when sizing the equipment.

9.2.1.2.5 pH Adjustment

Except for wastewaters having a high alkalinity or treatment systems employing lime coagulation prior to chlorination, provisions shall be made to feed an alkaline chemical to keep the pH of the wastewater in the proper range. A method for measuring and pacing the alkaline chemical feed pump to keep the pH in the desired range also should be provided.

9.2.2 Air Stripping

9.2.2.1 *Applicability*

The ammonia air stripping process is most economical if it is preceded by lime coagulation and settling. The ammonia stripping process can be used in a treatment system employing biological treatment or in a physical-chemical process. In most instances, more than 90 percent of the nitrogen in raw domestic wastewater is in the form of ammonia, and the ammonia stripping process can be readily applied to most physical-chemical treatment systems. However, when the ammonia stripping process is to be preceded by a biological process, care must be exercised to insure that nitrification does not occur in the secondary treatment process.

There is one serious limitation of the ammonia stripping process that should be recognized; namely, it is impossible to operate a stripping tower at air temperatures less than 0°C because of freezing within the tower. For treatment plants in cold weather locations, high pH stripping ponds may provide a simple solution to the problem of nitrogen removal.

9.2.2.2 *Design Considerations*

9.2.2.2.1 Tower Packing

Packings used in ammonia stripping towers may include 10 by 40 mm wood slats, plastic pipe, and a polypropylene grid. No specific packing spacing has been established. Generally, the individual splash should be spaced 40 to 100 mm horizontally and 50 to 100 mm vertically. A tighter spacing is used to achieve higher levels of ammonia removal and a more opening spacing is used where lower levels of ammonia removal are acceptable. Because of the large volume of air required, towers should be designed for a total air headloss of less than 50 to 75 mm of water. Packing depths of 6 to 7.5 m should be used to minimize power costs.

9.2.2.2.2 Hydraulic Loadings

Allowable hydraulic loading is dependent on the type and spacing of the individual splash bars. Although hydraulic loading rates used in ammonia stripping towers should range from 0.7 to 2.0 L/m^2.s removal efficiency is significantly decreased at loadings in excess of 1.3 L/m^2.s. The hydraulic loading rate should be such that a water droplet is formed at each individual splash bar as the liquid passes through the tower.

9.2.2.2.3 Air Requirements

Air requirements vary from 2200 to 3800 L/s for each L/s being treated in the tower. The 6 to 7.5 m of tower packing will normally produce a pressure drop of

15 to 40 mm of water.

9.2.2.2.4 Temperature

Air and liquid temperatures have a significant effect on the design of an ammonia stripping tower. Minimum operating air temperature and associated air density should be considered when sizing the fans to meet the desired air supply. Liquid temperature also affects the level of ammonia removal.

9.2.2.2.5 General Construction Features

The stripping tower may be either of the countercurrent (air inlet at base) or cross flow (air inlet along entire depth of fill) type. Generally, provisions should be made to have the capability to recycle tower effluent to increase the removal of ammonia nitrogen during cooler temperatures.

Provisions shall be made in the design of the tower structure and fill so that the tower packing is readily accessible or removable for removing possible deposits of calcium carbonate.

9.2.2.2.6 Process Control

During periods of tower operation when temperature, air and wastewater flow rates, and scale formation are under control, the major process requirement necessary to insure satisfactory ammonia removal is to control the influent pH. pH control should be practiced in the upstream lime-coagulation-settling process. This basin should be monitored closely to prevent excessive carryover of lime solids into the ammonia stripping process. Normal lime-addition required to raise the pH to 11.5 is 300 to 400 mg/L (as CaO).

9.3 BIOLOGICAL NUTRIENT REMOVAL

9.3.1 Biological Phosphorus Removal

A number of biological phosphorus removal processes exist that have been developed as alternatives to chemical treatment. Phosphorus is removed in biological treatment by means of incorporating orthophosphate, polyphosphate, and organically bound phosphorus into cell tissue. The key to the biological phosphorus removal is the exposure of the microorganisms to alternating anaerobic and aerobic conditions. Exposure to alternating conditions stresses the microorganisms so that their uptake of phosphorus is above normal levels. Phosphorus is not only used for cell maintenance, synthesis, and energy transport but is also stored for subsequent use by the microorganisms. The sludge containing the excess phosphorus is either wasted or removed through a sidestream to release the excess. The alternating exposure to anaerobic and aerobic conditions can be accomplished in the main biological treatment process, or "mainstream," or in the return sludge stream, or "sidestream."

9.3.1.1.1 Mainstream Phosphorus Removal (A/O Process)

The proprietary A/O process is a single sludge suspended-growth system that combines anaerobic and aerobic sections in sequence. Settled sludge is returned to the influent end of the reactor and mixed with the incoming wastewater. Under anaerobic conditions, the phosphorus contained in the wastewater and the recycled cell mass is released as soluble phosphates. Some BOD reduction also occurs in this stage. The phosphorus is then taken up by the cell mass in the aerobic zone. Phosphorus is removed from the liquid stream in the waste activated sludge. The concentration of phosphorus in the effluent is dependent mainly on the ratio of BOD to phosphorus of the wastewater treated.

9.3.1.2 *Sidestream Phosphorus Removal (PhoStrip Process)*

In the proprietary phostrip process a portion of the return activated sludge from the biological treatment process is diverted to an anaerobic phosphorus stripping tank. The retention time in the stripping tank typically ranges from 8 to 12 hours. The phosphorus released in the stripping tank passes out of the tank in the supernatant, and the phosphorus-poor activated sludge is returned to the aeration tank. The phosphorus-rich supernatant is treated with lime or another coagulant in a separate tank and discharged to the primary sedimentation tanks or to a separate flocculation/clarification tank for solids separation. Phosphorus is removed from the system in the chemical precipitant. Conservatively designed PhoStrip and associated activated-sludge systems are capable of consistently producing an effluent with a total phosphorus content of less than 1.5 mg/L before filtration.

9.3.1.3 *Design Criteria*

TABLE 9.3 - DESIGN CRITERIA FOR BIOLOGICAL PHOSPHORUS REMOVAL

DESIGN PARAMETER	TREATMENT PROCESS		
	A/O	PhoStrip	SBR
Food/Microorganism Ratio (kg BOD_5/kg MLVSS.d)	0.2 - 0.7	0.1 - 0.5	0.15 - 0.5
Solids Retention Time (d)	2 - 25	10 - 30	
MLSS (mg/L)	2000 - 4000	600 - 5000	2000 - 3000
Hydraulic Retention Time (hrs)			
Anaerobic Zone	0.5 - 1.5	8 - 12	1.8 - 3
Aerobic Zone	1 - 3	4 - 10	1.0 - 4
Return Activated Sludge (% of Influent Flowrate)	25 - 40	20 - 50	N/A
Stripper Underflow (% of Influent Flowrate)	N/A	10 - 20	N/A

9.3.2 Biological Nitrogen Removal

The principal nitrogen conversion and removal processes are conversion of ammonia nitrogen to nitrate by biological nitrification and removal of nitrogen by biological nitrification/denitrification.

9.3.2.1 *Nitrification*

Biological nitrification consists of the conversion of ammonia nitrogen to nitrite followed by the conversion of nitrite to nitrate. This process does not increase the removal of nitrogen from the waste stream over that achieved by conventional biological treatment. The principal effect is that nitrified effluent can be denitrified biologically. To achieve nitrification, all that is required is the maintenance of conditions suitable for the growth of nitrifying organisms.

Nitrification is also used when treatment requirements call for oxidation of ammonia-nitrogen. Nitrification may be carried out in conjunction with secondary treatment or in a tertiary stage. In each case, either suspended growth or attached growth reactors can be used.

9.3.2.1.1 Design Criteria

TABLE 9.4 - DESIGN CRITERIA FOR NITRIFICATION		
DESIGN PARAMETER	SINGLE STAGE	SEPARATE STAGE
Food/Microorganism Ratio (kg BOD_5/kg MLVSS.d)	012 - 0.25	0.05 - 0.2
Solids Retention Time (d)	8 - 20	15 - 100
MLSS (mg/L)	1500 - 3500	1500 - 3500
Hydraulic Retention Time (hrs)	6 - 15	3 - 6
Return Activated Sludge (% of Influent Flowrate)	50 - 150	50 - 200

9.3.2.2 *Combined Nitrification/Denitrification*

The removal of nitrogen by biological nitrification/denitrification is a two step process. In the first step, ammonia is converted aerobically to nitrate (NO3-) (nitrification). In the second step, nitrates are converted to nitrogen gas (denitrification).

The removal of nitrate by conversion to nitrogen gas can be accomplished biologically under anoxic conditions. The carbon requirements may be provided by internal sources, such as wastewater and cell material, or by an external source.

9.3.2.2.1 Bardenpho Process (Four-Stage)

The four-stage proprietary Bardenpho process uses both the carbon in the untreated wastewater and carbon from endogenous decay to achieve denitrification. Separate reaction zones are used for carbon oxidation and anoxic denitrification. The wastewater initially enters an anoxic denitrification zone to which nitrified mixed liquor is recycled from a subsequent combined carbon oxidation nitrification compartment. The carbon present in the wastewater is used to denitrify the recycled nitrate. Because the organic loading is high, denitrification proceeds rapidly. The ammonia in the wastewater passes unchanged through the first anoxic basin to be nitrified in the first aeration basin. The nitrified mixed liquor from the first aeration basin passes into a second anoxic zone, where additional denitrification occurs using the endogenous carbon source. The second aerobic zone is relatively small and is used mainly to strip entrained nitrogen gas prior to clarification. Ammonia released from the sludge in the second anoxic zone is also nitrified in the last aerobic zone.

9.3.2.2.2 Oxidation Ditch

In an oxidation ditch, mixed liquor flows around a loop-type channel, driven and aerated by mechanical aeration devices. For nitrification/denitrification applications, an aerobic zone is established immediately downstream of the aerator, and an anoxic zone is created upstream of the aerator. By discharging the influent wastewater stream at the upstream end of the anoxic zone, some of the wastewater carbon source is used for denitrification. The effluent from the reactor is taken from the end of the aerobic zone for clarification. Because the system has only one anoxic zone, nitrogen removals are lower than those of the Bardenpho process.

9.3.3 Combined Biological Nitrogen and Phosphorus Removal

A number of biological processes have been developed for the combined removal of nitrogen and phosphorus. Many of these are proprietary and use a form of the activated sludge process but employ combinations of anaerobic, anoxic, and aerobic zones or compartments to accomplish nitrogen and phosphorus removal.

9.3.3.1 *A^2/O Process*

The proprietary A^2/O process provides an anoxic zone for denitrification with a detention period of approximately one hour. The anoxic zone is deficient in dissolved oxygen, but chemically bound oxygen ion the form of nitrate or nitrite is introduced by recycling nitrified mixed liquor from the aerobic section. Effluent phosphorus concentrations of less than 2 mg/L can be expected without effluent filtration; with effluent filtration, effluent phosphorus concentrations may be less than 1.5 mg/L.

9.3.3.2 *Bardenpho Process (5 Stage)*

The proprietary Bardenpho process can be modified for combined nitrogen and phosphorus removal. The Phoredox modification of the bardenpho process incorporates a fifth (anaerobic) stage for phosphorus removal. The five-stage system provides anaerobic, anoxic, and aerobic stages for phosphorus, nitrogen, and carbon removal. A second anoxic stage is provided for additional denitrification using nitrate produced in the aerobic stage as the electron acceptor and the endogenous organic carbon as the electron donor. The final aerobic stage is used to strip residual nitrogen gas from solution and to minimize the release of phosphorus in the final clarifier. Mixed liquor from the first aerobic zone is recycled to the anoxic zone.

9.3.3.3 *UCT Process*

The UCT process eliminates return activated sludge to the anoxic stage and the internal recycle is from the anoxic stage to the anaerobic stage. By returning the activated sludge to the anoxic stage, the introduction of nitrate to the anaerobic stage is eliminated, thereby improving the release of phosphorus in the anaerobic stage. The internal recycle feature provides for increased organic utilization in the anaerobic stage. The mixed liquor from the anoxic stage contains substantial soluble BOD but little nitrate. The recycle of the anoxic mixed liquor provides for optimal conditions for fermentation uptake in the anaerobic stage.

9.3.3.4 *Design Criteria*

TABLE 9.5 - DESIGN CRITERIA FOR COMBINED BIOLOGICAL NITROGEN AND PHOSPHORUS REMOVAL

DESIGN PARAMETER	TREATMENT PROCESS			
	A^2/O	Bardenpho (5 Stage)	UCT	SBR
Food/Microorganism Ratio (kg BOD_5/kg MLVSS.d)	0.15 - 0.25	0.1 - 0.2	0.1 - 0.2	0.1
Solids Retention Time (d)	4 - 27	10 - 40	10 - 30	---
MLSS (mg/L)	3000 - 5000	2000 - 4000	2000 - 4000	600 - 5000
Hydraulic Retention Time (hrs)				Batch Times
Anaerobic Zone	0.5 - 1.5	1 - 2	1 - 2	0 - 3
Anoxic Zone - 1	0.5 - 1.0	2 - 4	2 - 4	0 - 1.6
Aerobic Zone - 1	3.5 - 6.0	4 - 12	4 - 12	0.5 - 1
Anoxic Zone - 2		2 - 4	2 - 4	0 - 0.3
Aerobic Zone - 2		0.5 - 1		0 - 0.3
Settle/Decant				1.5 - 2
Total	4.5 - 8.5	9.5 - 23	9 - 22	4 - 9
Return Activated Sludge (% of Influent Flowrate)	20 - 50	50 - 100	50 - 100	---
Internal Recycle (% of Influent Flowrate)	100 - 300	400	100 - 600	--

9.3.4 Sequencing Batch Reactor (SBR)

The SBR can be operated to achieve any combination of carbon oxidation, nitrogen reduction, and phosphorus removal. Reduction of these constituents can be accomplished with or without chemical addition by changing the operation of the reactor. Phosphorus can be removed by coagulant addition or biologically without coagulant addition. By modifying the reaction times, nitrification of nitrogen removal can also be accomplished. Overall cycle time may vary from 3 to 24 hours. A carbon source in the anoxic phase is required to support denitrification- either an external source or endogenous respiration of the existing biomass.

9.3.5 Detailed Design Manuals

The following sources contain detailed design information for biological nutrient removal:

Water Pollution Control Federation: *Nutrient Control, Manual of Practice* FD-7, Washington, DC, 1983.

U.S. Environmental Protection Agency: *Design Manual for Phosphorus Removal*, EPA 625/ 1-87-001, Cincinnati, OH, 1987.

U.S. Environmental Protection Agency: *Process Design Manual for Nitrogen Control*, Office of Technology Transfer, Washington, DC, October 1975.

U.S. Environmental Protection Agency: *Process Design Manual for Phosphorus Removal*, Office of Technology Transfer, Washington, DC, April 1976.

9.4 EFFLUENT FILTRATION

9.4.1 General

9.4.1.1 *Applicability*

Effluent filtration is generally necessary when effluent quality better than 15 mg/L BOD_5, 15 mg/L suspended solids and 1.0 mg/L phosphorus is required.

Where effluent suspended solids requirements are less than 10 mg/L, where secondary effluent quality can be expected to fluctuate significantly, or where filters follow a treatment process where significant amounts of algae will be present, a pre-treatment process such as chemical coagulation and sedimentation or other acceptable process should precede the filter units.

9.4.1.2 *Design Considerations*

Factors to consider when choosing between the different filtration systems which are available, include the following:

a. the installed capital and expected operating and maintenance costs;

b. the energy requirements of the systems (head requirements);

c. the media types and sizes and expected solids capacities and treatment efficiencies of the system; and

d. the backwashing systems, including type, backwash rate, backwash volume, effect on sewage works, etc.

Care should be given in the selection of pumping equipment ahead of filter units to minimize shearing of floc particles. Consideration should be given in the plant design to providing flow-equalization facilities to moderate filter influent quality and quantity.

9.4.2 Location of Filter System

Effluent filtration should precede the chlorine contact chamber to minimize chlorine usage, to allow more effective disinfection and to minimize the production of chloro-organic compounds.

To allow excessive biological growths and grease accumulations to be periodically removed from the filter media, a chlorine application point should be provided upstream of the filtration system (chlorine would only be dosed as necessary at this location).

9.4.3 Number of Units

Total filter area shall be provided in 2 or more units, and the filtration rate shall be calculated on the total available filter area with one unit out of service.

9.4.4 Filter Types

Filters may be of the gravity type or pressure type. Pressure filters shall be provided with ready and convenient access to the media for treatment or cleaning. Where greases or similar solids which result in filter plugging are expected, filters should be of the gravity type.

9.4.5 Filtration Rates

9.4.5.1 *Hydraulic Loading Rate*

Filtration rates at peak hourly sewage flow rates, including backwash flows, should not exceed 2.1 $L/m^2 \cdot s$ for shallow bed single media systems (if raw sewage flow equalization is provided, lower peak filtration rates should be used in order to avoid under-sizing of the filter).

Filtration rates at peak hourly sewage flow rates, including backwash flows, should not exceed 3.3 $L/m^2 \cdot s$ for deep bed filters (if raw sewage flow equalization is provided, lower peak filtration rates should be used in order to avoid undersizing of the filter). The manufacturer's recommended maximum filtration rate should, however, not be exceeded.

9.4.5.2 *Organic Loading Rate*

Peak solids loading rate should not exceed 50 $mg/m^2 \cdot s$ for shallow bed filters and 80 $mg/m^2.s$ for deep bed filters (if raw sewage flow equalization is provided, lower peak solids loading rates should be used in order to avoid undersizing of the filter).

9.4.6 Backwash

9.4.6.1 *Backwash Rate*

The backwash rate shall be adequate to fluidize and expand each media layer a minimum of 20 percent based on the media selected. The backwash system shall be capable of providing a variable backwash rate having a maximum of at least 14 $L/m^2 \cdot s$ and a minimum backwash period of 10 minutes.

9.4.6.2 *Backwash*

Pumps for backwashing filter units shall be sized and interconnected to provide the required rate to any filter with the largest pump out of service. Filtered water should be used as the source of backwash water. Waste filter backwash shall be adequately treated.

Air scour or mechanical agitation systems to improve backwash effectiveness are recommended.

If instantaneous backwash rates represent more than 15 percent of the average daily design flow rate of the plant, a backwash holding tank should be provided to equalize the flow of backwash water to the plant.

9.4.7 Filter Media

9.4.7.1 *Selection*

Selection of proper media size will depend on the filtration rate selected, the type of treatment provided prior to filtration, filter configuration, and effluent quality objectives. In dual or multi-media filters, media size selection must consider compatibility among media.

9.4.7.2 *Media Specifications*

The following table provides minimum media depths and the normally acceptable range of media sizes. The designer has the responsibility for selection of media to meet specific conditions and treatment requirements relative to the project under consideration.

TABLE 9.6 MEDIA DEPTHS AND SIZES

(Minimum Depth)
(Effective Size)

	Single Media	Multi-Media (2)	Multi-Media (3)
Anthracite	-	50 cm 1.0 - 2.0 mm	50 cm 1.0 - 2.0 mm
Sand	120 cm 1.0 - 4.0 mm	30 cm 0.5 - 1.0 mm	25 cm 0.6 - 0.8 mm
Garnet or Similar Material	-	-	5 cm 0.3 - 0.6 mm

Uniformity Coefficient shall be 1.7 or less

9.4.8 Filter Appurtenances

The filters shall be equipped with washwater troughs, surface wash or air scouring equipment, means of measurement and positive control of the backwash rate, equipment for measuring filter head loss, positive means of shutting off flow to a filter being backwashed, and filter influent and effluent sampling points. If automatic controls are provided, there shall be a manual override for operating equipment, including each individual valve essential to the filter operation. The underdrain system shall be designed for uniform distribution of backwash water (and air, if provided) without danger of clogging from solids in the backwash water. Provision shall be made to allow periodic chlorination of the filter influent or backwash water to control slime growths. If air is to be used for filter backwash, separate backwash blowers shall be provided.

9.4.9 Reliability

Each filter unit shall be designed and installed so that there is ready and convenient access to all components and the media surface for inspection and maintenance without taking other units out of service. The need for housing of filter units shall depend on expected extreme climatic conditions at the treatment plant site. As a minimum, all controls shall be enclosed. The structure housing filter controls and equipment shall be provided with adequate heating and ventilation equipment to minimize problems with excess humidity.

9.4.10 Backwash Surge Control

The rate of return of waste filter backwash water to treatment units should be controlled such that the rate does not exceed 15 percent of the design average daily flow rate to the treatment units. The hydraulic and organic load from waste backwash water shall be considered in the overall design of the treatment plant. Surge tanks shall have a minimum capacity of two backwash volumes, although additional capacity should be considered to allow for operational flexibility. Where waste backwash water is returned for treatment by pumping, adequate pumping capacity shall be provided with the largest unit out of service.

9.4.11 Backwash Water Storage

Total backwash water storage capacity provided in an effluent clearwell or other unit shall equal or exceed the volume required for two complete backwash cycles.

9.4.12 Proprietary Equipment

Where proprietary filtration equipment not conforming to the preceding requirements is proposed, data which supports the capability of the equipment to meet effluent requirements under design conditions shall be provided. Such equipment will be reviewed on a case-by case basis at the discretion of the regulatory agencies.

9.5 MICROSCREENING

9.5.1 General

9.5.1.1 *Applicability*

Microscreening units may be used following a biological treatment process for the removal of residual suspended solids. Selection of this unit process should consider final effluent requirements, the preceding biological treatment process, and anticipated consistency of the biological process to provide a high quality effluent.

9.5.1.2 *Design Considerations*

Pilot plant testing on existing secondary effluent is encouraged. Where pilot studies so indicate, where microscreens follow trickling filters or lagoons, or where effluent suspended solids requirements are less than 10 mg/L, a pre-treatment process such as chemical coagulation and sedimentation shall be provided. Care should be taken in the selection of pumping equipment ahead of microscreens to minimize shearing of floc particles. The process design shall include flow equalization facilities to moderate microscreen influent quality and quantity.

9.5.2 Screen Material

The microfabric shall be a material demonstrated to be durable through long-term performance data. The aperture size must be selected considering required removal efficiencies, normally ranging from 20 to 35 microns. The use of pilot plant testing for aperture size selection is recommended.

9.5.3 Screening Rate

The screening rate shall be selected to be compatible with available pilot plant test results and selected screen aperture size, but shall not exceed 3.4 $L/m^2 \cdot s$ of effective screen area based on the maximum hydraulic flow rate applied to the units. The effective screen area shall be considered as the submerged screen surface area less the area of screen blocked by structural supports and fasteners. The screening rate shall be that applied to the units with one unit out of service.

9.5.4 Backwash

All waste backwash water generated by the microscreening operation shall be recycled for treatment. The backwash volume and pressure shall be adequate to assure maintenance of fabric cleanliness and flow capacity. Equipment for backwash of at least 1.65 L/m·s of screen length and 4.22 kgf/cm^2, respectively, shall be provided. Backwash water shall be supplied continuously by multiple pumps, including one standby, and should be obtained from microscreened effluent. The rate of return of waste backwash water to treatment units shall be controlled such that the rate does not exceed 15 percent of the design average daily flow rate to the treatment plant. The hydraulic and organic load from waste backwash water shall be considered in the overall design of the treatment plant. Where waste backwash is returned for treatment by pumping, adequate pumping capacity shall be provided with the largest unit out of service. Provisions should be made for measuring backwash flow.

9.5.5 Appurtenances

Each microscreen unit shall be provided with automatic drum speed controls with provisions for manual override, a bypass weir with an alarm for use when the screen becomes blinded to prevent excessive head development, and means for dewatering the unit for inspection and maintenance. Bypassed flows must be segregated from water used for backwashing. Equipment for control of biological slime growths shall be provided. The use of chlorine should be restricted to those installations where the screen material is not subject to damage by the chlorine.

9.5.6 Reliability

A minimum of two microscreen units shall be provided, each unit being capable of independent operation. A supply of critical spare parts shall be provided and maintained. All units and controls shall be enclosed in a heated and ventilated structure with adequate working space to provide for ease of maintenance.

9.6 ACTIVATED CARBON ADSORPTION

9.6.1 Applicability

In tertiary treatment, the role of activated carbon is to remove the relatively small quantities of refractory organics, as well as inorganic compounds such as nitrogen, sulfides, and heavy metals, remaining in an otherwise well-treated wastewater.

Activated carbon may also be used to remove soluble organics following chemical-physical treatment.

9.6.2 Design Considerations

The usefulness and efficiency of carbon adsorption for municipal wastewater treatment depends on the quality and quantity of the delivered wastewater. To be fully effective, the carbon unit should receive an effluent of uniform quality, without surges in the flow. Other wastewater qualities of concern include suspended solids, oxygen demand, other organics such as methylene blue active substance (MBAS) or phenol, and dissolved oxygen. Environmental parameters of importance include pH and temperature. Consideration also should be given to the type of activated carbon available. Activated carbons produced from different base materials and by different activation processes will have varying adsorptive capacities. Some factors influencing adsorption at the carbon/liquid interface are:

a. attraction of carbon for solute;
b. attraction of carbon for solvent;
c. solubilizing power of solvent or solute;
d. association;
e. ionization;
f. effect of solvent on orientation at interface;
g. competition for interface in presence of multiple solutes;
h. coadsorption;
i. molecular size of molecules in the system;
j. pore size distribution in carbon;
k. surface area of carbon; and
l. concentration of constituents.

There are several different activated carbon contactor systems that can be selected. The carbon columns can be either of the pressure or gravity type.

9.6.3 Unit Sizing

9.6.3.1 *Contact Time*

The contact time shall be calculated on the basis of the volume of the column occupied by the activated carbon. Generally, carbon contact times of 15 to 35 min are used depending on the application, the wastewater characteristics, and the desired effluent quality. For tertiary treatment applications, carbon contact times of 15 to 20 min should be used where the desired effluent quality is a COD of 10 to 20 mg/L, and 30 to 35 min when the desired effluent COD is 5 to 15 mg/L. For chemical-physical treatment plants, carbon contact times of 20 to 35 min should be used, with a contact time of 30 min being typical.

9.6.3.2 *Hydraulic Loading Rate*

Hydraulic loading rates of 2.5 to 7.0 $L/m^2 \cdot s$ of cross section of the bed shall be used for upflow carbon columns. For downflow carbon columns, hydraulic loading rates of 2.0 to 3.3 $L/m^2 \cdot s$ are used. Actual operating pressure seldom rises above 7 kN/m^2 for each 0.3 m of bed depth.

9.6.3.3 *Depth of Bed*

The depth of bed will vary considerably, depending primarily on carbon contact time, and may be from 3 to 12 m. A minimum carbon depth of 3 m is recommended. Typical total carbon depths range from 4.5 to 6 m. Freeboard has to be added to the carbon depth to allow an expansion of 10 to 50 percent for the carbon bed during backwash or for expanded bed operation. Carbon particle size and water temperature will determine the required quantity of backwash water to attain the desired level of bed expansion.

9.6.3.4 ***Number of Units***

A minimum of two parallel carbon contactor units are recommended for any size plant. A sufficient number of contactors should be provided to insure an adequate carbon contact time to maintain effluent quality while one column is off line during removal of spent carbon for regeneration or for maintenance.

9.6.4 **Backwashing**

The rate and frequency of backwash is dependent on hydraulic loading, the nature and concentration of suspended solids in the wastewater, the carbon particle size, and the method of contacting. Backwash frequency can be prescribed arbitrarily (each day at a specified time), or by operating criteria, (headloss or turbidity). Duration of backwash may be 10 to 15 min.

The normal quantity of backwash water employed is less than 5 percent of the product water for a 0.8 m deep filter and 10 to 20 percent for a 4.5 m filter.

Recommended backwash flow rates for granular carbons of 8 x 12 or 12 x 30 mesh are 8 to 14 $L/m^2 \cdot s$.

9.6.5 **Valve and Pipe Requirements**

Upflow units shall be piped to operate either as upflow or downflow units as well as being capable of being backwashed. Downflow units shall be piped to operate as downflow and in series. Each column must be valved to be backwashed individually. Furthermore, downflow series contactors should be valved and piped so that the respective position(s) of the individual contactors can be interchanged.

9.6.6 **Instrumentation**

The individual carbon columns should be equipped with flow and headloss measuring devices.

9.6.7 **Hydrogen Sulphide Control**

Methods that can be incorporated into the plant design to cope with hydrogen sulphide production include:

1. Providing upstream biological treatment to satisfy as much of the biological oxygen demand as possible prior to carbon treatment;

2. Reducing detention time in the carbon columns based on dissolved oxygen concentrations of the effluent;

3. Backwashing the columns at more frequent intervals;

4. Chlorinating carbon column influent; and

5. In upflow expanded beds, the introducing of an oxygen source, such as air or hydrogen peroxide, to keep the columns aerobic.

9.6.8 Carbon Transport

Provisions must be made to remove spent carbon from the carbon contactors. It is important to obtain a uniform withdrawal of carbon over the entire horizontal surface area of the carbon bed. Care must be taken to insure that gravel or stone supporting media used in downflow contactors does not enter the carbon transport system.

Activated carbon shall be transported hydraulically. Carbon slurries can be transported using water or air pressure, centrifugal or diaphragm pumps, or eductors. The type of motive equipment selected requires a balance of owner preference, column control capabilities, capital and maintenance costs, and pumping head requirements.

Carbon slurry piping systems shall be designed to provide approximately 8 L of transport water for each kg of carbon removed. Pipeline velocities of 0.9 to 1.5 m/s are recommended.

Long-radius elbows or tees and crosses with cleanouts should be used at points of pipe direction change. Valves should be of the ball or plug type. No valves should be installed in the slurry piping system for the purpose of throttling flows.

9.6.9 Carbon Regeneration

9.6.9.1 *Quantities of Spent Carbon*

The carbon dose used to size the regeneration facilities depends on the strength of the wastewater applied to the carbon and the required effluent quality. Typical carbon dosages that might be anticipated for municipal wastewaters are shown in Table 9.7.

TABLE 9.7 – TYPICAL CARBON DOSAGES FOR DIFFERENT COLUMN WASTEWATER INFLUENTS

PRETREATMENT	TYPICAL CARBON DOSAGE REQUIRED PER m^3 OF COLUMN THROUGHPUT (g/m^3)*
COAGULATED, SETTLED AND FILTERED ACTIVATED SLUDGE EFFLUENT	35 - 70
FILTERED SECONDARY EFFLUENT	70 - 100
COAGULATED, SETTLED, AND FILTERED RAW WASTEWATER (PHYSICAL - CHEMICAL)	100 - 300

**LOSS OF CARBON DURING EACH REGENERATION CYCLE TYPICALLY WILL BE 5 TO 10 PER CENT. MAKE-UP CARBON IS BASED ON CARBON DOSAGE AND THE QUALITY OF THE REGENERATED CARBON*

9.6.9.2 *Carbon Dewatering*

Dewatering of the spent carbon slurry prior to thermal regeneration may be accomplished in spent carbon drain bins. The drainage bins shall be equipped with screens to allow the transport of water to flow from the carbon. Two drain bins shall be provided.

Dewatering screws may also be used to dewater the activated carbon. A bin must be included in the system to provide a continuous supply of carbon to the screw, as well as maintain a positive seal on the furnace.

9.6.9.3 *Regeneration Furnace*

Partially dewatered carbon may be fed to the regeneration furnace with a screw conveyor equipped with a variable speed drive to control the rate of carbon feed precisely.

The theoretical furnace capacity is determined by the anticipated carbon dosage. An allowance for furnace downtime on the order of 40 percent should be added to the theoretical capacity.

Based on the experience gained from two full-scale facilities, provisions should be made to add approximately 1 kg of steam per kg of carbon regenerated. Fuel requirements for the carbon regeneration furnace are 7000 kJ/kg of carbon when regenerating spent carbon on tertiary and secondary effluent applications. To this value, the energy requirements for steam and an afterburner, if required, must be added.

The furnace shall be designed to control the carbon feed rate, rabble arm speed, and hearth temperatures. The off-gases from the furnace must be within acceptable air pollution standards. Air pollution control equipment shall be designed as an integral part of the furnace and include a scrubber for removing carbon fines and an afterburner for controlling odours.

9.7 CONSTRUCTED WETLANDS

9.7.1 General

Constructed wetlands are inundated land areas with water depths typically less than 0.6 m that support the growth of emergent plants such as cattail, bulrush, reeds, and sedges. The vegetation provides surface for the attachment of bacterial films, aids in the filtration and adsorption of wastewater constituents, transfers oxygen into the water column, and controls the growth of algae by restricting the penetration of sunlight.

Although plant uptake is an important consideration in contaminant, particularly nutrient, removal it is only one of many active removal mechanisms in the wetland environment. Removal mechanisms have been classified as physical, chemical and biological and are operative in the water column, the humus and soil column beneath the growing plants, and at the interface between the water and soil columns. Because most of the biological transformations take place on or near a surface to which bacteria are attached, the presence of vegetation and humus is very important. Wetland systems are designed to provide maximum production of humus material through profuse plant growth and organic matter decomposition.

9.7.2 Types

Wastewater treatment systems using constructed wetlands have been categorized as either free water surface (FWS) or subsurface flow (SFS) types.

a. Free Water Surface Wetlands (FWS)

A FWS system consists of basins or channels with a natural or constructed subsurface barrier to minimize seepage. Emergent vegetation is grown and wastewater is treated as it flows through the vegetation and plant litter. FWS wetlands are typically long and narrow to minimize short-circuiting.

b. Subsurface Flow Wetlands (SFS)

A SFS wetland system consists of channels or basins that contain gravel or sand media which will support the growth of emergent vegetation. The bed of impermeable material is sloped typically between 0 and 2 percent. Wastewater flows horizontally through the root zone of the wetland plants about 100 to 150 mm below the gravel surface. Treated effluent is collected in an outlet channel or pipe.

9.7.3 Site Evaluation

Site characteristics that must be considered in wetland system design include topography, soil characteristics, existing land use, flood hazard, and climate.

a. Topography

Level to slightly sloping, uniform topography is preferred for wetland sites because free water systems (FWS) are generally designed with level basins or channels, and subsurface flow systems (SFS) are normally designed and constructed with slopes of 1 percent or slightly more. Although basins may be constructed on steeper sloping or uneven sites, the amount of earthwork required will affect the cost of the system. Thus, slope gradients should be less than 5 percent.

b. Soil

Sites with slowly permeable (<0.5 cm/h) surface soils or subsurface layers are most desirable for wetland systems because the objective is to treat the wastewater in the water layer above the soil profile. Therefore, percolation losses through the soil profile should be minimized. As with overland-flow systems, the surface soil will tend to seal with time due to deposition of solids and growth of bacterial slimes. Permeabilities of native soils may be purposely reduced by compacting during construction. Sited with rapidly permeable soils may be used for small systems by constructing basins with clay or artificial liners. The depth of soil to groundwater should be a minimum of 0.3 - 0.6 m to allow sufficient distance for treatment of any percolate entering the groundwater.

c. Flood Hazard

Wetland sites should be located outside of flood plains, or protection from flooding should be provided.

d. Existing Land Use

Open space or agricultural lands, particularly those near existing natural wetlands, are preferred for wetland sites. Constructed wetlands can enhance existing natural wetlands by providing additional wildlife habitat and , in some cases, by providing a more consistent water supply.

e. Climate

The use of wetland systems in cold climates is possible. Because the principle treatment systems are biological, treatment performance is strongly temperature sensitive. Storage will be required where treatment objectives cannot be met due to low temperatures.

9.7.4 Preapplication Treatment

Artificial wetlands may be designed to accept wastewater with minimal (coarse screening and comminution) pretreatment. However, the level of pretreatment will influence the quality of the final effluent and therefore overall treatment objectives must be considered. Since there is no permanent escape mechanism for phosphorus within the wetland. Phosphorus reduction by chemical addition is also recommended as a pretreatment step to ensure continued satisfactory phosphorus removal within the marsh.

9.7.5 Vegetation Selection and Management

The plants most frequently used in constructed wetlands include cattails, reeds, rushes, bulrushes, and sedges. All of these plants are ubiquitous and tolerate freezing conditions. The important characteristics of the plants related to design are the optimum depth of water for FWS systems and the depth of rhizome and root systems for SFS systems. Cattails tend to dominate in water depths over 0.15 m. Bulrushes grow well at depths of 0.05 - 0.25 m. Reeds grow along the shoreline and i water up to 1.5 m deep, but are poor competitors in shallow waters. Sedges normally occur along the shoreline and in shallower water than bulrushes. Cattail rhizomes and roots extend to a depth of approximately 0.3 m, whereas reeds extend to more than 0.6 m and bulrushes to more than 0.75 m. Reeds and bulrushes are normally selected for SFS systems because the depth of rhizome penetration allows for the use of deeper basins.

Harvesting of wetland vegetation is generally not required, especially for SFS systems. However dry grasses in FWS systems are burned off periodically to maintain free-flow conditions and to prevent channelling of the flow. Removal of the plant biomass for the purpose of nutrient removal is normally not practical.

9.7.6 Design Parameters

9.7.6.1 *Detention Time*

a. Free Water Surface Wetlands (FWS)

The relationship between BOD removal and detention times for FWS is represented by the equation:

$$C_c = Co \exp (-k_T t)]$$

where :

C_e = effluent BOD, mg/L
C_o = influent BOD, mg/L

k_T = temperature dependent rate constant, d^{-1}

	=	$k_{20} \times 1.06^{(T-20)}$
k_{20}	=	$0.678 d^{-1}$
T	=	average monthly water temperature, ^oC
t	=	average detention time, d
	=	LWnd/Q
L	=	basin length, m
W	=	basin width, m
n	=	fraction of cross sectional area not used by plants (0.65-0.75)
d	=	depth of basin, m
Q	=	average flowrate through system [(Qin + Qout) / 2], m^3/d

b. Subsurface Flow Systems (SFS)

The relationship between BOD removal and detention times for SFS is represented by the equation:

$$C_e = C_o \exp(-K_T t')$$

where

C_e	=	effluent BOD, mg/L
C_o	=	influent BOD, mg/L
K_T	=	temperature dependent rate constant, d^{-1}
	=	$K_{20} \times 1.06^{(T-20)}$
T	=	average monthly water temperature, ^oC
K_{20}	=	$1.104\ d^{-1}$
t'	=	$(LW\alpha d) / Q$
L	=	basin length, m
W	=	basin width, m
α	=	porosity of basin medium
	=	0.35 (gravelly sand); 0.30 (coarse sand); 0.28 (medium sand)
d	=	depth of basin, m
Q	=	average flowrate through system [(Qin + Qout) / 2], m^3/d

9.7.6.2 *Water Depth*

For FWS, the design water depth depends on the optimum depth for the selected vegetation. In cold climates, the operating depth is normally increased in the winter to allow for ice formation on the surface and to provide the increased detention time required at colder temperatures. Systems should be designed with an outlet structure that allows for varied operating depths. Water depths should range from 0.1 - 0.5 m.

The design depth of SFS systems is controlled by the depth of penetration of the plant rhizomes and roots because the plants supply oxygen to the water through the root/rhizome system. The media depth may range from 0.3 - 0.75 m.

9.7.6.3 *Aspect Ratio*

The aspect ration for FWS wetlands is important to the performance for removal of BOD, TSS, NH_3, and total nitrogen. Length to width ratios of 4:1 to 6:1 are needed to achieve expected performances and avoid short circuiting of wastewater through the wetland. For large systems, an aspect ratio of 2:1 is the minimum recommended.

For SFS wetlands the bed width is determined by the hydraulic flowrate. The length of the bed is determined by the needed detention time for pollutant removal. Therefore SFS wetlands may have aspect ratios less than or greater than 1:1 depending on the treatment goal.

9.7.6.4 *Loading Rates*

Table 9.8 summarizes the hydraulic, BOD, and SS loading rates for BOD removal in both FWS and SFS systems.

TABLE 9.8 - LOADING RATES FOR CONSTRUCTED WETLANDS

Wetland Type	Hydraulic Loading Rate	Maximum BOD Loading Rate	Maximum SS Loading Rate at inlet
Free Water Surface	150 - 500 m^3/ha.d	65 kg/ ha·d	Not applicable
Subsurface Flow System	Not Applicable	65 kg/ha·d	0.08 kg/m^2.d

9.7.6.5 *Nutrient Removal*

a. Free Water Surface Wetlands (FWS)

Detention times for nutrient removal need to be longer than the 5 - 10 days required for BOD and SS. For ammonia or total nitrogen removal, both minimum temperature and detention time are important. Detention times for significant nitrogen removal should be 8 - 14 days or more. Nitrogen removal and nitrification will be reduced when water temperatures fall below 10 ^oC and should not be expected when water temperatures fall below 4 ^oC.

Plant uptake of phosphorus is rapid, and following plant death, phosphorus may be quickly recycled to the water column or deposited in the sediments. The only major sink for phosphorus in most wetlands is in the soil. Significant phosphorus removal requires long detention times (15 - 20 days) and low phosphorus loading rates (< 0.3 kg/ha·d).

b. Subsurface Flow Wetlands (SFS)

Both detention time and oxygen transfer can limit nitrification and subsequent nitrogen removal in SFS wetlands. Because nitrification of 20 mg/L of ammonia will require 100 mg/L of oxygen, oxygen transfer is critical to nitrification in SFS wetlands. Plant roots can generate a portio of this demand for oxygen in the subsurface, however, direct oxygen transfer from the atmosphere may be required to achieve effective nitrification. The detention time and temperature limits for FWS apply to SFS wetlands.

9.7.7 Vector Control

FWS systems provide ideal breeding habitat for mosquitoes. Plans for biological control of mosquitoes through the use of mosquito fish and sparrows plus application of chemical control agents as necessary must be incorporated in the design. Thinning of vegetation may also be necessary to eliminate pockets of water that are inaccessible to fish.

Mosquito breeding should not be a problem in SFS systems, provided the system is designed to prevent mosquito access to the subsurface water zone. The surface is normally covered with pea gravel or coarse sand to achieve this purpose.

9.7.8 Vegetation Harvesting

Harvesting of the emergent vegetation is only required to maintain hydraulic capacity, promote active growth, and avoid mosquito growth. Harvesting for nutrient removal is not practical and is not recommended.

9.7.9 Monitoring

Monitoring is necessary to maintain loadings within design limits. A routine monitoring program should be established for the following parameters:

a. wastewater application rates (m^3/d);

b. discharge flow rates (m^3/d);

c. wastewater quality, including BOD_5 and COD, suspended solids, total dissolved solids, total nitrogen, total phosphorous, pH and sodium adsorption ratio; and

d. discharge water quality according to the analyses summarized in item (c).

9.8 FLOATING AQUATIC PLANT TREATMENT SYSTEMS

9.8.1 General

Aquatic treatment systems consist of one or more shallow ponds in which one or more species of water tolerant vascular plants such as water hyacinths or duckweed are grown. The shallower depths and the presence of aquatic macrophytes in the place of algae are the major differences between aquatic treatment systems and stabilization ponds. The presence of plants is of great practical significance because the effluent from aquatic systems is of higher quality than the effluent from stabilization pond systems for equivalent or shorter detention times. This is true, particularly when the systems are situated after conventional pond systems which provide greater than primary treatment.

In aquatic systems, wastewater is treated principally by bacterial metabolism and physical sedimentation, as is the case in conventional trickling filter systems. The aquatic plants themselves bring about very little actual treatment of the wastewater. Their function is to provide components of the aquatic environment that improve the wastewater treatment capability and/or reliability of that environment.

9.8.2 Plant Selection

The principal floating aquatic plants used in aquatic treatment systems are water hyacinth, duckweed and pennywort. These plants are described in greater detail in the following discussion.

9.8.2.1 Water Hyacinths

Water hyacinth is a perennial, fresh water aquatic vascular plant with rounded, upright, shiny green leaves and spikes of lavender flowers. The petioles of the plant are spongy with many air spaces and contribute to the buoyancy of the hyacinth plant. When grown in waste water, individual plants range from 0.5 to 1.2 m from the top of the flower to the root tips. The plants spread laterally until the water surface is covered, and then the vertical growth increases. The growth of water hyacinth is influenced by efficiency of the plant to use solar energy, nutrient composition of the water, cultural methods, and environmental factors.

Under normal conditions, loosely packed water hyacinths can cover the water surface at relatively low plant densities, about 10 kg/m^2 wet weight. Plant densities as high as 80 kg/m^2 wet weight can be reached. As in other biological processes, the growth rate of water hyacinths is dependent on temperature. Both air and water temperatures are important in assessing plant vitality.

9.8.2.2 Duckweed

Duckweed are small, green freshwater plants with fronds from one to a few millimetres in width with a short root, usually less than 12 mm in length. Duckweed are the smallest and the simplest of the flowering plants and have one of the fastest reproduction rates. Duckweed grown in wastewater effluent (at 27°C) doubles in frond numbers, and therefore in area covered, every four days. The plant is essentially all metabolically active cells with very little structural fibre.

Small floating plants, particularly duckweed, are sensitive to wind and may be blown in drifts to the leeward side of the pond unless baffles are used. Redistribution of the plants requires manual labour. If drifts are not redistributed, decreased treatment efficiency may result due to incomplete coverage of the pond surface. Odours have also developed where accumulated plants are allowed to remain and undergo anaerobic decomposition.

9.8.2.3 Pennywort

Pennywort is generally a rooted plant. However, under high-nutrient conditions, it may form hydroponic rafts that extend across water bodies. Pennywort tends to intertwine and grows horizontally; at high densities, the plants tend to grow vertically. Unlike water hyacinth, the photosynthetic leaf area of pennywort is small, and, at dense plant stands, yields are significantly reduced as a result of self shading. Pennywort exhibits mean growth rates greater than 0.010 $kg/m^2{\cdot}d$ in warm climates. Although rates of nitrogen and phosphorous uptake by water hyacinth drop sharply during the winter, nutrient uptake by pennywort is approximately the same during both warm and cool seasons. Pennywort is a cool season plant that can be integrated into water hyacinth/water lettuce biomass production systems.

9.8.3 Types of Systems

The principal types of floating aquatic plant treatment systems used for wastewater treatment are those employing water hyacinth and duckweed.

9.8.3.1 *Water Hyacinth Systems*

Water hyacinth systems represent the majority of aquatic plant systems that have been constructed. Three types of hyacinth systems can be described based on the level of dissolved oxygen and the method of aerating the pond: (1) aerobic nonaerated, (2) aerobic aerated, and (3) facultative anaerobic.

A nonaerated aerobic hyacinth system will produce secondary treatment or nutrient (nitrogen) removal depending on the organic-loading rate. This type of system is the most common of the hyacinth systems now in use. The advantages of this type of system include excellent performance with few mosquitoes or odours.

For plant locations in which no mosquitoes or odours can be tolerated, an aerated aerobic hyacinth system is required. The added advantages of such a system are that with aeration, higher organic-loading rates are possible, and reduced land area is required.

The third configuration for a hyacinth system is known as a facultative anaerobic hyacinth system. These systems are operated at very high organic-loading rates. Odours and increased mosquito populations are the principal disadvantages of this type of system. Facultative anaerobic hyacinth systems are seldom used because of these problems.

9.8.3.2 *Duckweed Systems*

Duckweed and pennywort have been used primarily to improve the effluent quality from facultative lagoons or stabilization ponds by reducing the algae concentration. Conventional lagoon design may be followed for this application, except for the need to control the effects of wind. Without controls, duckweed will be blown to the downwind side of the pond, resulting in exposure of large surface areas and defeating the purpose of the duckweed cover. As noted previously, accumulations of decomposing plants can also result in the production of odours. Floating baffles can be used to construct cells of limited size to minimize the amount of open surface area exposed to wind action.

9.8.4 Climatic Constraints

The water hyacinth systems that are currently used to treat wastewater are located in the warm temperature climates. The optimum water temperature for water hyacinth growth is 21-30°C. Air temperatures of -3°C for 12 hours will destroy the leaves and exposure at -5°C for 48 hours will kill the plants. If a water hyacinth system were to be used in a colder climate, it would be necessary to house the system ion a greenhouse and maintain the temperature in the optimum range. Duckweed is more cold tolerant than water hyacinths and can be grown practically at temperatures as low as 7°C.

9.8.5 Preapplication Treatment

The minimum level of preapplication treatment should be primary treatment, short detention time aerated ponds or the equivalent. Treatment beyond primary depends on the effluent requirements. Use of oxidation ponds or lagoons in which high concentrations of algae are generated should be avoided prior to aquatic treatment because algae removal is inconsistent. When there are effluent limitations on phosphorus, it should be removed in the preapplication treatment step because phosphorus removal in aquatic treatment systems is minimal.

9.8.6 Design Parameters

The principal design parameters for aquatic treatment systems include hydraulic detention time, water depth, pond geometry, organic-loading rate, and hydraulic loading rate. Typical design guidelines for water hyacinth and duckweed systems are summarized in Table 9.9 for different levels of pre-application treatment.

TABLE 9.9 - FLOATING AQUATIC PLANT SYSTEM DESIGN CRITERIA

ITEM	TYPE OF WATER HYACINTH TREATMENT SYSTEM			DUCKWEED TREATMENT SYSTEM
	Secondary Aerobic (non-aerated)	Secondary Aerobic (aerated)	Nutrient Removal Aerobic (nonaerated)	
Influent Wastewater	Primary Effluent	Primary Effluent	Secondary Effluent	Facultative Pond Effluent
Influent BOD_5 (mg/L)	130 – 180	130 - 180	30	40
BOD_5 Loading (kg/ ha·d)	45 – 90	170 - 340	10 - 45	22 - 28
Water Depth (m)	0.5 - 1.0	1.0 - 1.3	0.7 - 1.0	1.3 - 2.0
Detention Time (d)	10 – 36	4 - 8	6 – 18	20 - 25
Hydraulic Loading Rate (m^3/ ha·d)	190 - 570	95 - 285	375 – 1500	570 - 860
Water Temperature (^oC)	> 10	>10	>10	>7
Harvest Schedule	Seasonally	Bi - monthly	Bi – monthly	Monthly

9.8.7 Pond Configuration

9.8.7.1 Water Hyacinth Systems

Typical pond configurations used for water hyacinth systems involve rectangular basins in series similar to stabilization ponds. Recycle and step feed are employed to reduce the concentration of the organic constituent at the plant root zone, improve the transport of wastewater to the root zone, and reduce the formation of odours.

9.8.7.2 Duckweed Systems

Duckweed systems should be designed as conventional stabilization ponds except for the need to control the effects of wind. Floating baffles are used to minimize the amount of surface area exposed to direct wind action. Without this control, duckweed will be blown by the wind and treatment efficiencies cannot be achieved.

9.8.8 Plant Harvesting and Processing

The need for plant harvesting depends on water quality objectives, the growth rates of the plants, and the effects of predators such as weevils. Harvesting of aquatic plants is needed to maintain a crop with high metabolic uptake of nutrients. Frequent harvesting of hyacinths is practiced to achieve nutrient removal. Significant phosphorus removal is achieved only with frequent harvesting. In areas where weevils pose a threat to healthy hyacinth populations, selective harvesting is often used to keep the plants from being infected. Duckweed harvesting for nutrient removal may be required as often as once per week during warm periods.

Harvested water hyacinth plants are typically dried and landfilled or spread on land and tilled into the soil. Water hyacinth can also be composted readily. However, if the plants are not first partially dried or squeezed, the high moisture content tends to reduce the effectiveness of the compost process and results in the production of a liquid stream that must be disposed of. Ground duckweed can be used as animal feed without air drying.

9.8.9 Detailed Design Guidelines

The following sources contain detailed design information for natural wastewater treatment systems:

Water Pollution Control Federation: *Natural Systems for Wastewater Treatment*, Manual of Practice FD-16, Alexandria, VA, 1990.

U.S. Environmental Protection Agency: *Design Manual for Constructed Wetlands and Aquatic Plant Systems for Municipal Wastewater Treatment*, EPA 625/ 1-88-022, Cincinnati, OH, 1988.

Chapter 10 RECLAIMED WATER USE

10.1 GENERAL

With many communities throughout the world approaching or reaching the limits of their available water supplies, reclaimed water use has become an attractive option for conserving and extending available water supplies. Reclaimed water use is the controlled application of treated wastewater onto the land surface to achieve disposal, utilization, and/or treatment of the wastewater.

Water reclamation and nonpotable reuse only require conventional water and wastewater treatment technology that is widely practised and readily available in countries throughout the world. Furthermore, because properly implemented nonpotable reuse does not entail significant health risks, it has generally been accepted and endorsed by the public in the urban and agricultural areas where it has been introduced. This section provides information on planning considerations, re-use applications, water quality considerations, and guidelines for wastewater irrigation and other re-use criteria.

10.2 PRE-DESIGN REPORT

In addition to the required pre-design report, the designer shall include supplemental information, as outlined below, where wastewater application to the land is being considered. This information shall include any material that is pertinent about the location, geology, topography, hydrology, soils, areas for future expansion, and adjacent land use.

10.3 REQUIRED SUPPLEMENTAL INFORMATION

10.3.1 Location

1. A copy of the topographic map of the area showing the exact boundaries of the proposed application area.

2. A topographic map of the total area owned by the applicant at a scale of approximately 1:500. It should show all buildings, the waste disposal system, the spray field boundaries and the buffer zone. An additional map should show the spray field topography in detail with a contour interval of 0.5 m and include buildings and land use on adjacent lands within 400 m of the project boundary.

3. All water supply wells which might be affected shall be located and identified as to use; e.g., potable, industrial, agricultural, and class of ownership; e.g., public, private, etc.

4. All abandoned wells, shafts, etc., shall be located and identified. Pertinent information therein shall be furnished.

10.3.2 Geology

1. The geologic formations (name) and the rock types at the site.

2. The degree of weathering of the bedrock.

3. The local bedrock structure including the presence of faults, fractures and joints.

4. The character and thickness of the surficial deposits (residual soils and glacial deposit).

5. In limestone terrain, additional information about solution openings and sinkholes is required.

6. The source of the above information must be indicated.

10.3.3 Hydrology

1. The depth to seasonal high water table (perched and/or regional) must be given, including an indication of seasonal variations. Static water levels must be determined at each depth for each aquifer in the depth under concern. Critical slope evaluation must be given to any differences in such levels.

2. The direction of groundwater movement and the point(s) of discharge must be shown on one of the attached maps.

3. Chemical analyses indicating the quality of groundwater at the site must be included.

4. The source of the above data must be indicated.

5. The following information shall be provided from existing wells and from such test wells as may be necessary:

    a. Construction details - where available; Depth, well log, pump capacity, static levels, pumping water levels, casing, grout material, and such other information as may be pertinent.

    b. Groundwater quality: e.g., Nitrates, total nitrogen, chlorides, sulphates, pH, alkalinities, total hardness, coliform bacteria, etc.

6. A minimum of one groundwater monitoring well must be drilled in each dominant direction of groundwater movement and between the project site and public wells(s) and/or high-capacity private wells, with provision for sampling at the surface of the water table and at 1.5 m below the water table at each monitoring site. The location and construction of the monitoring well(s) must be approved by the regulatory authority. These may include one or more of the test wells where appropriate.

10.3.4 Soils

1. A soils map of the spray field should be furnished, indicating the various soil types. This may be included on the large-scale topographic map. Soils information can normally be secured through the Federal Department of Energy Mines and Resources, the Federal Department of Agriculture, or the applicable provincial department.

2. The soils should be named and their texture described.

3. Slopes and agricultural practice on the sprayfield are closely related. Slopes on cultivated fields should be limited to 4%.

   Slopes on sodded fields should be limited to 8%. Forested slopes should be limited to 8% for year-round operation, but some seasonal operation slopes up to 14% may be acceptable.

4. The thickness of soils should be indicated. Method of determination should be included.

5. Data should be furnished on the exchange capacity of the soils. In case of industrial wastes particularly, this information must be related to special characteristics of the wastes.

6. Information must be furnished on the internal and surface-drainage characteristics of the soil materials. This includes the soil's infiltration capacity and permeability.

7. Proposed application rates should take into consideration the drainage and permeability of the soils, the discharge capacity, and the distance to the water table.

10.3.5 Agricultural Practice

1. The present and intended soil-crop management practices, including forestation, shall be stated.

2. Pertinent information shall be furnished on existing drainage systems.

3. When cultivated crops are anticipated, the kinds used and the harvesting frequency should be given; the ultimate use of the crop should also be given.

10.3.6 Adjacent Land Use

1. Present and anticipated use of the adjoining lands must be indicated. This information can be provided on one of the maps and may be supplemented with notes.

2. The plan shall show existing and proposed screens, barriers, or buffer zones to prevent blowing spray from entering adjacent land areas.

3. If expansion of the facility is anticipated, the lands which are likely to be used for expanded spray fields must be shown on the map.

10.4 WASTEWATER APPLICATION METHODS

10.4.1 General

Land application of treated sewage effluent is a method of disposing of effluent without direct discharge to surface waters. Ground disposal installations are normally used where the waste contains pollutants which can successfully be removed through distribution to the soil mantle. These pollutants can be removed through organic decomposition in the vegetation-soil complex and by adsorptive, physical, and chemical reactions with earth materials. Preliminary considerations of a site for ground disposal should be the compatibility of the waste with the organic and earth materials and the percolation rates and exchange capacity of the soils. The ground disposal of wastewater will eventually recharge the local groundwater; therefore, the quality, direction and rate of movement, and local use of the groundwater, present and potential, are prime considerations in evaluating a proposed site.

It is essential to provide good vegetation growth conditions and removal of nutrients. It must be realized that a groundwater mound will develop below after it is in use. The major factors in design of ground disposal fields are topography, soils, geology, hydrology, weather, agricultural practice, adjacent land use, and equipment selection and installation.

The primary methods used for distributing wastewater on the land are irrigation, infiltration, and overland flow.

Table 10.1 outlines various features and performance for wastewater land application systems.

TABLE 10.1 – COMPARISON OF FEATURES AND PERFORMANCE FOR WASTEWATER UTILIZATION, TREATMENT AND DISPOSAL SYSTEMS				
SYSTEM REQUIREMENT	**STANDARD RATE IRRIGATION**	**HIGH RATE IRRIGATION**	**OVERLAND FLOW**	**RAPID INFILTRATION**
SOIL PERMEABILITY	MODERATE (MEDIUM TEXTURE SOIL)	MODERATE TO HIGH (MEDIUM TO COARSE TEXTURE SOIL)	LOW (FINE TEXTURE)	RAPID (LOAMY SANDS AND GRAVELS)
UTILIZATION OF WATER AND NUTRIENTS	HIGH	MEDIUM	MEDIUM TO LOW	NONE
SLOPE	UP TO 30% FOR SPRINKLER AND 6% FOR SURFACE METHODS	SAME AS FOR STANDARD RATE	1 - 12%	NOT CRITICAL
STORAGE	HIGH (7-9 MONTHS)	MEDIUM (5-7 MONTHS)	MEDIUM (5-7 MONTHS)	NIL
LAND AREA	HIGH	MEDIUM	MEDIUM	LOW
WATER QUALITY – SALINITY, ETC.	VERY HIGH	HIGH	MEDIUM TO LOW	MEDIUM TO LOW
TREATMENT EFFICIENCY	VERY HIGH	HIGH	HIGH	MEDIUM

10.4.2 Irrigation

10.4.2.1 Piping to Sprinklers

The piping should be arranged to allow the irrigation pattern to be varied easily. Stationary systems are preferred; but if a moveable system is proposed, one main header must be provided with individual connections for each field and sufficient spare equipment must be available to assure non-interrupted irrigation. Facilities must be provided to allow the pipes to be completely drained at suitable points to prevent pollution and freezing.

10.4.2.2 Sprinkling System

Sprinklers must be so located as to give a non-irrigated buffer zone around the irrigated area, and design of the buffer zone must consider wind transport of the wastewaters. The system shall be designed to provide an even distribution over the entire field.

The application rate must be selected low enough to allow the waters to percolate into the soil and to assure proper residency within the soil mantle. Proposed application rates will not be accepted without substantiating data.

In general, sufficient monitoring controls should be provided to indicate the degree of efficiency with which the sprinklers are working. A pressure gauge and flow meter should be provided.

10.4.2.3 Site Buffer Zone

In the absence of detailed assessments, the distance from spray nozzles to the property limit shall be 150m. Spraying is possible at closer distances from the property limit provided that low pressure, low angle, closely spaced sprinklers are used to minimize the formation of aerosols. In addition, the risk associated with aerosols can be minimized by providing a fence or tree screen around the site perimeter and by terminating spraying operations when wind speeds exceed that of a gentle breeze (15 km/h).

10.4.3 Overland Flow

10.4.3.1 Applicability

Overland flow differs from spray irrigation primarily in that a substantial portion of the applied wastewater becomes runoff which must be collected or discharged to a receiving water or stored for further treatment.

Overland flow is best suited for sites having surface soils that are slowly permeable or have a restrictive layer such as a clay pan at depths of 0.3 to 0.6 m. The topsoil should be of sufficient depth and quality for good cover crop establishment. It is possible to design an overland flow system on very permeable soils by constructing an artificial barrier to prevent downward water movement through the soil, although the capital costs of such construction may be prohibitive for all but the smallest systems. A minimum depth to groundwater of one metre is required to maintain aerobic conditions for plant growth. Underdrainage may be used to lower the water table.

Overland flow may be used at sites with gently sloping terrain with grades in the range of 1 to 12%. Slopes can be constructed on nearly level terrain and terraced construction can be used when the natural slope exceeds about 10%. If reuse of the runoff is not possible, then a drainage system must be constructed to transport the treated wastewater from the site.

10.4.3.2 Wastewater Loading

The maximum seasonal wastewater loading rate shall be determined by the following equation:

$$R_W = P + SR$$

where

R_W = loading rate (cm/yr)

P = soil permeability (cm/yr)

SR = seasonal runoff rate (cm/yr)

Typical permeability rates used in overland flow systems range from 0.15 cm/h to 0.3 cm/h. Runoff rates typically range from 40% in the summer to 80% in the early spring and late fall. Actual runoff should be confirmed by on-site pilot studies.

The following equation can be used to calculate land requirements:

$A_W = 0.01\ Q / R_W$

where

A_W = total land area required (ha)

Q = total annual wastewater production (m^3/yr)

R_W = seasonal water removal rate (cm/yr), as determined from the equation above.

10.4.3.3 Crop Selection

Crops best suited to the overland flow system are grasses with a long growing season, high moisture tolerance and extensive root formation. Reed canary grass has a very high nutrient uptake capacity and yields a good quality hay. Other suitable grasses include ryegrass and tall fescue.

10.4.3.4 Treatment Efficiency

Removal efficiencies of overland flow systems for pathogens such as viruses and indicator organisms are comparable to conventional secondary treatment systems without chlorination. Consequently, chlorination of runoff water may be required by regulatory agencies prior to discharge into streams.

Typical treatment efficiencies are given in Table 10.2

TABLE 10.2 – TYPICAL WASTEWATER CONSTITUENT REDUCTIONS FOR OVERLAND FLOW

Constituent	% Reduction Range*
BOD_5	25 - 95
Suspended Solids	33 - 85
Total N	50 - 82
Total P	0 - 80
Calcium	63 - 85
Zinc	75 - 84

* Reduction from concentrations in untreated wastewater.

10.4.3.5 Monitoring

Monitoring is necessary to maintain loadings within design limits. A routine monitoring program should be established for the following parameters:

a. wastewater application rate (m^3/d);

b. runoff flow rates (m^3/d);

c. wastewater quality, including BOD_5 and COD, suspended solids, total dissolved solids, total nitrogen, total phosphorus, pH, and sodium adsorption ratio (SAR); and

d. runoff water quality according to the analyses summarized in item (c) above.

10.4.4 Rapid Infiltration (RI)

10.4.4.1 Applicability

Rapid infiltration (RI) involves the application of wastewater (usually after some pretreatment) to land by means of basins. The wastewater percolates through the soil, undergoes a variety of physical, chemical and biological reactions and eventually reaches the groundwater. The loss of water via plants or evaporation is minor compared to the loss by percolation. The loading must be intermittent to allow for the restoration of aerobic conditions in the soil. Acceptable salinity, boron, nitrogen and phosphorus levels in the wastewater will be governed by the potential use of the groundwater downstream of the RI site. The permeability of the site is, however, very important to the performance of a RI system. Therefore, the sodium adsorption ratio of the effluent should be below 9.

Optimum site conditions for rapid infiltration (RI) are dependent upon the quantity of wastewater to be treated and the degree of treatment required. Generally, there will be an inverse relationship between maximum wastewater application rate and the degree of treatment. Soil conditions required for a good RI site are a deep uniform sandy loam to loamy sand having the following chemical characteristics;

pH	6.0 - 8.5
Organic Matter	0.5 - 3.0%
Electrical Conductivity	2 dS/m
Sodium Adsorption Ratio	10
Cation Exchange Capacity	10 meq/100 g

Free Ca or Mg CO_3 should be present

Rapid infiltration installations require permeable granular subsurface materials. A minimum of 4 m separation between the water table and the basin bottom before flooding is recommended. The water table should be greater than 1 m below the bottom of the basin during operation. Adverse natural groundwater conditions can be modified by the installation of underdrains and/or recovery wells.

Excessive slopes will restrict the usefulness of a RI site. The maximum slope is that which maintains downward infiltration with no premature lateral discharge. Generally, the maximum slope is 5% unless considerable earth moving is undertaken. Uniform flat topography will reduce construction costs. Ponds or lagoons are not recommended for RI pretreatment methods unless cold weather storage is provided. In areas where facultative lagoons are used for treatment, the lagoons will generally be large enough to provide cold weather storage. However, the infiltration area will have to be large enough to treat the annual wastewater production during the warm weather period. Wastewater from treatment plants with short detention times will retain sufficient heat to allow continuous RI treatment and eliminate the need for storage.

10.4.4.2 *Area and Infiltration Rate*

Prior to site selection the planner must determine the approximate land area required for an RI system. This can be obtained by using sewage flow data and the annual amount of infiltration per unit area. The hydraulic conductivity required to estimate total infiltration can be determined from Table 10.3 and the following calculations. It is then suggested that a factor of 1.5 be applied to the calculated area requirements.

TABLE 10.3 – HYDRAULIC CONDUCTIVITIES OF VARIOUS GRANULAR DEPOSITS

Deposit	Hydraulic Conductivity (cm/s)
Clean, well sorted sand and gravel	10^{-1}
Clean sand, moderately sorted gravel	10^{-2}
Moderately sorted sand and gravel	10^{-3}
Poorly sorted sand and gravel	10^{-4}

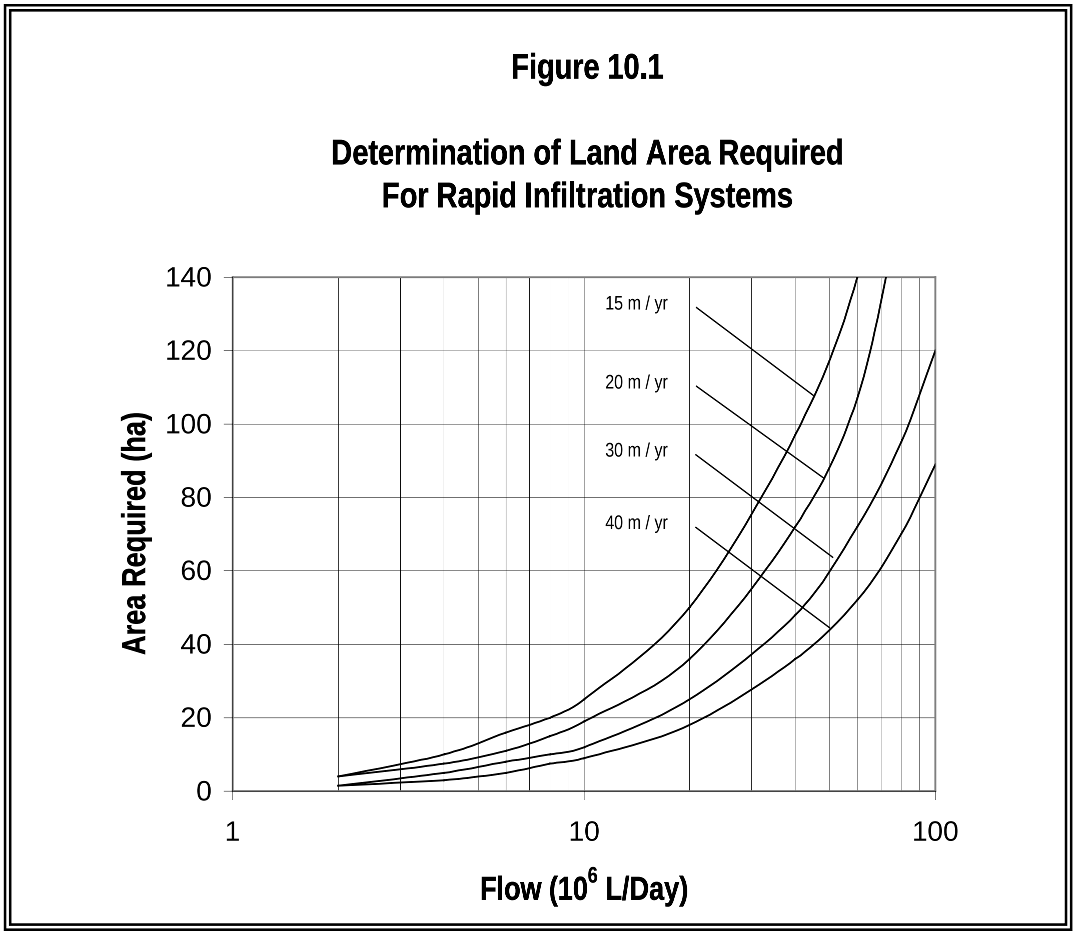
Infiltration capacity is estimated by the following procedure:

a. Estimate site hydraulic conductivity, in cm/s.

b. Determine annual hydraulic loading and convert cm/s to m/yr (multiply by 3.15×10^5).

c. Extrapolate site area (on the y-coordinate of Figure 10.1) using the line most closely representing the estimated hydraulic loading rate determined.

d. Maximum daily infiltration capacity of the site in question can be read off the x-coordinate (Figure 10.1).



The infiltration rate must be confirmed by field testing. Areal requirements for an RI system must include:

a. infiltration basins and dykes;

b. maintenance and laboratory buildings(s);

c. possibly on-site treatment facilities;

d. on-site roads;

e. expansion and emergency use areas;

f. buffer strips.

10.4.4.3 Loading Cycle

Canadian RI systems would likely require an altered loading cycle with respect to seasons because longer resting periods may be required for soil drying and aeration during winter. Decreasing the application rate and increasing the length of the application and resting period are possible means of overcoming the problems of winter application.

Suggested loading cycles are shown in Table 10.4. The values given in this table are considered guidelines. Actual loading cycles should take into account site-specific conditions.

TABLE 10.4 – SUGGESTED HYDRAULIC LOADING CYCLES FOR RAPID INFILTRATION SYSTEMS				
Objective of Preapplication Treatment Period (Days)	Preapplication Treatment Level	Season	Application Period (Days)	Drying (Days)
Maximize infiltration rates of nitrification	Primary	Summer	1 - 2	5 - 7
		Winter	1 - 2	7 - 14
	Secondary	Summer	1 - 3	4 - 5
		Winter	1 - 3	5 - 12
Maximize nitrogen removal	Primary	Summer	1 - 2	10 - 14
		Winter	1 - 2	12 - 18
	Secondary	Summer	7 - 9	10 - 15
		Winter	9 - 12	12 - 18

10.4.4.4 Application Rate

Once the loading rate and loading cycle have been established, the application rate can be calculated. For example, if the hydraulic loading rate is 20 m/annum and the loading cycle is one day of application alternated with seven days of drying, the application rate is as follows:

$$\text{Hydraulic loading rate} \times \frac{\text{(time on + time off)}}{\text{time on}} \times \text{conversion factor annual to daily} = \text{daily application rate}$$

The application rate should be used to determine the maximum depth of the applied wastewater. For instance, if the measured basin infiltration rate is 15 cm/d, the maximum wastewater depth will be the daily application rate minus 15.

In general, maximum wastewater depth should not exceed 46 cm with a preferable maximum depth of 30 cm. If the wastewater depth calculation indicates the recommended maximum will be exceeded, either the loading rate should be decreased or the loading cycle adjusted until the maximum basin depth is acceptable.

10.4.4.5 Monitoring

A monitoring program should provide applied wastewater quality, the quality of groundwater affected by the RI system and, if required, an analysis of the soil affected by the RI system. Several groundwater samples should be collected from sites expected to be influenced by RI and compared with samples from areas not affected by wastewater infiltration.

10.4.5 Runoff

The system shall be designed to prevent surface runoff from entering or leaving the project site.

10.4.6 Fencing and Warning Signs

The project area shall be enclosed with a suitable fence to exclude livestock and discourage trespassing. A vehicle access gate of sufficient width to accommodate mowing equipment should be provided. All access gates should be provided with locks.

Appropriate signs should be provided along the fence around the project boundaries to designate the nature of the facility and advise against trespassing.

10.5 GUIDELINES FOR WASTEWATER IRRIGATION

Treated effluent does not always meet a quality standard that would enable its unrestricted discharge to the receiving environment. For land application, concerns still remain with respect to elevated concentrations of soluble salts, nutrients, and microbiological quality of the treated effluent.

The major difference between municipal wastewater and “high quality irrigation water” is the higher concentration of living and nonliving organic material, nitrogen, phosphorus, and in some instances, higher sodium and salt levels in the municipal wastewater. Wastewater suitability for irrigation is based on a select set of water quality parameters to be tested prior to and during their release. Site acceptability is to be based on pertinent soil and geological properties, topography, hydrology, climate, and zoning and cropping intentions.

In contrast with natural irrigation waters, municipal wastewater has numerous additional health and environmental factors that need to be evaluated to ensure no detrimental impacts occur for their use. Due to the origin, the variety and the often changing quality of wastewater generated by municipalities, it is imperative that municipal wastewaters be tested for a much wider range of water quality parameters than is currently necessary for irrigation with natural waters.

10.5.1 General Health Related Aspects

Potential human pathogens of concern found in domestic wastewater may be grouped into four categories:

- Bacteria,
- Protozoan parasites,
- Helminth parasites, and
- Viruses.

There is no single wastewater treatment process which will remove all pathogenic microorganisms. Many potentially disease causing microorganisms will therefore continue to exist in municipal wastewater. The types and amounts of these microorganisms will vary greatly with the treatment process or combination of the processes utilized.

Therefore for wastewater irrigation to be authorized, the minimum treatment requirement is secondary treatment followed by at least six months storage. The potentially harmful microorganisms are killed over a period of time by exposure to strong sunlight, high temperatures, and dry weather that may allow their direct application for sites with restricted access. However, where unrestricted access is allowed (i.e. golf courses) the minimum standards are generally not acceptable and faecal coliform levels in the irrigation water should approach 0 counts per 100 mL.

10.5.2 Minimum Quality Standards and Monitoring Requirements

The treated effluent quality and monitoring requirements for wastewater irrigation shall meet the standards specified in table 10.5. These standards represent the minimum acceptable levels and can therefore be adjusted to more stringent requirements by the governing regulatory authority on a case by case basis.

TABLE 10.5 MINIMUM QUALITY STANDARD FOR RECLAIMED WATER AND MONITORING REQUIREMENTS

Parameter	Points of Measurement	Requirements	Minimum Monitoring
Sodium Absorption Ratio	Prior to discharge to Irrigation System	<4 unrestricted, 4-9 restricted, >9 unacceptable	Grab* - once/month
Electrical Conductivity	Prior to discharge to Irrigation System	<1dS/m unrestricted >9 dS/m unacceptable	Grab* - once/month
Treated Effluent BOD	Prior to discharge to Irrigation System	Not to exceed 20 mg/L	Grab* - twice/month
Treated Effluent TSS	Prior to discharge to Irrigation System	Not to exceed 20 mg/L	Grab* - twice/month
Daily Treated Effluent Flow	Prior to discharge to Irrigation System	m^3/day	Daily
Total Coliform	Prior to discharge to Irrigation System	Not to exceed <1000/100ml	Grab* - twice/month
Faecal Coliform	Prior to discharge to Irrigation System	Not to exceed <200/100ml	Grab* - twice/month
Nitrate and Total Nitrogen	Prior to discharge to Irrigation System	See section 10.5.3	Grab Sample Monthly
Phosphorus	Prior to discharge to Irrigation System	See section 10.5.3	Grab Sample Monthly
Potassium	Prior to discharge to Irrigation System	See section 10.5.3	Grab Sample Monthly
pH	Prior to discharge to Irrigation System	6.5 to 9.5	Grab – twice/week

*Grab sample would suffice if storage is provided; composite sample is required if storage is not provided.

10.5.3 Nutrients

One of the main advantages of using wastewater irrigation is that it may often enhance the fertility of the lands to which it is applied. The following nutrients should be analyzed and reported as part of the initial wastewater characterization:

a) Nitrogen can be evaluated in a number of different forms. Regular evaluation of nitrogen by analyzing for NO_3-N, NH_3-N, NO_2-N, and TKN should be conducted. The typical range for total nitrogen of most municipal wastewater is 10 to 20 mg/L. Wastewater that consists of a total nitrogen concentration within the typical range can easily be assimilated by the growing crop without harmful health or environmental concerns provided wastewater it is not applied in quantities that exceed the field moisture capacity and it is applied during active crop growing season.

b) Phosphorus is to be evaluated as total phosphorus. The typical range of total phosphorus in municipal wastewater is between 2 and 6 mg/L. Since phosphorus is effectively immobilized in most soils at shallow depths, the potential for adverse impacts on groundwater quality is remote.

c) Potassium is another major nutrient present in wastewater of value for crop production that should be evaluated. The typical range for potassium in most municipal wastewaters is 5 to 40 mg/L Such levels are normally assimilated by crops and are thus not considered to be an environmental or health risk.

10.5.4 Metals

Uptake of harmful amounts of toxic heavy metals by plants is not considered a potential risk in use of municipal wastewater, as most metals are removed from the wastewater in the primary treatment process. However as a precautionary measure, all wastewater should be initially tested for the following metals to ensure levels are below recommended CCME water quality standards prior to granting authorization for irrigation: Cadmium (Cd), Chromium (Cr), Cobalt (Co), Copper (Cu), Lead (Pb), Mercury (Hg), Nickel (Ni), Zinc (Zn).

10.5.5 Site Suitability

Land classification and other relevant site characterization activities are generally performed after completing the initial wastewater characterization, and the results indicate that the wastewater is suitable for irrigation.

In general, development of a municipal wastewater irrigation system must:
a) Provide safe long term management of the landbase;
b) Ensure public acceptance; and
c) Protect community's capital cost of the project.

10.5.5.1 Soil

Soil-related information must be provided for each different land classification unit delineated within the area evaluated where the size of that unit is 10 acres or greater. Therefore, if a land area was extremely complex, 16 separate land classification units could exist per quarter section of land classified, thus requiring soil sampling at 16 separate locations within the quarter section. All soil sampling locations must be clearly identified and mapped. The boundaries of the specific quarter section in which any sampling is conducted and the boundaries of each land classification unit identified, must be clearly illustrated and included with mapping of the sampling locations. The scale of map used to portray this information must be at least a 1:10000 scale. The soil sampling to be conducted at each sampling site location must be conducted in the following manner:

a) Soil samples are to be collected from the following depth intervals, at each sampling location: 0 to 15cm 15 to 30cm 30 to 60cm 60 to 100cm to 150cm;

b) Each soil sample is to be at least 1 kg in weight.;

c) In addition to collecting the soil samples, the depth of the groundwater table below ground surface at each ample site location should be denoted;

d) The soil samples from each sampling location shall then be forwarded to a laboratory for analysis.

10.5.5.1.1 Soil Parameters

The following analysis should be completed on all soil samples collected on site:

a) A soil particle size distribution. Results of the analysis should indicate the percentage of sand, silt, and clay and denote the appropriate textural classification as per the Canadian System of Soil Classification.

b) Standard field percolation tests or ring/cylindrical infiltrometer tests should be completed for each irrigable land class unit defined in the irrigable land classification process. These tests will ultimately serve to assist with evaluation of system design needs, and provide a basis for establishing acceptable limits in the regulation of irrigation rates, frequencies, and application duration.

c) Soil pH.

d) Soil Sodium Adsorption Ratio.

e) Major cations.

f) Major anions.

g) Plant available nitrogen.

h) NO_3 and NH_4.

10.5.5.2 Topography

Topographic features such as relief, site and shape of fields, soil type and texture, brush/tree cover, and surface drainage features must be evaluated for site suitability. Land may not be considered suitable for irrigation due to one or a combination of factors such as: steep slopes, hummody relief, brush/tree cover, small or irregular shape, sloughs, wetlands, and rough broken topography.

10.5.5.3 Other Requirements

Other information in the initial site assessment process must include:

a) Location and mapping of any surface water courses, water bodies, or domestic wells located on or within 150m of the wastewater development site.
b) Location and mapping of any residential dwelling on or within 150m of the wastewater development site.
c) Location and mapping of all public roads, highways, or other public corridors on or within 30m of the wastewater development site.

These site-specific requirements are intended to provide baseline information on all sites to be developed for wastewater irrigation purposes. The knowledge is intended to assist in evaluating potential impacts of long-term wastewater irrigation on the land base over time.

10.5.6 Assessment of System Design

Wastewater irrigation system design is undertaken once water quality assessment and land suitability assessment are affirmed. The design integrates wastewater quality with land base limitations and restrictions that relate to cropping, climate, application, and public acceptance issues.

10.5.6.1 Climate

There are two climate factors that must be considered to ensure an effective wastewater irrigation system design. These factors are defined as follows:

a) Wastewater irrigation applications are restricted to the period of May 1st to September 30th. Storage must be provided for the seventh month period when wastewater irrigation application is not authorized.

b) Wastewater irrigation is a suitable disposal option only in regions where the additional moisture applied can be utilized for improved crop production.

10.5.6.2 Land Area

There are a number of land-related factors relevant to irrigation system design that must be considered. The factors are defined as follows:

a) Specific irrigation design features must be provided that will avoid application of irrigation wastewater to any non-irrigable land areas

b) The amount of land and equipment required will depend upon the mean annual consumptive use of water by plants, natural precipitation from April through September, an irrigation efficiency factor, and a deep leaching requirement.

c) The land area to be accumulated must also allow for any buffer zones or setback limits that apply on or around land areas where wastewater irrigation is to be undertaken. Setbacks and buffer zones that will apply are tabulated below:

TABLE 10.6 SETBACK REQUIREMENTS

Parameter	Requirements
Adjacent Properties	Buffer Zone of 15 M between irrigated land and adjacent property owners.*
Adjacent Dwellings	Buffer Zone of a minimum of 50 M between irrigated land and any occupied dwellings.*
Public Rights of Way	Buffer Zone of a minimum of 25 M between irrigated land and any public right of way.
Potable Water Wells	Buffer Zone of a minimum of 30 M between irrigated land and any potable water well.
Watercourses, Rivers, Streams, etc.	Buffer Zone of a minimum of 20 M between irrigated land and any Watercourse.**

* Distance maybe reduced with the signed permission of adjacent property owner.

** Watercourses used for golf course irrigation area exempt from the buffer zone. Distance maybe reduced depending on the actual quality of irrigation water.

10.5.6.3 Crop Considerations

Only certain crops are deemed suitable for production on lands to be irrigated with municipal wastewater. The current authorized crops include: forages, course grains, turf, and oil seeds.

10.5.6.4 Wastewater Storage Ponds

The design of any storage reservoir required to retain wastewater during periods of restricted irrigation must meet current design criteria as described in Chapter 7.

10.5.7 System Operation

Operating conditions and requirements for the system must be described prior to receiving approval. Due to the great variation in waste concentration, soils, and climate, no attempt will be made to elaborate further on irrigation management in this document. Specific operational requirements will be stated in the certificate of approval.

11.1 GENERAL

Sludge handling and disposal must be considered as an integral part of any complete sewage treatment system. The following is a summary of the sludge handling and disposal options and the various process and treatment requirements best suited to the option selected. Re-use and recovery alternatives of sludge by-products are also included as disposal options.

Plans and specifications for sludge handling disposal must be incorporated in the design of all sewage treatment facilities.

11.2 PROCESS SELECTION

The selection of sludge handling unit processes should be based upon at least the following considerations:

a. Local land use;

b. system energy requirements;

c. Cost effectiveness of sludge thickening and dewatering;

d. Equipment complexity and staffing requirements;

e. Adverse effects of heavy metals and other sludge components upon the unit processes;

f. Sludge digestion or stabilization requirements;

g. Side stream or return flow treatment requirements (e.g., digester or sludge storage facilities supernatant, dewatering unit filtrate, wet oxidation return flows);

h. Sludge storage requirements;

i. Methods of ultimate disposal; and

j. Back-up techniques of sludge handling and disposal.

11.3 SLUDGE CONDITIONING

11.3.1 Chemical Conditioning

11.3.1.1 Chemical Requirements

Chemical conditioning methods involve the use of organic or inorganic flocculants to promote the formation of a porous, free draining cake structure. The ranges of some chemical conditioning requirements are outlined in Table 11.1.

TABLE 11.1 - SOME CHEMICAL CONDITIONING REQUIREMENTS

SLUDGE	$FeCl_3$ (kg/tonne DRY SOLIDS)	$Ca(OH)_2$ (kg/tonne DRY SOLIDS)	POLYMERS (kg/tonne DRY SOLIDS)
RP	10 - 30	0 - 50	1.5 - 2.5
R(P + TF)	30 - 60	0 - 150	2 - 5
R(P + AS)	40 - 80	0 - 150	3 - 7.5
AS	60 - 100	50 - 1500	4 - 12.5
DP	20 - 30	30 - 80	1.5 - 4
D(P + TF)	40 - 80	50 - 150	3 - 7.5
D(P + AS)	60 - 100	50 - 150	3 - 10

KEY : R = RAW; P = PRIMARY; TF = TRICKLING FILTER; AS = ACTIVATED SLUDGE; D = DIGESTED

11.3.1.2 Laboratory Testing

The selection of the most suitable chemical(s) and the actual dosage requirements for sludge conditioning shall be determined by full-scale testing.

Laboratory testing should, however, only 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.

11.3.1.3 Conditioning Chemicals

11.3.1.3.1 General

With most thickening operations and with belt filter press dewatering operations the most commonly used chemicals are polymers. For dewatering by vacuum filtration, ferric salts, often in conjunction with lime, are most commonly used, although with centrifuge dewatering, chemical conditioning using polymers is most prevalent, with metal salts being avoided mainly due to corrosion problems. The ultimate disposal methods may also have an effect on the choice of conditioning chemicals. For instance, lime and ferric compounds should be avoided with incineration options.

11.3.1.3.2 Iron or Aluminum Salts

Most raw sludges can be filtered with ferric salts alone, although digested sludge will require an addition of lime with the ferric salt. The lime:ferric chloride ratio is typically 3:1 to 4:1 for best results. If metallic salts are used without lime, the resulting low pH sludge will be highly corrosive to carbon steel and shall require materials such as plastic, stainless steel, or rubber for proper handling.

11.3.1.3.3 Lime

Hydrated limes, both the high calcium and dolomitic types, can be used for sludge conditioning in conjunction with metal salts or alone.

11.3.1.3.4 Polymers

Polymers used for sludge conditioning are long-chain water-soluble organic molecules of high molecular weight. They are used in wastewater suspensions to cause flocculation through adsorption. Equipment for polymer addition must be able to withstand potential corrosion.

11.3.1.3.5 Chemical Feed System

The chemical feed system shall be paced at the rate of sludge flow to the dewatering unit. The chemical feed system should be either close to the dewatering unit or controllable from a point near the dewatering unit. Sufficient mixing shall be provided so as to disperse the conditioner throughout the sludge. The chemical feed rates should allow for at least a 10:1 range of chemical flow to the dewatering unit.

11.3.2 Heat Conditioning

11.3.2.1 General

Heat conditioning of sludge consists of subjecting the sludge to high levels of heat and pressure. Heat conditioning can be accomplished by either a non-oxidative or oxidative system. Heat conditioning high temperatures cause hydrolysis of the encapsulated water-solids matrix and lysing of the biological cells. The hydrolysis of the water matrix destroys the gelatinous components of the organic solids and thereby improves the water-solids separation characteristics.

11.3.2.2 Operating Temperatures and Pressures

Typical operating temperatures range from 150 to 260°C. Operating pressures range from 1100 to 2800 kPa. Typical sludge detention times vary between 15 and 60 minutes.

11.3.2.3 Increase in Aeration Tank Organic Loading

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 supernatant is returned to the aeration system. This is 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.

11.3.2.4 Design Considerations

11.3.2.4.1 Materials

Heat conditioning results in the production of extremely corrosive liquids requiring the use of corrosion-resistant materials for the liquid handling.

11.3.2.4.2 Sludge Grinding

Sludge grinders shall be provided to macerate the sludge to a particle size less than 6 mm to prevent fouling of the heat exchangers.

11.3.2.4.3 Feed Pumps

Feed pumps shall be capable of discharging sludge at pressures of 1400 to 2800 Kpa and must be resistant to abrasion.

11.3.2.4.4 Heat Exchangers

The efficiency of the heat exchangers is dependent on the transfer coefficients and the temperature differences of the incoming and outgoing sludges.

11.3.2.4.5 Reaction Vessel

The reaction vessel shall be of sufficient volume to provide for a sludge detention time of 15 to 60 min. The detention time depends on the sludge characteristics, temperature and the level of hydrolysis required.

11.3.2.4.6 Hot Water Recirculation Pump

The hot water recirculation pump shall be capable of handling hot water at a temperature of 25 to 65°C.

11.3.2.4.7 Odour Control

Heat conditioning, particularly the non-oxidative process, can result in the production of odorous gases in the decant tank. If ultimate sludge disposal is via incineration, these gases can be incinerated in the upper portion of the furnace. If incineration is not a part of the sludge handling process, a catalytic or other type of oxidating unit should be used.

11.3.2.4.8 Solvent Cleaning

Scale formation in the heat exchangers, pipes and reaction vessel require acid washing equipment to be provided.

11.3.2.4.9 Piping

All the high pressure piping for the sludge heat conditioning system shall be tested at a pressure of 3500 kPa. Low pressure piping shall be tested at 1.5 times the working pressure or 1400 kPa, whichever is greater.

11.3.2.4.10 Decant Tank

The decant tank functions as a storage and sludge consolidation unit. The tank should be covered and provided with venting and a deodorization arrangement. The tank should be designed using loadings of 245 $kg/m^2 \cdot d$ for primary sludge and 145 $kg/m^2 \cdot d$ for biological sludges. The underflow will range from 10 to 15 percent TS.

11.3.2.5 Laboratory Testing

Since process efficiency is dependent on achieving a degree of solubilization (hydrolysis) that reduces the specific resistant to an acceptable range, batch testing with a laboratory autoclave should be employed. This procedure permits accurate control of the time and temperature functions affecting the level of hydrolysis. The level of solubilization is determined from the loss of TSS during heat treatment.

11.3.3 Addition of Admixtures

Another common form of physical conditioning is the addition of admixtures such as fly ash, incinerator ash, diatomaceous earth, or waste paper. These conditioning techniques are most commonly used with filter presses or vacuum filters. The admixtures when added in sufficient quantities produce a porous lattice structure in the sludge which results in decreased compressibility and improved filtering characteristics. When considering such conditioning techniques, the beneficial and detrimental effects of the admixture on such parameters as overall sludge mass, calorific value, etc., must be evaluated along with the effects on improved solids content.

11.4 SLUDGE THICKENING

11.4.1 General

11.4.1.1 Applicability

As the first step of sludge handling, the need for sludge thickeners to reduce the volume of sludge should be considered.

The design of thickeners (gravity, dissolved-air flotation, centrifuge and others) should consider the type and concentration of sludge, the sludge stabilization processes, the method of ultimate sludge disposal, chemical needs and the cost of operation. Particular attention should be given to the pumping and piping of the concentrated sludge and possible onset of anaerobic conditions. Sludge thickening to at least 5% solids prior to transmission to digesters should be considered.

Wherever possible, pilot-plant and/or bench-scale data should be used for the design of sludge thickening facilities. With new plants, this may not always be possible and, in such cases, empirical design parameters must be used. The following subsections outline the normal ranges for the design parameters of such equipment.

In considering the need for sludge thickening facilities, the designer should evaluate the economics of the overall treatment processes, with and without facilities for sludge water content reduction. This evaluation should consider both capital and operating costs of the various plant components and sludge disposal operations affected.

11.4.1.2 Multiple Units

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

11.4.1.3 Thickener Location

Sludge thickening can be employed in the following locations in a sewage treatment plant:

- prior to digestion for raw primary, excess activated sludge or mixed sludges;
- prior to dewatering facilities;
- following digestion for sludges or supernatant; or
- following dewatering facilities for concentration of filtrate, decant, centrate, etc.

Where thickeners are to be housed, adequate ventilation shall be provided.

11.4.2 Thickening Methods and Performance With Various Sludge Types

The commonly employed methods of sludge thickening and their suitability for the various types of sludge are shown in Table 11.2 In selecting a design figure for the thickened sludge concentration, the designer should keep in mind that all thickening devices are adversely affected by high Sludge Volume Indices (SVI's) and benefitted by low SVI's in the influent activated sludges. The ranges of thickened sludge concentrations given in Table 11.2 assume an SVI of approximately 100.

TABLE 11.2 - SLUDGE THICKENING METHODS AND PERFORMANCE WITH VARIOUS SLUDGE TYPES

THICKENING METHOD	SLUDGE TYPE	PERFORMANCE EXPECTED
Gravity	Raw Primary	Good, 8-10% Solids
	Raw Primary and Waste Activated	Poor, 5-8% Solids
	Waste Activated	Very Poor, 2-3% Solids (Better results reported for oxygen excess activated sludge)
	Digested Primary Digested Primary and Waste Activated	Very Good, 8-14% Solids Poor, 6-9% Solids
Dissolved Air Flotation	Waste Activated (Not generally used for other sludge types)	Good, 4-6% Solids and $\geq$ 95% Solids Capture With Flotation Aids.
Centrifugation	Waste Activated	8-10% and 80-90% Solids Capture with Basket Centrifuges; 4-6% and 80-90% Solids Capture with Disc-nozzle Centrifuges; 5-8% and 70-90% Solids Capture with Solid Bowl Centrifuges

11.4.3 Sludge Pretreatment

Wherever thickening devices are being installed, special consideration must be given to the need for sludge pretreatment in the form of sludge grinding to avoid plugging pumps, lines and thickening equipment. Sludge conditioning by chemical conditioning is also considered as a type of pretreatment.

11.4.4 Gravity Thickening

11.4.4.1 Process Application

Gravity thickening is principally used for primary sludge, and mixtures of primary and waste activated sludges, with little use for waste activated sludges alone. Due to the better performance of other methods for waste activated sludges, gravity thickening has limited application for such sludges.

11.4.4.2 Design Criteria

11.4.4.2.1 Tank Shape

The gravity thickener shall be circular in shape.

11.4.4.2.2 Tank Dimensions

Typical maximum tank diameters should range between 21 and 24 m. Sidewater depth shall be between 3 and 3.7 m.

11.4.4.2.3 *Floor Slope*

The acceptable range for gravity sludge thickener floor slopes is 2:12 to 3:12.

11.4.4.2.4 *Solids Loading*

The type of sludge shall govern the design value for solids loading to the gravity thickener. Table 11.3 outlines recommended solids loading values.

TABLE 11.3 - SOLIDS LOADING ON GRAVITY THICKENERS FOR VARIOUS SLUDGE TYPES

TYPE OF SLUDGE	SOLIDS LOAD ($kg/m^2 \cdot day$) ACCEPTABLE RANGE
Primary	95 - 120
Waste Activated	12 - 40
Modified Activated	50 - 100
Trickling Filter	40 - 50

Solids loading for any combination of primary sludge and waste activated sludge shall be based on a weighted average of the above loading rates.

Use of metal salts for phosphorus removal may affect the solids loading rates.

11.4.4.2.5 *Dilution*

Improved thickening is achieved by diluting sludge to 0.5 to 1% solids because that dilution reduces the interface between the settling particles. Primary sewage effluent or secondary effluent may be utilized to dilute sludge before thickening.

11.4.4.2.6 *Hydraulic Overflow Rate*

The hydraulic overflow rate shall be kept sufficiently high to prevent septic conditions from developing in the thickener. The acceptable ranges for overflow rates are as follows:

Primary Sludge	0.28-0.38 $L/m^2 \cdot s$
Secondary Sludge	0.22-0.34 $L/m^2 \cdot s$
Mixture	0.25-0.36 $L/m^2 \cdot s$

11.4.4.2.7 *Sludge Volume Ratio*

The sludge volume ratio (SVR) is defined as the volume of the sludge blanket divided by the daily volume of sludge (underflow) pumped from the thickener. Though deeper sludge blankets and longer SVR are desirable for maximum concentrations, septic conditions due to anaerobic biodegradation on warmer months limit the upper values of SVR to about 2 days.

	Warmer Months	Colder Months
Recommended SVR values (days)	0.3 to 1	0.5 to 2

11.4.4.2.8 Hydraulic Retention Time

A minimum of 6 hr. detention of liquid is required. For maximum compaction of the sludge blanket, 24 hrs. is the recommended time required.

During peak conditions, the retention time may have to be shortened to keep the sludge blanket depth below the overflow weirs, thus, preventing excessive solids carry-over.

11.4.4.2.9 Sludge Underflow Piping

The length of suction lines should be kept as short as possible. Consideration should be given to the use of dual sludge withdrawal lines.

11.4.4.2.10 Chemical Conditioning

Provision should be made for the addition of conditioning chemicals into the sludge influent lines (polymers, ferric chloride or lime are the most likely chemicals to be used to improve solids capture).

11.4.4.2.11 Mechanical Rake

The mechanical rake should have a tip speed of 50 to 100 mm/s. The rake shall be equipped with hinged-lift mechanisms when handling heavy sludges such as lime treated primary sludge. The use of a surface skimmer is recommended.

11.4.4.2.12 Overflow Handling

The normal quality of thickener overflow (also known as thickener overhead or supernatant) is about the same as raw sewage quality. Consequently, returning the overflow to primary settling tank or aeration tank should not present any operational problem.

Direct recycling of thickener overflow to the grit chamber, primary settling tank, trickling filter, RBC or aeration tank is permitted. The supernatant shall not be discharged into the secondary settling tank, disinfection tank, sewer outfall, or receiving water.

11.4.5 Air Flotation

11.4.5.1 Applicability

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. In general, air flotation thickening can be employed whenever particles tend to float rather, than sink. These procedures are also applied if the materials have a long subsidence period and resist compaction for thickening by gravity.

The advantages of air flotation compared with gravity thickeners for excess activated sludges include its reliability, production of higher sludge concentrations, and better solids capture. Its disadvantages include the need for greater operating skill and higher operating costs.

11.4.5.2 Pilot Scale Testing

Experience has shown that flotation operations cannot be designed on the basis of purely mathematical formulations or by the use of generalized design parameters, and therefore some bench-scale and/or pilot-scale testing will be necessary.

11.4.5.3 Design Parameters

The following design parameters are given only as a guide to indicate the normal range of values experienced in full-scale operations.

11.4.5.3.1 Recycle Ratio

The recycle ratio varies with suppliers and typically falls between 0 and 500% of the influent flow. Recycled flows may be pressurized up to 520 kPa.

11.4.5.3.2 Air to Solids Weight Ratio

Typical air to solids weight ratios shall be between 0.02 and 0.05.

11.4.5.3.3 Feed Concentration

Feed concentration of activated sludge (including recycle) to the flotation compartment should not exceed 5000 mg/L.

11.4.5.3.4 Hydraulic Feed Rate

Where the hydraulic feed rate includes influent plus recycle, the flotation units shall be designed hydraulically to operate in the range of 0.3 to 1.5 L/m^2·s. A maximum hydraulic loading rate of 0.5 L/m^2·s shall be adhered to when no coagulant aids are used to improve flotation. The feed rate should be continuous rather than on-off.

11.4.5.3.5 Solids Loading

Without any addition of flocculating chemicals, the solids loading rate for activated sludge to a flotation unit should be between 40 and 100 kg/m^2·d. With the proper addition of flocculating chemicals, the solids loading rate may be increased to 240 kg/m^2·d. These loading rates will generally produce a thickened sludge of 3 to 5 percent total solids.

11.4.5.3.6 Chemical Conditioning

Chemicals used as coagulant aids shall be fed directly to the mixing zone of the feed sludge and recycle flow.

11.4.5.3.7 Detention Time

Detention time is not critical provided particle rise rate is sufficient and horizontal velocity in the unit does not produce scouring of the sludge blanket.

11.4.5.4 Thickened Sludge Withdrawal

The surface skimmer shall move thickened sludge over the dewatering beach into the sludge hopper. Either positive displacement, or centrifugal pumps which will not air bind should be used to transfer sludge from the hopper to the next phase of the process.

In selecting pumps, the maximum possible sludge concentrations should be taken into consideration.

11.4.5.5 Bottom Sludge

A bottom collector to move draw off settled sludge into a hopper must be provided. Draw off from the hopper may be by gravity or pumps.

11.4.6 Centrifugation

11.4.6.1 Types of Centrifuges

Three types of centrifuges may be utilized for sludge thickening. These include the solid bowl conveyor, disc-nozzle and basket centrifuges.

11.4.6.2 Applicability

To date, there has only been limited application of centrifuges for sludge thickening, despite their common use for sludge dewatering. As thickening devices, their use has been generally restricted to excess activated sludges.

In the way of general comments, the following are given:

- centrifugal thickening operations can have substantial maintenance and operating costs;

- where space limitations, or sludge characteristics make other methods unsuitable, or where high-capacity mobile units are needed, centrifuges have been used; and

- thickening capacity, thickened sludge concentration and solids capture of a centrifuge are greatly dependent on the SVI of the sludge.

11.4.6.3 Solids Recovery

The most suitable operating range is generall 85 – 95% solids recovery.

11.4.6.4 Polymer Feed Range

A polymer feed range of 0 to 4.0 g/kg of dry solids is generally acceptable.

11.5 ANAEROBIC SLUDGE DIGESTION

11.5.1 General

11.5.1.1 Applicability

Anaerobic digestion may be considered beneficial for sludge stabilization when the sludge volatile solids content is 50% or higher and if no inhibitory substances are present or expected. Anaerobic digestion of primary sludge is preferred over activated sludge because of the poor solids-liquid separation characteristics of activated sludges. Combining primary and secondary sludges will result in settling characteristics better than activated sludge but less desirable than primary alone. Chemical sludges containing lime, alum, iron, and other substances can be successfully digested if the volatile solids content remains high enough to support the biochemical reactions and no toxic compounds are present. If an examination of past sludge characteristics indicates wide variations in sludge quality, anaerobic digestion may not be feasible because of its inherent sensitivity to changing substrate quality. Table 11.4 lists sludges which are suitable for anaerobic digestion.

TABLE 11.4 - SLUDGES SATISFACTORY FOR ANAEROBIC DIGESTION
PRIMARY AND LIME
PRIMARY AND FERRIC CHLORIDE
PRIMARY AND ALUM
PRIMARY AND TRICKLING FILTER
PRIMARY, TRICKLING FILTER, AND ALUM
PRIMARY AND WASTE ACTIVATED
PRIMARY, WASTE ACTIVATED, AND LIME
PRIMARY, WASTE ACTIVATED, AND ALUM
PRIMARY, WASTE ACTIVATED, AND FERRIC CHLORIDE
PRIMARY, WASTE ACTIVATED, AND SODIUM ALUMINATE

The advantages offered by anaerobic digestion include:

- Excess energy over that required by the process is produced. Methane is produced and can be used to heat and mix the reactor. Excess methane gas can be used to heat space or produce electricity, or as engine fuel.
- The quantity of total solids for ultimate disposal is reduced. The volatile solids present are converted to methane, carbon dioxide, and water thereby reducing the quantity of solids. About 30 to 40% of the total solids may be destroyed and 40 to 60% of the volatile solids may be destroyed.
- The product is a stabilized sludge that may be free from strong or foul odours and can be used for land application as ultimate disposal because the digested sludge contains plant nutrients.
- Pathogens are destroyed to a high degree during the process. Thermophilic digestion enhances the degree of pathogen destruction.
- Most organic substances found in municipal sludge are readily digestible except lignins, tannins, rubber, and plastics.

The disadvantages associated with anaerobic digestion include:

- The digester is easily upset by unusual conditions and erratic or high loadings and very slow to recover.
- Operators must follow proper operating procedures.
- Heating and mixing equipment are required for satisfactory performance.
- Large reactors are required because of the slow growth of methanogens and required solid retention times (SRT's) of 15 to 20 days for a high-rate system. Thus capital cost is high.

- The resultant supernatant sidestream is a strong waste stream that greatly adds to the loading of the wastewater plant. It contains high concentrations of BOD, COD, suspended solids and ammonia nitrogen.

- Cleaning operations are difficult because of the closed vessel. Internal heating and mixing equipment can become major problems as a result of corrosion and wear in harsh inaccessible environments.

- A sludge poor in dewatering characteristics is produced.

- The possibility of explosion as a result of inadequate operation and maintenance, leaks, or operator carelessness exists.

- Gas line condensation or clogging can cause major maintenance problems.

11.5.1.2 Digestion Tanks and Number of Stages

With anaerobic 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. Single stage digesters will generally not be satisfactory due to the usual need for sludge storage, and effective supernating. They will be considered, however, where the designer can show that the above concerns can be satisfied and that alternate means of sludge processing or emergency storage can be used in the event of breakdown.

11.5.1.3 Access Manholes

At least two, 1-meter diameter access manholes should be provided in the top of the tank in addition to the gas dome. There should be stairways to reach the access manholes. A separate sidewall manhole shall be provided. The opening should be large enough to permit the use of mechanical equipment to remove grit and sand. This manhole shall be located near the bottom of the sidewall. All manholes shall be provided with gas-tight and water-tight covers.

11.5.1.4 Safety

Non-sparking tools, safety lights, rubber-soled shoes, safety harness, gas detectors for inflammable and toxic gases and at least two self-contained breathing units shall be provided for workers involved in cleaning the digesters.

Necessary safety facilities shall be included where sludge gas is produced. All tank covers shall be provided with pressure and vacuum relief valves and flame traps together with automatic safety shut-off valves. Water seal equipment shall not be installed.

11.5.2 Field Data

Wherever possible, such as in the case of plant expansions, actual sludge quantity data should be considered for digester design. Often, due to errors introduced by poor sampling techniques, inaccurate flow measurements or unmeasured sludge flow streams, the sludge data from existing plants may be unsuitable for use in design. Therefore, before sludge data is used for design, it should be assessed for its accuracy.

11.5.3 Typical Sludge Qualities and Generation Rates for Different Unit Processes

When reliable data are not available, the sludge generation rates and characteristics given in Table 11.5 may be used.

11.5.4 Solids Retention Time

The minimum solids retention time for a low rate digester shall be 30 days. The minimum solids retention time of a high rate digester shall be 15 days.

11.5.5 Design of Tank Elements

11.5.5.1 Digester Shape

Anaerobic digesters are generally cylindrical in shape with inverted conical bottoms. Heat loss from digesters can be minimized by choosing a proper depth-diameter ratio such that the total surface area is the least for a given volume. A cylinder with diameter equal to depth can be shown to be the most economical shape from heat loss viewpoint. However, structural requirements and scum control aspects also govern the optimum depth-diameter ratio.

11.5.5.2 Floor Shape

To facilitate draining, cleaning and maintenance, the following features are desirable:

The tank bottom should slope to drain toward the withdrawal pipe. For tanks equipped with mechanisms for withdrawal of sludge, a bottom slope not less than 1:12 (vertical:horizontal) is recommended. Where the sludge is to be removed by gravity alone, 1:4 slope is recommended.

11.5.5.3 Depth and Freeboard

For those units proposed to serve as supernatant development tanks, the depth should be sufficient to allow for the formation of a reasonable depth of supernatant liquor. A minimum water depth of 6 meters is recommended. The acceptable range for sidewater depth is between 6 and 14 m.

The freeboard provided must take into consideration the type of cover and maximum gas pressure. For floating covers, the normal working water level in the tank under gas pressure is approximately 0.8 m below the top of the wall, thus providing from 0.5 to 0.6 m of freeboard between the liquid level and the top of the tank wall. For fixed flat slab roofs, a freeboard of 0.3 to 0.6 m above the working liquid level is commonly provided. For fixed conical or domed roofs, the freeboard between the working liquid level and the top of the wall inside the tank can be reduced to less than 0.3 m.

TABLE 11.5 - TYPICAL SLUDGE QUALITIES AND GENERATION RATES FOR DIFFERENT UNIT PROCESSES

UNIT PROCESS	LIQUID SLUDGE	SOLIDS CONCENTRATION		VOLATILE SOLIDS	DRY SOLIDS	
	(L/m^3)	RANGE (%)	AVERAGE (%)	(%)	(g/m^3)	(g/cap·d)
PRIMARY SEDIMENTATION WITH ANAEROBIC DIGESTION						
UNDIGESTED (NO P REMOVAL)	2.0	(3.5-8)	5.0	65	120	55
UNDIGESTED (WITH P REMOVAL)	3.2	(3.5-7)	4.5	65	170	77
DIGESTED (NO P REMOVAL)	1.1	(5-13)	6.0	50	75	34
DIGESTED (WITH P REMOVAL)	1.6	(5-13)	5.0	50	110	50
PRIMARY SEDIMENTATION AND CONVENTIONAL ACTIVATED SLUDGE WITH ANAEROBIC DIGESTION						
UNDIGESTED (NO P REMOVAL)	4.0	(2-7)	4.5	65	180	82
UNDIGESTED (WITH P REMOVAL)	5.0	(2-6.5)	4.0	60	220	100
DIGESTED (NO P REMOVAL)	2.0	(2-6)	5.0	50	115	52
DIGESTED (WITH P REMOVAL)	3.5	(2-6)	4.0	45	150	68
CONTACT STABILIZATION AND HIGH RATE WITH AEROBIC DIGESTION						
UNDIGESTED (NO P REMOVAL)	15.5	(0.4-2.8)	1.1	70	170	77
UNDIGESTED (WITH P REMOVAL)	19.1	(0.4-2.8)	1.1	60	210	95
DIGESTED (NO P REMOVAL)	6.1	(1-3)	1.9	70	115	52
DIGESTED (WITH P REMOVAL)	8.1	(1-3)	1.9	60	155	70
EXTENDED AERATION WITH AERATED SLUDGE HOLDING TANK						
WASTE ACTIVATED (NO P REMOVAL)	10.0	(0.4-1.9)	0.9	70	90	41
WASTE ACTIVATED (WITH P REMOVAL)	13.3	(0.4-1.9)	0.9	60	120	55
SLUDGE HOLDING TANK (NO P REMOVAL)	4.0	(0.4-5.0)	2.0	70	80	36
SLUDGE HOLDING TANK (WITH P REMOVAL)	5.5	(0.4-4.5)	2.0	60	110	50

NOTE:

1. (L/cu. m) DENOTES LITRES OF LIQUID SLUDGE PER CUBIC METRE OF TREATED SEWAGE
2. (g/cu. m) DENOTES GRAMS OF DRY SOLIDS PER CUBIC METRE OF TREATED SEWAGE
3. THE ABOVE VALUES ARE BASED ON TYPICAL RAW SEWAGE WITH TOTAL BOD = 170 mg/L, SOLUBLE BOD = 50%, SS = 200 mg/L, P = 7 mg/L, NH_4 = 20 mg/L

11.5.5.4 Scum Control

Scum accumulation can be controlled by including any of the following provisions in the equipment design:

a. Floating covers keep the scum layer submerged and thus moist and more likely to be broken up;

b. Discharging recirculated sludge on the scum mat serves the same purpose as (a);

c. Recirculating sludge gas under pressure through the tank liquors and scum;

d. Mechanically destroying the scum by employing rotating arms or a propeller in a draft tube;

e. A large depth-area ratio; or

f. A concentrated sludge feed to the digester.

Items (e) and (f) would release large volumes of gas per unit area, keep the scum in motion and mix the solids in the digester.

11.5.5.5 *Grit and Sand Control*

The digesters should be designed to minimize sedimentation of the particles and facilitate removal if settling takes place. These objectives can be achieved if tank contents are kept moving at 0.23 to 0.3 m/s and the floor slopes are about 1:4.

11.5.5.6 *Alkalinity and pH Control*

The effective pH range for methane producers is approximately 6.5 to 7.5 with an optimum range of 6.8 to 7.2. Maintenance of this optimum range is important to ensure good gas production and to eliminate digester upsets.

The stability of the digestion process depends on the buffering capacity of the digester contents; the ability of the digester contents to resist pH changes. The alkalinity is a measure of the buffer capacity of a freshwater system. Higher alkalinity values indicate a greater capacity for resisting pH changes. The alkalinity shall be measured as bicarbonate alkalinity. Values for alkalinity in anaerobic digesters range from 1500 to 5000 mg/L as $CaCO_3$. The volatile acids produced by the acid producers tend to depress pH. Volatile acid concentrations under stable conditions range from 100 to 500 mg/L. Therefore, a constant ratio below 0.25 of volatile acids to alkalinity shall be maintained so that the buffering capacity of the system can be maintained.

Sodium bicarbonate, lime, sodium carbonate, and ammonium hydroxide application are recommended for increasing alkalinity of digester contents.

11.5.5.7 *Mixing*

Thorough mixing via digester gas (compressor power requirement 5 to 8 W/m^3) or mechanical means (6.6 W/m^3) in the primary stage will be necessary in all cases when digesters are proposed. This mixing shall assure the homogeneity of the digester contents, and prevent stratification.

Gas mixing methods are preferred. Gas mixing may be accomplished in any one of the following manners:

a. Short mixing tubes;
b. One or more deep-draft tubes;
c. Diffusers at the digester floor; or
d. Gas discharge below scum level.

11.5.5.8 *Sludge Inlets, Outlets, Recirculation, and High Level Overflow*

11.5.5.8.1 *Multiple Inlets and Draw-Offs*

Multiple sludge inlets and draw-offs and, where used, multiple recirculation suction and discharge points to facilitate flexible operation and effective mixing of the digester contents, shall be provided unless adequate mixing facilities are provided within the digester.

11.5.5.8.2 *Inlet Configurations*

One inlet should discharge above the liquid level and be located at approximately the center of the tank to assist in scum breakup. The second inlet should be opposite to the suction line at approximately the 0.7 diameter point across the digester.

11.5.5.8.3 *Inlet Discharge Location*

Raw sludge inlet discharge points should be so located as to minimize short circuiting to the digester sludge or supernatant draw-offs.

11.5.5.8.4 *Sludge Withdrawal*

Sludge withdrawal to disposal should be from the bottom of the tank. The bottom withdrawal pipe should be interconnected with the necessary valving to the recirculation piping, to increase operational flexibility in mixing the tank contents.

11.5.5.8.5 *Emergency Overflow*

An unvalved vented overflow shall be provided to prevent damage to the digestion tank and cover in case of accidental overfilling. This emergency overflow shall be piped to an appropriate point and at an appropriate rate in the treatment process or sidestream treatment facilities to minimize the impact on process units.

11.5.5.9 *Primary Tank Capacity*

The primary digestion tank capacity should be determined by rational calculations based upon such factors as volume of sludge added, its percent solids and character, the temperature to be maintained in the digesters, the degree or extent of mixing to be obtained and the degree of volatile solids reduction required. Calculations shall be submitted to justify the basis of design.

When such calculations are not based on the above factors, the minimum primary digestion tank capacity outlined in Sections 11.5.5.9.1 and 11.5.5.9.2 will be required. Such requirements assume that a raw sludge is derived from ordinary domestic wastewater, that a digestion temperature is to be maintained in the range of 32°C to 39°C, that 40 to 50 percent volatile matter will be maintained in the digested sludge and that the digested sludge will be removed frequently from the system.

11.5.5.9.1 *High Rate Digester*

The primary high rate digester shall provide for intimate and effective mixing to prevent stratification and to assure homogeneity of digester content. The system may be loaded at a rate up to 1.6 kg of volatile solids per cubic meter of volume per day in the active digestion unit. When grit removal facilities are not provided, the reduction of digester volume due to grit accumulation should be considered.

11.5.5.9.2 Low Rate Digester

For low rate digesters where mixing is accomplished only by circulating sludge through an external heat exchanger, the system may be loaded up to 0.64 kg of volatile solids per cubic meter of volume per day in the active digestion unit. This loading may be modified upward or downward depending upon the degree of mixing provided.

11.5.5.10 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. The necessary total storage time will depend upon the means of ultimate sludge disposal, with the greatest time required with soil conditioning operations (winter storage), and with less storage required with landfilling or incineration ultimate disposal methods. Offsite storage in sludge lagoons, sludge storage tanks, or other facilities may be used to supplement the storage capacity of the secondary digester. If high-rate primary digesters are used and efficient dewatering within the secondary digester is required, the secondary digester must be conservatively sized to allow adequate solids separation (secondary to primary sizing ratios of 2:1 to 4:1 are recommended).

11.5.5.11 Digester Covers

To provide gas storage volume and to maintain uniform gas pressures, a separate gas storage sphere should be provided, or at least one digester cover should be of the gas-holder floating type. If only one floating cover is provided, it shall be on the secondary digester. Insulated pressure and vacuum relief valves and flame traps shall be provided. Access manholes and at least two 200 mm sampling wells should also be provided on the digester covers.

Steel is the most commonly used material for digester covers. However, other properly designed and constructed materials can also be successfully employed, such as concrete and fibreglass.

11.5.5.12 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. The minimum diameter of sludge pipes shall be 200 mm for gravity withdrawal and 150 mm for pump suction and discharge lines. 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. Additional transfer lines should be from intermediate points in the primary digester (these can be dual-purpose supernatant and sludge lines).

11.5.5.13 Overflows

Each digester should be equipped with an emergency overflow system.

11.5.6 Gas Collection, Piping and Appurtenances

11.5.6.1 General

All portions of the gas system including the space above the tank liquor, storage facilities and piping shall be so designed that under all normal operating conditions, including sludge withdrawal, the gas will be maintained under positive pressure. All enclosed areas where any gas leakage might occur shall be adequately ventilated. All gas collection equipment, piping and appurtenances shall comply with the Canadian Gas Association Standard B105-M93.

11.5.6.2 *Safety Equipment*

All necessary safety facilities shall be included where gas is produced. Pressure and vacuum relief valves and flame traps together with automatic safety shut-off valves, are essential. Water seal equipment shall not be installed. Gas safety equipment and gas compressors should be housed in a separate room with an exterior entrance.

Provision should also be made for automatically purging the combustion chamber of the heating unit thoroughly with air after a shut-down or pilot light failure, and before it can be ignited. This will provide certainty that no explosive mixture exists within the unit.

11.5.6.3 *Gas Piping and Condensate*

The main gas collector line from the digestion tanks shall be at least 64 mm in diameter with the gas intake being well above the digester scum level, generally at least 1.2 m above the maximum liquid level in the tank. If gas mixing is used, the gas withdrawal pipe must be of sufficient size to limit the pressure drop in terms of the total gas flow from the digester. Such flow includes not only the daily gas production, but also the daily gas recycling flow. The recycling gas flow information should be combined with the estimate peak daily gas flow data to determine the proper piping size.

Gas pipe slopes of 20 mm/m are desirable with a minimum slope of 10 mm/m for drainage. The maximum velocity in sludge-gas piping shall be limited to not more than 3.4 or 3.7 m/s.

Gas piping shall slope to condensation traps at low points. The use of float controlled condensate traps is not permitted.

Adequate pipe support is essential to prevent breaking, and special care should be given where pipes are located underground.

Gas piping and pressure relief valves must include adequate flame traps. They should be installed as close as possible to the device serving as a source of ignition.

11.5.6.4 *Gas Utilization Equipment*

Gas burning boilers, engines, etc., should be located at ground level and in well ventilated rooms, not connected to the digester gallery. Gas lines to these units shall be provided with suitable flame traps.

11.5.6.5 *Electrical Systems*

Electrical fixtures and controls, in places enclosing anaerobic digestion appurtenances, where hazardous gases are normally contained in the tanks and piping, shall comply with the Canadian Electrical Code, Part 1 and the applicable provincial power standards. Digester galleries should be isolated from normal operating areas.

11.5.6.6 Waste Gas

Waste gas burners shall be readily accessible and should be located at least 15 m away from any plant structure if placed at ground level, or may be located on the roof of the control building if sufficiently removed from the tank. In remote locations it may be permissible to discharge the gas to the atmosphere through a return-bend screened vent terminating at least 3 m above the walking surface, provided the assembly incorporates a flame trap. Waste gas burners shall be of sufficient height and so located to prevent injury to personnel due to wind or downdraft conditions.

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 light.

Provision for condensate removal, pressure control, and flame protection ahead of waste burners is always required.

11.5.6.7 Ventilation

Any underground enclosures connecting with digestion tanks or containing sludge or gas piping or equipment shall be provided with forced ventilation in accordance with Section 3.2.7. Tightly fitting self-closing doors should be provided at connecting passageways and tunnels to minimize the spread of gas.

11.5.6.8 Meter

A gas meter with bypass shall be provided, to meter total gas production for each active digestion unit. Total gas production for two-stage digestion systems operated in series may be measured by a single gas meter with proper interconnected gas piping.

Where multiple primary digestion units are utilized with a single secondary digestion unit, a gas meter shall be provided for each primary digestion unit. The secondary digestion unit may be interconnected with the gas measurement unit of one of the primary units. Interconnected gas piping shall be properly valved with gas tight gate valves to allow measurement of gas production from either digestion unit or maintenance of either digestion unit.

Gas meters may be of the orifice plate, turbine or vortex type. Positive displacement meters should not be utilized. The meter must be specifically designed for contact with corrosive and dirty gases.

11.5.7 Digestion Tank Heating

11.5.7.1 Heating Capacity

11.5.7.1.1 Capacity

Sufficient heating capacity shall be provided to consistently maintain the design sludge temperature considering insulation provisions and ambient cold weather conditions. Where digestion tank gas is used for other purposes, an auxiliary fuel may be required.

11.5.7.2 Insulation

Wherever possible, digestion tanks should be constructed above ground-water level and should be suitably insulated to minimize heat loss.

11.5.7.3 *Heating Facilities*

Sludge may be heated by circulating the sludge through external heaters or by heating units located inside the digestion tank. The external heat exchanger systems are preferred.

11.5.7.3.1 *External Heating*

Piping shall be designed to provide for the preheating of feed sludge before introduction to the digesters. Provisions shall be made in the lay-out of the piping and valving to facilitate heat exchanger tube removal and cleaning of these lines. Heat exchanger sludge piping shall be sized for peak heat transfer requirements. Heat exchangers should have a heating capacity of 130 percent of the calculated peak heating requirement to account for the occurrence of sludge tube fouling.

11.5.7.3.2 *Other Heating Methods*

a. The use of hot water heating coils affixed to the walls of the digester, or other types of internal heating equipment that require emptying the digester contents for repair, are not acceptable.

b. Other systems and devices have been developed recently to provide both mixing and heating of anaerobic digester contents. These systems will be reviewed on their own merits. Operating data detailing their reliability, operation, and maintenance characteristics will be required.

11.5.7.4 *Hot Water Internal Heating Controls*

11.5.7.4.1 *Mixing Valves*

A suitable automatic mixing valve shall be provided to temper the boiler water with return water so that the inlet water to the removable heat jacket or coils in the digester can be held below a temperature at which caking will be accentuated. Manual control should also be provided by suitable by-pass valves.

11.5.7.4.2 *Boiler Controls*

The boiler should be provided with suitable automatic controls to maintain the boiler temperature at approximately 80°C to minimize corrosion and to shut off the main gas supply in the event of pilot burner or electrical failure, low boiler water level, excessive temperature, or low gas pressure.

11.5.7.4.3 *Boiler Water Pumps*

Boiler water pumps shall be sealed and sized to meet the operating conditions of temperature, operating head, and flow rate. Duplicate units shall be provided.

11.5.7.4.4 *Thermometers*

Thermometers should be provided to show inlet and outlet temperatures of sludge, hot water feed, hot water return and boiler water.

11.5.7.4.5 *Water Supply*

The chemical quality should be checked for suitability for this use.

11.5.7.5 *External Heater Operating Controls*

All controls necessary to insure effective and safe operation are required.

Provision for duplicate units in critical elements should be considered.

11.5.8 Supernatant Withdrawal

11.5.8.1 Piping Size

Supernatant piping should not be less than 150 mm in diameter. Precaution must be taken to avoid loss of digester gas through supernatant piping.

11.5.8.2 Withdrawal Arrangement

11.5.8.2.1 Withdrawal Levels

Piping should be arranged so that withdrawal can be made from three or more levels in the tank. A positive unvalved vented overflow shall be provided.

Both primary and secondary digesters should be equipped with supernating lines, so that during emergencies the primary digester can be operated as a single stage process.

11.5.8.2.2 Supernatant Selector

A fixed screen supernatant selector or similar type device shall be limited for use in an unmixed secondary digestion unit.

If a supernatant selector is provided, provisions shall be made for at least one other draw-off level located in the supernatant zone of the tank, in addition to the unvalved emergency supernatant draw-off pipe. High pressure back-wash facilities shall be provided.

11.5.8.2.3 Withdrawal Selection

On fixed cover tanks the supernatant withdrawal level should preferably be selected by means of interchangeable extensions at the discharge end of the piping.

11.5.8.3 Sampling

Provision should be made for sampling at each supernatant draw-off level. Sampling pipes should be at least 40 mm in diameter and should terminate at a suitably-sized sampling sink or basin.

11.5.8.4 Alternate Supernatant Disposal

An alternate disposal method for the supernatant liquor such as a lagoon, an additional sand bed or hauling from the plant site should be provided for use in case supernatant is not suitable or other conditions make it advisable not to return it to the plant. Consideration should be given to supernatant conditioning where appropriate in relation to its effect on plant performance and effluent quality.

11.5.9 Sludge Sampling Requirements

An adequate number of sampling pipes at proper locations should enable the operator to assess the quality of the contents and to know how much sludge is in the digesters. The following requirements shall govern the design:

a. To avoid clogging, sludge sampling pipes should be at least 75 mm in diameter;

b. Provision should be made for the connection of a water source of adequate pressure to these pipes for back flushing when the need arises; and

c. There shall be at least three sampling pipes each separately valved for the primary digesters and four for the secondary digesters.

11.6 AEROBIC SLUDGE DIGESTION

11.6.1 General

Aerobic digestion is accomplished in single or multiple tanks, designed to provide effective air mixing, reduction of the organic matter, supernatant separation and sludge concentration under controlled conditions.

11.6.2 Applicability

Aerobic digestion is considered suitable for secondary sludge or a combination of primary and secondary sludge. Table 11.6 presents the advantages and disadvantages in the use of aerobic sludge digestion.

TABLE 11.6 - ADVANTAGES AND DISADVANTAGES OF AEROBIC SLUDGE DIGESTION

ADVANTAGES	DISADVANTAGES
Low initial cost particularly for small plants	High energy costs
Supernatant less objectionable than anaerobic	Generally lower VSS destruction than anaerobic
Simple operational control	Reduced pH and alkalinity
Broad applicability	Potential for pathogen spread through aerosol drift
If properly designed, does not generate nuisance odours	Sludge is typically difficult to dewater by mechanical means
Reduces total sludge mass	Cold temperatures adversely affect performance

11.6.3 Field Data

Wherever possible, such as in the case of plant expansions, actual sludge quantity data should be considered for digester design. Often, due to errors introduced by poor sampling techniques, inaccurate flow measurements or unmeasured sludge flow streams, the sludge data from existing plants may be unsuitable for use in design. Before sludge data is used for design, it should be assessed for its accuracy.

11.6.4 Multiple Units

Multiple digestion units capable of independent operation are desirable and shall be provided in all plants where the design average flow exceeds 455 m^3/d . All plants not having multiple units shall provide alternate sludge handling and disposal methods.

11.6.5 Pretreatment

Thickening of sludge is recommended prior to aerobic digestion.

11.6.6 Design Considerations

Factors which should be considered when designing aerobic digesters include:

a. the type of sludge to be digested;
b. the ultimate method of disposal;
c. required winter storage;
d. digester pH;
e. sludge temperature; and
f. raw sludge qualities.

11.6.7 Solids Retention Time

Where land disposal of digested sludge is practised, a minimum solids retention time of 45 days is required. If local conditions require a more stable sludge, a sludge age of 90 days shall be necessary. To produce a completely stable sludge, a sludge age in excess of 120 days is required.

11.6.8 Hydraulic Retention Time

The minimum required hydraulic retention time for aerobic digesters provided with pre-thickening facilities are as follows:

Minimum HRT (days)	Type of Sludge
25	Waste Activated Sludge Only
25	Trickling Filter Sludge Only
30	Primary Plus Secondary Sludge

The more critical of the two guidelines, solids retention time and hydraulic retention time, shall govern the design.

11.6.9 Tank Design

11.6.9.1 Tank Capacity

The determination of tank capacities shall be based on rational calculations, including such factors as quantity of sludge produced, sludge characteristics, time of aeration and sludge temperature.

Calculations shall be submitted to justify the basis of design.

When such calculations are not based on the above factors, the minimum combined digestion tank capacity shall be based on the following:

Volatile solids loading shall not exceed 1.60 $kg/m^3 \cdot d$ in the digestion units. Lower loading rates may be necessary depending on temperature, type of sludge and other factors.

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 tank and 1/3 be in the second tank. Actual storage requirements will depend upon the ultimate disposal operation. Any minor additional storage requirements may be made up in the second stage digester, but if major additional storage volumes are required, separate on-site or off-site sludge storage facilities should be considered to avoid the power requirements associated with aerating greatly oversized aerobic digesters.

11.6.9.2 Air and Mixing Requirements

Aerobic sludge digestion tanks shall be designed for effective mixing by satisfactory aeration equipment. Sufficient air shall be provided to keep the solids in suspension and maintain dissolved oxygen from 1-2 mg/L. A minimum mixing and air requirement of 0.85 litres per second per cubic meter of tank volume shall be provided with the largest blower out of service. If diffusers are used, the non-clog type is recommended and they should be designed to permit continuity of service. If mechanical aerators are utilized, at least two turbine aerators per tank shall be provided. Use of mechanical equipment is discouraged where freezing temperatures are normally expected.

Air supply to each tank should be separately valved to allow aeration shut-down in either tank.

11.6.9.3 Tank Configuration

Aerobic digesters are generally open tanks. The tankage should be of common wall construction or earthen-bermed to minimize heat loss. Tank depths shall be between 3.5-4.5 m; tanks and piping should be designed to permit sludge addition, sludge withdrawal, and supernatant decanting from various depths. Freeboard depths of at least 0.9 to 1.2 m should be provided to account for excessive foam levels. Floor slopes of 1:12 to 3:12 should be provided.

11.6.9.4 Supernatant Separation and Scum and Grease Removal

11.6.9.4.1 Supernatant Separation

Facilities shall be provided for effective separation or decanting of supernatant. Separate facilities are recommended; however, supernant separation may be accomplished in the digestion tank provided additional volume is provided. The supernatant drawoff unit shall be designed to prevent recycle of scum and grease back to plant process units. Provision should be made to withdraw supernatant from multiple levels of the supernatant withdrawal zone.

11.6.9.4.2 Scum and Grease Removal

Facilities shall be provided for the effective collection of scum and grease from the aerobic digester for final disposal and to prevent its recycle back to the plant process and to prevent long term accumulation and potential discharge in the effluent.

11.6.10 High Level Emergency Overflow

An unvalved high level overflow and any necessary piping shall be provided to return digester overflow back to the head of the plant or to the aeration process in case of accidental overfilling. Design considerations related to the digester overflow shall include waste sludge rate and duration during the period the plant is unattended, potential effects on plant process units, discharge location of the emergency overflow, and potential discharge or suspended solids in the plant effluent.

11.6.11 Digested Sludge Storage Volume

11.6.11.1 Sludge Storage Volume

Sludge storage must be provided in accordance with Section 11.11 to accommodate daily sludge production volumes and as an operational buffer for unit outage and adverse weather conditions. Designs utilizing increased sludge age in the activated sludge system as a means of storage are not acceptable.

11.6.11.2 Liquid Sludge Storage

Liquid sludge storage facilities shall be based on the values in table 11.7 unless digested sludge thickening facilities are utilized to provide solids concentrations of greater than 2 percent.

TABLE 11.7 – LIQUID SLUDGE STORAGE VOLUMES

SLUDGE SOURCE	VOLUME m^3/P.E. · day	VOLUME $ft.^3$/P.E. · day
Waste activiated sludge – no primary settling, primary plus waste aeration activated sludge	0.004	0.13
Waste activated sludge exclusive of primary sludge	0.002	0.06
Primary plus fixed film reactor sludge	0.003	0.10

11.7 HIGH pH STABILIZATION

11.7.1 General

Alkaline material may be added to liquid primary or secondary sludges for sludge stabilization in lieu of digestion facilities; to supplement existing digestion facilities; or for interim sludge handling. There is no direct reduction of organic matter or sludge solids with the high pH stabilization process. There is an increase in the mass of dry sludge solids. Without supplemental dewatering, additional volumes of sludge will be generated. The design shall account for the increased sludge quantities for storage, handling, transportation, and disposal methods and associated costs.

11.7.2 Operational Criteria

Sufficient alkaline material shall be added to liquid sludge in order to produce a homogeneous mixture with a minimum pH of 12 after 2 hours of vigorous mixing. Facilities for adding supplemental alkaline material shall be provided to maintain the pH of the sludge during interim sludge storage periods.

11.7.3 Odour Control and Ventilation

Odour control facilities shall be provided for sludge mixing and treated sludge storage tanks when located with 800 m of residential or commercial areas. The reviewing authority should be contacted for design and air pollution control objectives to be met for various types of air scrubber units. Ventilation is required for indoor sludge mixing, storage or processing facilities.

11.7.4 Mixing Tanks and Equipment

11.7.4.1 Tanks

Mixing Tanks may be designed to operate as either a batch or continuous flow process. A minimum of two tanks shall be provided of adequate size to provide a minimum 2 hours contract time in each tank. The following items shall be considered in determining the number and size of tanks:

a. peak sludge flow rates;

b. storage between batches;

c. dewatering or thickening performed in tanks;

d. repeating sludge treatment due to pH decay of stored sludge;

e. sludge thickening prior to sludge treatment; and

f. type of mixing device used and associated maintenance or repair requirements.

11.7.4.2 Equipment

Mixing equipment shall be designed to provide vigorous agitation within the mixing tank, maintain solids in suspension and provide for a homogeneous mixture of the sludge solids and alkaline material. Mixing may be accomplished either by diffused air or mechanical mixers. If diffused aeration is used, an air supply of 0.85 L/m^3-s of mixing tank volume shall be provided with the largest blower out of service. When diffusers are used, the nonclog type is recommended, and they should be designed to permit continuity of service. If mechanical mixers are used, the impellers shall be designed to minimize fouling with debris in the sludge and consideration shall be made to provide continuity of service during freezing weather conditions.

11.7.5 Chemical Feed and Storage Equipment

11.7.5.1 General

Alkaline material is caustic in nature and can cause eye and tissue injury. Equipment for handling or storing alkaline material shall be designed for adequate operator safety. Storage, slaking, and feed equipment should be sealed as airtight as practical to prevent contact of alkaline material with atmospheric carbon dioxide and water vapour and to prevent the escape of dust material. All equipment and associated transfer lines or piping shall be accessible for cleaning.

11.7.5.2 Feed and Slaking Equipment

The design of the feeding equipment shall be determined by the treatment plant size, type of alkaline material used, slaking required, and operator requirements. Equipment may be either of batch or automated type. Automated feeders may be of the volumetric or gravimetric type depending on accuracy, reliability, and maintenance requirements. Manually operated batch slaking of quicklime (CaO) should be avoided unless adequate protective clothing and equipment are provided. At small plants, used of hydrated lime [$Ca(OH)_2$] is recommended over quicklime due to safety and labour-saving reasons. Feed and slaking equipment shall be sized to handle a minimum of 150% of the peak sludge flow rate including sludge that may need to be retreated due to pH decay. Duplicate units shall be provided.

11.7.5.3 Chemical Storage Facilities

Alkaline materials may be delivered either in bag or bulk form depending upon the amount of material used. Material delivered in bags must be stored indoors and elevated above floor level. Bags should be of the multi-wall moisture-proof type. Dry bulk storage containers must be as airtight as practical and shall contain a mechanical agitation mechanism. Storage facilities shall be sized to provide a minimum of a 30-day supply.

11.7.6 Sludge Storage

Refer to Section 11.11 for general design considerations for sludge storage facilities.

The design shall incorporate the following considerations for the storage of high pH stabilized sludge:

11.7.6.1 Liquid Sludge

Liquid high pH stabilized sludge shall not be stored in a lagoon. Said sludge shall be stored in a tank or vessel equipped with rapid sludge withdrawal mechanisms for sludge disposal or retreatment. Provisions shall be made for adding alkaline material in the storage tank. Mixing equipment in accordance with Section 11.7.4.2 shall also be provided in all storage tanks.

11.7.6.2 Dewatered Sludge

On-site storage of dewatered high pH stabilized sludge should be limited to 30 days. Provisions for rapid retreatment or disposal of dewatered sludge stored on-site shall also be made in case of sludge pH decay.

11.7.6.3 Off-Site Storage

There shall be no off-site storage of high pH stabilized sludge unless specifically permitted by the regulatory agency.

11.7.7 Disposal

Immediate sludge disposal methods and options are recommended to be utilized in order to reduce the sludge inventory on the treatment plant site and amount of sludge that may need to be retreated to prevent odours if sludge pH decay occurs. If the land application disposal option is utilized for high pH stabilized sludge, said sludge must be incorporated into the soil during the same day of delivery to the site.

11.8 SLUDGE DEWATERING

11.8.1 General

Sludge dewatering will often be required at sewage treatment plants prior to ultimate disposal of sludges. 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 sewage 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. With raw sludge, the freshness of the sludge will have a significant effect on dewatering performance (septic sludge will be more difficult to dewater than fresh raw sludge).

As with thickening systems, dewatering facilities may require sludge pre-treatment in the form of sludge grinding to avoid plugging pumps, lines and plugging or damaging dewatering equipment. Also, adequate ventilation equipment will be required in buildings housing dewatering equipment.

In evaluating dewatering system alternatives, the designer must consider the capital and operating costs, including labour, parts, chemicals and energy, for each alternative as well as for the effects which each alternative will have on the sewage treatment and subsequent sludge handling and ultimate sludge disposal operations.

In considering the need for sludge dewatering facilities, the designer should evaluate the economics of the overall treatment processes, with and without facilities for sludge water content reduction. This evaluation should consider both capital and operating costs of the various plant components and sludge disposal operations affected.

Wherever possible, pilot-plant and/or bench-scale data should be used for the design of dewatering facilities. With new plants, this may not always be possible and, in such cases, empirical design parameters must be used. The following subsections outline the normal ranges for the design parameters of such equipment.

For calculating dewatering design sludge handling needs, a rational basis of design for sludge production from sludge stabilization processes shall be developed and provided to the regulatory agencies for approval on a case-by case basis.

11.8.2 Dewatering Process Compatibility with Subsequent Treatment or Disposal Techniques

Table 11.8 outlines the relationship of dewatering to other processes.

TABLE 11.8 - THE RELATIONSHIP OF DEWATERING TO OTHER SLUDGE TREATMENT PROCESSES FOR TYPICAL MUNICIPAL SLUDGES

METHOD	PRETREATMENT NORMALLY PROVIDED		NORMAL USE OF DEWATERED CAKE			
	THICKENING	CONDITIONING	LANDFILL	LAND SPREAD	HEAT DRYING	INCINERATION
Rotary vacuum filter	Yes	Yes	Yes	Yes	Yes	Yes
Centrifuge (solid bowl)	Yes	Yes	Yes	Yes	Yes	Yes
Centrifuge (basket)	Variable	Variable	Yes	Yes	No	No
Drying beds	Variable	Not Usually	Yes	Yes	No	No
Lagoons	No	No	Yes	Yes	No	No
Filter presses	Yes	Yes	Yes	Variable	Not Usually	Yes
Horizontal belt filters	Yes	Yes	Yes	Yes	Yes	Yes

11.8.3 Sludge Drying Beds

11.8.3.1 Pre-Treatment

Sludge shall be pre-treated before being air-dried by either one of the following methods:

(a) Anaerobic digesters;

(b) Aerobic digesters with provision to thicken;

(c) Digestion in aeration tanks of extended aeration plants (with long sludge age, greater than about 20 days) preferably with provision to thicken using thickeners, lagoons or by other means; or

(d) Well designed and maintained oxidation ditches with sludge age longer than about 20 days (preferably after thickening).

11.8.3.2 Chemical Conditioning

The dewatering characteristics can be considerably improved by chemical conditioning of sludge prior to treatment in beds.

Since sludge conditioning can reduce the required drying time to 1/3 or less, of the unconditioned drying time, provision should be made for the addition of conditioning chemicals, usually polymers.

11.8.3.3 Design Criteria

11.8.3.3.1 Factors Influencing Design

The design and operation of sludge drying beds depend on the following factors:

(a) Climate in the area;

(b) Sludge characteristics;

(c) Pre-treatment (such as conditioning, thickening, etc.)

(d) Sub-soil permeability.

11.8.3.3.2 Bed Area

Consideration should be given to the following when calculating the bed area:

a. The volume of wet sludge produced by existing and proposed processes.

b. Depth of wet sludge drawn to the drying beds. For design calculation purposes a maximum depth of 200 mm shall be utilized. For operational purposes, the depth of sludge placed on the drying bed may increase or decrease from the design depth based on the percent solids content and type of digestion utilized.

c. Total digester volume and other wet sludge storage facilities.

d. Degree of sludge thickening provided after digestion.

e. The maximum drawing depth of sludge which can be removed from the digester or other sludge storage facilities without causing process or structural problems.

f. The time required on the bed to produce a removable cake. Adequate provision shall be made for sludge dewatering and/or sludge disposal facilities for those periods of time during which outside drying of sludge on beds in hindered by weather.

g. Capacities of auxiliary dewatering facilities.

Sludge drying beds may be designed from basic principles, laboratory tests, and/or pilot plant field studies. Calculations must be presented to the reviewing authority supporting any design based on the above methods. In the absence of such calculations the minimum sludge drying bed area shall be based on the criteria presented in Table 11.9.

TABLE 11.9 – SLUDGE DRYING BED AREAS

TYPE OF WASTEWATER TREATMENT	AREA (m^2/capita)		
	OPEN BEDS	COVERED BEDS	COMBINATION OF OPEN AND COVERED BEDS
Primary Plants (No secondary treatment)	0.12	0.10	0.10
Activated Sludge (No primary treatment)	0.16	0.13	0.13
Primary and Activated Sludge	0.20	0.16	0.16

The area of the bed may be reduced by up to 50% if it is to be used solely as a back-up dewatering unit. An increase of bed area by 25% is recommended for paved beds.

11.8.3.3.3 Percolation Type Beds

a. Pond Bottom

The bottom of the cell should be of impervious material such as clay or asphalt.

b. Underdrains

Underdrains should at least 100 mm in diameter laid with open joints. Perforated pipe may also be used. Underdrains should be spaced 2.5 to 3.0 m apart, with a slope of one per cent, or more. Underdrains should discharge back to the secondary treatment section of the sewage treatment plant. Various pipe materials may be selected provided the material is of suitable strength and corrosion resistant.

c. Gravel

The lower course of gravel around the underdrains should be properly graded and should be 300 mm in depth, extending at least 150 mm above the top of the underdrains. It is desirable to place this in two or more layers. The top layer, of at least 75 mm in depth, should consist of gravel three mm to six mm in size.

The gravel should be graded from 25 mm on the bottom to 3 mm on the top.

d. Sand

The top course should consist of 250 to 450 mm of clean coarse sand. The effective size should range from 0.3 to 1.2 mm with a uniformity co-efficient of less than 5.0. The finished sand surface should be level.

e. Additional Dewatering Provisions

Consideration shall be given for providing a means of decanting supernatant of sludge placed on the sludge drying beds. More effective decanting of supernatant may be accomplished with polymer treatment of sludge.

11.8.3.3.4 Impervious Type Beds

Paved drying beds should be designed with consideration for space requirements to operate mechanical equipment for removing the dried sludge.

11.8.3.3.5 Location

Depending on prevailing wind directions, a minimum distance of 100 to 150 m shall be kept from open sludge drying beds and dwellings. However, the minimum maybe reduced to 60 m to 80 m for enclosed beds. The selected location for open beds shall be at least 30 m from public roads and 25 m for enclosed beds. The plant owner may be required to spray deodorants and odour masking chemicals whenever there are complaints from the population in the neighbourhood.

11.8.3.3.6 Winter Storage

Alternative methods of disposal should be arranged for the non-drying season which may start as early as October (or November) and end in April (or March).

11.8.3.3.7 Dimensions

The bed size generally should be 4.5 to 7.5 m wide with the length selected to satisfy desired bed loading volume.

11.8.3.3.8 Depth of Sludge

The sludge dosing depth shall generally be 200 to 300 mm for warm weather operating modes; for winter freeze drying depths of 1 to 3 m can be used depending upon the number of degree days in winter.

11.8.3.3.9 Number of Beds

Three beds are desirable for increased flexibility of operation. Not less than two beds shall be provided.

11.8.3.3.10 Walls

Walls should be watertight and extend 400 to 500 mm above and at least 150 mm below the surface. Outer walls should be extended at least 100 mm above the outside grade elevation to prevent soil from washing on to the beds.

11.8.3.3.11 Sludge Influent

The sludge pipe to the beds should terminate at least 300 mm above the surface and be so arranged that it will drain. Concrete splash plates for percolation type beds should be provided at sludge discharge points. One inlet pipe per cell should be provided.

11.8.3.3.12 Sludge Removal

Each bed shall be constructed so as to be readily and completely accessible to mechanical cleaning equipment. Concrete runways spaced to accommodate mechanical equipment shall be provided. Special attention should be given to assure adequate access to the areas adjacent to the sidewalls. Entrance ramps down to the level of the sand bed shall be provided. These ramps should be high enough to eliminate the need for an entrance end wall for the sludge bed.

Atlantic Canada climatological conditions may permit 3 or 4 cycles (consisting of filling the open bed with digested sludge, drying and emptying) during the drying season. However, the number of cycles may be increased to approximately 10 with covered beds. These values are <u>tentative</u> and subject to revision after field observations.

11.8.3.3.13 Covered Beds

Consideration should be given to the design and use of covered sludge drying beds.

11.8.4 Sludge Lagoons

11.8.4.1 General

Sludge drying lagoons may be used as a substitute for drying beds for the dewatering of digested sludge. Lagoons are not suitable for dewatering untreated sludges, limed sludges, or sludges with a high strength supernatant because of their odour and nuisance potential. The performance of lagoons, like that of drying beds, is affected by climate; precipitation and low temperatures inhibit dewatering. Lagoons are most applicable in areas with high evaporation rates.

Sludge lagoons may also be used as temporary sludge storage facilities, when spreading on agricultural land cannot be carried out due to such factors as wet ground, frozen ground or snow cover.

Sludge lagoons as a means of dewatering digested sludge will be permitted only upon proof that the character of the digested sludge and the design mode of operation are such that offensive odours will not result. Where sludge lagoons are permitted, adequate provisions shall be made for other sludge dewatering facilities or sludge disposal in the event of upset or failure of the sludge digestion process.

11.8.4.2 Design Considerations

The design and location of sludge lagoons must take into consideration many factors, including the following:

- Possible nuisances - odours, appearance, mosquitos;
- Design - number, size, shape and depth;
- Loading factors - solids concentration of digested sludge, loading rates;
- Soil conditions - permeability of soil, need for liner, stability of berm slopes, etc.,
- Groundwater conditions - elevation of maximum groundwater level, direction of groundwater movement, location of wells in the area;
- Sludge and supernatant removal - volumes, concentrations, methods of removal, method of supernatant treatment and final sludge disposal; and
- Climatic effects - evaporation, rainfall, freezing, snowfall, temperature, solar radiation.

11.8.4.3 Pre-Treatment

Pre-treatment requirements for sludge lagoons are the same as those for sludge drying beds.

11.8.4.4 Soil and Groundwater Conditions

The soil must be reasonably porous and the bottom of the lagoons must be at least 1.2 m above the maximum ground water table. Surrounding areas shall be graded to prevent surface water entering the lagoon. In some critical instances, the reviewing authority may require a lagoon to be lined with plastic or rubber material.

11.8.4.5 Depth

Lagoons should be at least 1 m in depth while maintaining a minimum of 0.6 m of freeboard.

11.8.4.6 Seal

Adequate provisions shall be made to seal the sludge lagoon bottom and embankments in accordance with the requirements of Section 7.4.6. to prevent leaching into adjacent soils or ground water.

11.8.4.7 Area

The area required will depend on local climatic conditions. Not less than two lagoons should be provided.

11.8.4.8 Location

Consideration shall be given to prevent pollution of ground and surface water. Adequate isolation shall be provided to avoid nuisance production.

11.8.4.9 *Cycle Time and Sludge Removal*

The cycle time for lagoons varies from several months to several years. Typically, sludge is pumped to the lagoon for 18 months and then the lagoon is rested for six months.

Sludge is removed mechanically, usually at a moisture content of about 70 percent.

11.8.5 Mechanical Dewatering Facilities

11.8.5.1 *General*

Provisions shall be made to maintain sufficient continuity of service so that sludge may be dewatered without accumulation beyond storage capacity. If it is proposed to dewater the sludge by mechanical methods such as rotary vacuum filters, centrifuges, filter presses or belt filters, a detailed description of the process and design data shall accompany the plans.

Unless standby facilities are available, adequate storage facilities shall be provided. The storage capacity should be sufficient to handle at least 4 days of sludge production volume.

11.8.5.2 *Performance of Mechanical Dewatering Methods*

Table 11.10 outlines the solids capture, solids concentrations normally achieved and energy requirements for various mechanical dewatering methods.

TABLE 11.10 - SLUDGE DEWATERING METHODS AND PERFORMANCE WITH VARIOUS SLUDGE TYPES

DEWATERING METHOD	SOLIDS CAPTURE (%)	SOLIDS CONCENTRATIONS NORMALLY ACHIEVED (1)	MEDIAN ENERGY REQUIRED (MJ/DRY TONNE) (2)
Vacuum Filter	90 - 95	Raw primary + was (10-25%) Digested primary + was (15-20%) Was (8-12%)	1080
Filter Press	90 - 95	Raw primary + was (30-50%) Digested primary + was (35-50%) Was (25-50%)	360
Centrifuge (Solid Bowl)	95 - 99	Raw or Digested primary + was (15-25%) Was (12-15%)	360
Belt Filter	85 - 95	Raw or Digested primary + was (14-25%) Was (10-15%)	130

1. INCLUDING CONDITIONING CHEMICALS, IF REQUIRED.
2. MJ/DRY TONNE - DENOTES MEGAJOULES PER DRY TONNE OF SLUDGE THROUGHOUT.

11.8.5.3 *Number of Units*

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

There shall be a back-up pump and filtrate pump installed for each vacuum filter.

11.8.5.4 *Ventilation*

Adequate facilities shall be provided for ventilation of the dewatering area. The exhaust air should be properly conditioned to avoid odour nuisance.

11.8.5.5 *Chemical Handling Enclosures*

Lime-mixing facilities should be completely enclosed to prevent the escape of lime dust. Chemical handling equipment should be automated to eliminate the manual lifting requirement.

11.8.5.6 *Drainage and Filtrate Disposal*

Drainage from beds or filtrate from dewatering units shall be returned to the sewage treatment process at appropriate points.

11.8.5.7 *Other Dewatering Facilities*

If it is proposed to dewater sludge by mechanical means, other than those outlined below, a detailed description of the process and design shall accompany the plans.

11.8.5.8 *Vacuum Filters*

Of primary importance with vacuum filters is the solids concentration of sludge fed to the units. With all other operating variables remaining constant, increases in filtration rates vary in direct proportion to feed solids. Sludge thickening prior to vacuum filters is therefore extremely important. Higher concentrations in the sludge feed also result in lower filtrate solids.

Vacuum filtration systems should be designed in accordance with the following parameters:

a. Sludge feed pumps:
Variable capacity;

b. Vacuum pumps:
Generally one per machine with capacity of 10 $L/m^2 \cdot s$ at 65 kPa or more, vacuum;

c. Vacuum receiver:
Generally one per machine; max. air velocity 0.8 to 1.5 m/s; air retention time 2-3 minutes; filtrate retention time 4-5 minutes; all lines shall slope downward to the receiver from the vacuum filter.

d. Filtrate pumps:

Generally self-priming centrifugal; suction capacity greater than vacuum pump, 65 to 85 kPa vacuum; with flooded pump suctions; with check valve on the discharge side to minimize air leakage into the system; pumps must be sized for the maximum expected sludge drainage rates (usually produced by polymers);

e. Sludge flocculation tank:

Constructed of corrosion-resistant materials; with slow speed variable drive mixer, detention time 2-4 minutes with ferric and lime (with polymers shorter time may be used);

f. Wash water:

Filtered final effluent generally used;

g. Sludge measurement:

Should be provided unless measured elsewhere in plant;

h. Solids loading rate:

7-14 $g/m^2{\cdot}s$ for raw primary; 2.75-7 $g/m^2{\cdot}s$ for raw primary + WAS; 4-7 $g/m^2{\cdot}s$ for digested primary + WAS; not considered practical for use with WAS alone.

11.8.5.9 Filter Presses

As with vacuum filters, the capacity of filter presses is greatly affected by the initial solids concentration. With low feed solids, chemical requirements increase significantly.

Sludge, thickening should therefore be considered as a pre-treatment step.

Filter press systems should be designed in accordance with the following guidelines:

a. Sludge conditioning tank:

Detention time maximum 20 minutes at peak pumpage rate;

b. Feed pumps:

Variable capacity to allow pressures to be increased gradually, without underfeeding or overfeeding sludge; pumps should be of a type to minimize floc shear; pumps must deliver high volume at low head initially and low volume at high head during latter part of cycle; ram or piston pumps, progressing cavity pumps or double diaphragm pumps are generally used;

c. Cake handling:

Filter press must be elevated above cake conveyance system to allow free fall; cake can be discharged directly to trucks, into dumper boxes, or onto conveyors (usually cable cake breakers may be needed);

d. Cycle times:
1.5 to 6 h (normally 1.5 to 3 h); and

e. Operating pressures:
Usually 700 to 1400 kPa, but may be as high as 1750 kPa. The operating pressure shall not exceed 1000 to 1050 kPa, if polymer is applied as the conditioning agent.

11.8.5.10 Solid Bowl Centrifuges

Bowl length/diameter ratios of 2.5 to 4.0 should be provided to ensure adequate settling time and surface area. Bowl angles must be kept shallow.

The bowl flow pattern can be either counter-current or concurrent. Pool depth can be varied by adjustable weirs.

Conveyor design and speed will affect the efficiency of solids removal. Differential speed must be kept low enough to minimize turbulence and internal wear yet high enough to provide sufficient solids handling capacity.

For most wastewater sludges, the capacity of the centrifuge will be limited by the clarification capacity (hydraulic capacity) and therefore the solids concentration. Increasing the feed solids will increase the solids handling capacity. Thickening should, therefore, be considered as a pre-treatment operation.

Since temperature affects the viscosity of sludges, if the temperatures will vary appreciably (as with aerobic digestion), the required centrifuge capacity should be determined for the lowest temperature expected.

Other general design guidelines for solid bowl centrifuges are as follows:

a. Feed pump:
Sludge feed should be continuous; pumps should be variable flow type; one pump should be provided per centrifuge for multiple centrifuge systems; chemical dosage should vary with the pumpage rate;

b. Sludge pre-treatment:
Depending upon the sewage treatment process, grit removal, screening or maceration may be required for the feed sludge stream;

c. Solids capture:
85 - 95 percent is generally desirable;

d. Machine materials:
Generally carbon steel or stainless steel; parts subject to wear should be protected with hard facing materials such as a tungsten carbide material;

e. Machine foundations:
Foundations must be capable of absorbing the vibratory loads;

f. Provision for Maintenance:

Sufficient space must be provided around the machine(s) to permit disassembly; an overhead hoist should be provided; hot and cold water supplies will be needed to permit flushing out the machine; drainage facilities will be necessary to handle wash water.

11.8.5.11 Belt Filter Presses

Most types of waste water sludges can be dewatered with belt filter presses and the results achieved are generally superior to those of vacuum filters.

Chemical conditioning is generally accomplished with polymer addition.

Solids handling capabilities are likely to range from 50 g/m·s (based on belt width) for excess activated sludge to 330 g/m·s for primary sludge.

11.9 SLUDGE PUMPS AND PIPING

11.9.1 Sludge Pumps

11.9.1.1 General Sludge Pumping Requirements

Table 11.11 outlines general sludge pumping requirements for various sludge types.

11.9.1.2 Capacity

Pump capacities should be adequate but not excessive. Provision for varying pump capacity is desirable.

11.9.1.3 Duplicate Units

Duplicate units shall be provided where failure of one unit would seriously hamper plant operation.

TABLE 11.11 - GENERAL SLUDGE PUMPING REQUIREMENTS

SLUDGE SOURCE	SLURRY (% TOTAL SOLIDS)	STATIC HEAD (m)	TDH (m)	ABRASIVE SERVICE	DUTY
Pre-Treatment-Grit	0.5 - 10.0	0 - 1.5 (GRAVITY)	1.5 - 3	Yes –High	Heavy
Primary Sedimentation Unthickened					
Thickened	0.2 - 2.0	3 – 12	10 - 200	Yes	Medium
	4.0 - 10.0	3 – 12	12 - 25	Yes	Heavy
Secondary Sedimentation (for Recirculation)	0.5 - 2.0	1 - 2	3 - 4.5	No	Light
Secondary Sedimentation (for Thickening)	0.5 - 2.0	1.2 - 2.4	3 - 4.5	No	Light
Thickener	5 - 10	6 - 12	25 - 45	Yes/No*	Heavy
Underflow	5 - 10	60 - 120**	75 - 170	Yes/No*	Very Heavy
Digester					
Recirculation	3 - 10	0 - 1.5	2.4 - 3.6	No	Medium
Underflow	3 - 10	0 - 6	15 - 30	Yes/No*	Very Heavy
Chemically Produced Sludges:					
Alum/Ferric - primary	0.5 - 310	3 - 12	9 - 20	No	Light
Lime – Primary	1.0 - 6.0	3 - 12	9 - 25	No	Medium
Lime – Secondary	2.0 - 15.0	3 - 12	9 - 25	No	Medium
Incinerator Slurries	0.5 - 10	0 - 15	6 - 30	Yes- High	Heavy

* DEPENDS ON DEGRITTING EFFICIENCY

** HIGH PRESSURE FOR HEAT TREATMENT

11.9.1.4 Type

Plunger pumps, screw feed pumps or other types of pumps with demonstrated solids handling capability should be provided for handling raw sludge. Where centrifugal pumps are used, a parallel positive displacement pump should be provided as an alternate to pump heavy sludge concentrations, such as primary or thickened sludge, that may exceed the pumping head of the centrifugal pump.

11.9.1.5 Minimum Head

A minimum positive head of 600 mm shall be provided at the suction side of centrifugal type pumps and is desirable for all types of sludge pumps. Maximum suction lifts should not exceed three m for plunger pumps.

11.9.1.6 Head Loss

Figure 11.1 shows the multiplication factor to apply to the friction losses for turbulent flow for clean water to calculate the friction losses for untreated primary and concentrated sludges and digested sludge. Use of Figure 11.1 will often provide sufficiently accurate results for design, especially at solids concentrations below 3 percent. However, as pipe length, percent total solids and percent volatile solids increase, more elaborate methods may have to be used to calculate the friction losses with sufficient accuracy.

11.9.1.7 Sampling Facilities

Unless sludge sampling facilities are otherwise provided, quick closing sampling valves shall be installed at the sludge pumps. The size of valve and piping should be at least 40 mm and terminate at a suitable sized sampling sink or floor drain.

11.9.2 Sludge Piping

11.9.2.1 Size and Head

Sludge withdrawal piping should have a minimum diameter of 200 mm for gravity withdrawal and 150 mm for pump suction and discharge lines. Where withdrawal is by gravity, the available head on the discharge pipe should be adequate to provide at least 1.0 m/s velocity. With sludge pumpage velocities of 0.9 to 1.5 m/s should be developed. For heavier sludges and grease, velocities of 1.5 to 2.4 m/s are needed.

11.9.2.2 Slope

Gravity piping should be laid on uniform grade and alignment. The slope on gravity discharge piping should not be less than three percent. Provisions should be made for draining and flushing discharge lines.

11.9.2.3 Supports

Special consideration should be given to the corrosion resistance and continuing stability of supporting systems for piping located inside the digestion tank.

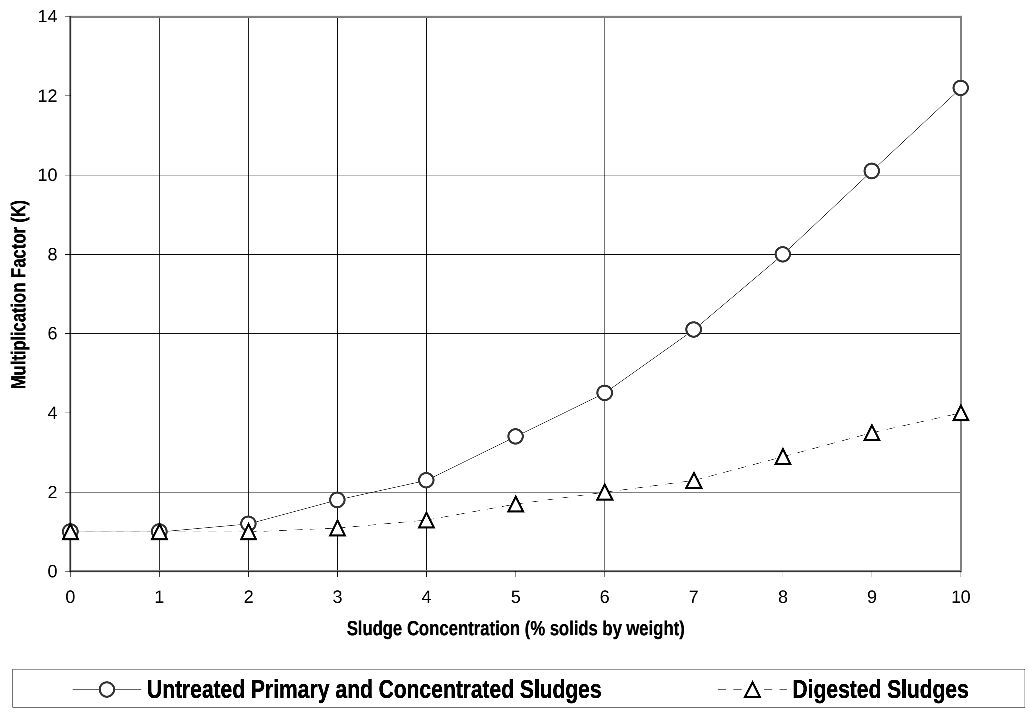
11.10 SLUDGE UTILIZATION ON LAND

11.10.1 General

The program of land spreading of sludge must be evaluated as an integral system which includes stabilization, storage, transportation, application, soil, crop and groundwater. The following guidelines were formulated to provide the criteria of municipal sludge utilization on land. Sewage sludge and septage is useful to crop and soil by providing nutrients and organic matter.

Figure 11.1

Approximate Friction Head Loss for Laminar Flow of Sludge



NOTES: 1. Multiply loss with clean water by K to estimate friction loss under laminar conditions (see text).

2. The Information on this figure has been extracted from EPA 625/1-79-011 "Process Design Manual for Sludge Treatment and Disposal:, September 1979.

Sewage sludge and septage contains heavy metals and other substances which could affect soil productivity and the quality of food. Sufficient information is not available to completely evaluate the deleterious effects. The purpose of the guidelines is to indicate the acceptable method of sludge utilization on land surface, based on current knowledge. It is recognized that these guidelines should be revised as more information becomes available.

11.10.2 General Limitations to be Observed

11.10.2.1 Stabilized Sludge

Only stabilized sludge shall be surface applied to farmland or pasture. Stabilized sludge is defined as processed sludge in which the organic and bacterial contents of raw sludge are reduced to levels deemed necessary by the regulatory agency to prevent nuisance odours and public health hazards.

A sludge may be considered stabilized if one of the following conditions can be met:

a. volatile solids in the sludge have been reduced by at least 38% during the treatment of the sludge;

b. the specific oxygen uptake rate (SOUR) of the sludge is less than 1.5 mg O_2/h.g of total sludge on a dry weight basis corrected to 20 °C. This test is only applicable to liquid aerobic biosolids withdrawn from an aerobic process.

c. Sludge meets the high pH stabilazion criteria described in section 11.7.

Any process which produces sludge equivalent in quality to the above in terms of public health factors and odour potential may be accepted. Additional treatment would be required to further reduce pathogens when the sludge is to be spread on dairy pastures and other crops which are in the human food chain.

11.10.2.2 Raw Vegetables

Sludge should not be applied to land which is used for growing food crops to be eaten raw, such as leafed vegetables and root crops.

11.10.2.3 Minimum pH

No sludge shall be applied on land if the soil pH is less than 6.0 at the time the sludge is applied. However, sludges containing lime may be applied to soils of lower pH, when they raise the soil pH to at least 6.0. The soil pH should range between 6.0 and 6.8. The pH shall be maintained above 6.0 for at least two years following the end of sludge application.

11.10.2.4 Persistent Organic Chemicals

Sufficient information is not available to establish criteria of sludge spreading in regard to persistent organic chemicals, such as pesticides and polychlorinated biphyenyls (PCB). However, if there is a known source in the sewer system service area which discharges or discharged in the past such chemicals, the sludge should be analyzed for such chemicals and the regulatory agency shall be consulted for recommendations concerning sludge spreading.

11.10.3 SITE SELECTION

11.10.3.1 General

By proper selection of the sludge application site, the nuisance potential and public health hazard should be minimized. The following items should be considered and the regulatory agency should be consulted for specific limits:

a. land ownership information;

b. groundwater table and bedrock location;

c. location of dwellings, roads and public access;

d. location of wells, springs, creeks, streams and flood plains;

e. slope of land surface;

f. soil characteristics;

g. climatological information;

h. land use plan; and

i. road weight restrictions.

11.10.3.2 Site Location

The following restrictions shall apply to the location of a proposed sludge to land application site:

a. The site should be remote from surface water courses. The minimum distance between the site and the high water mark of the surface water course should be determined by the land slope as follows:

TABLE 11.12 - MINIMUM DISTANCE TO WATERCOURSE

MAXIMUM SUSTAINED SLOPE	FOR SLUDGE APPLICATION DURING MAY TO NOVEMBER INCLUSIVE	FOR SLUDGE APPLICATION DURING DECEMBER TO APRIL INCLUSIVE
0 to 3%	100 m	360 m
3 to 6%	125 m	No Sludge to be Applied
6 to 8%	180 m	No Sludge to be Applied
Greater than 8%	No Sludge to be Applied unless special conditions exist	No Sludge to be Applied

b. The site shall be located a minimum distance from certain physical features, as specified in the following table:

c. The site shall be so located that the maximum level of the groundwater table at the site is at a sufficient distance below the surface to prevent the impairment of groundwater in aquifers as determined by the permeability of the soil;

d. No processed organic waste shall be applied to the site during any period in which conditions are such that surface runoff is likely to occur taking into account land slope, soil permeability and the climatic conditions of the area;

TABLE 11.13 - MINIMUM DISTANCE TO PHYSICAL FEATURES

TYPE OF FEATURE	MINIMUM SETBACK DISTANCE
Public Wells	150 m **
Private Wells	90 m **
Property Line	10 m *
Bedrock Outcrops	10 m *
Dwellings	90 m **
Uninhabited Buildings	30 m
Perennial Water Bodies & Watercourses	90 m
Intermittent Water Bodies & Watercourses	60 m
Swales and Man-Made Drainage Ditches	15 m
Primary & Secondary Roads	30 m *
Unimproved Dirt Roads	10 m *

NOTE: * 100 m setback required for spray irrigation areas

** 300 m setback required for storage lagoons and spray irrigation areas

e) The site shall be established only on land that is, or is intended to be, used for pasture, fallow, the growing of forage crops, scrub lands, or tree plantations.

   i) during the current growing season, or

   ii) where application of the processed organic waste is made sometime after the current growing season, to the end of the subsequent growing season; and

f. Berms and dykes of low permeability shall be constructed on the site where necessary to isolate the site and effectively prevent the egress of contaminants.

g. No sewage sludge handling facility should be located on a flood plain, an area which is inundated by a flood that has a 1% or greater change in recurring in any year, or a flood of a magnitude equalled or exceeded once in 100 years on the average.

h. No sewage sludge handling facility should be installed within the area of any municipal or city watershed unless water treatment consists of chemical precipitation.

i. A sewage sludge handling facility should not be located over land areas with a seasonal high water table at less than 450 mm below the ground surface, or with bedrock at less than 900 mm.

11.10.3.3 Land Characteristics

The following restrictions shall apply to the land characteristics of a proposed sludge to land application site:

(a) The land slope and soil permeability will determine the time of year that sludge may be applied.

TABLE 11.14 - SLUDGE APPLICATION PERIODS

MAXIMUM SUSTAINED SLOPE	SOIL PERMEABILITY	ALLOWABLE DURATION OF APPLICATION
0 to 3%	any	12 mo./yr.
3 to 6%	rapid to moderately rapid ($>5x10^{-5}$ to $8x10^{-6}$ m/s)	7 mo./yr. (May to November)
	($2x10^{-6}$ to $5x10^{-7}$ m/s) moderate to slow	6 mo./yr. (May to October)
6 to 8%	($>5x10^{-5}$ to $8x10^{-6}$ m/s) rapid to moderately rapid	7 mo./yr. (May to November)
	($2x10^{-6}$ to $5x\ 10^{-7}$ m/s) moderate to slow	6 mo./yr. (May to October)
Greater than 8%	any	No sludge applications unless warranted by special conditions

b. The ground water table during sludge application should be not less than 1 m from the surface for soils with moderate to slow permeability. For soils with rapid to moderately rapid permeability the groundwater table should be not less than 1.5 m from the surface; and

c. Where sludge application is carried out by tank truck, untiled land should be given preference to tiled land. Where tiled land is used the sludge hauling contractor should request instructions from the landowner, with regards to minimizing the possibility of damage to the tile system.

11.10.4 Sludge Application Rates

11.10.4.1 Nitrogen Restrictions

The rate of sewage sludge application is restricted to 5.6 t/ha of dry sludge solids content. This provides approximately 160 kg/ha of total nitrogen which is adequate for most grass crops during the year of application.

11.10.4.2 Phosphorus Restrictions

Due to the phosphorus content of digested sewage sludges and the upperlimits of phosphorus acceptable in the soil, the applications indicated in Section 11.10.4.1 may only be made once every two years. The phosphorus balance in the soil limits the application rates even when sludge is applied to crops with higher annual nitrogen requirements such as corn. In these cases ammonium nitrate fertilizer should be used in addition to the sludge.

11.10.4.3 Additive Metal Loading Restrictions

Unrestricted addition of metals to agricultural soils will result in both elevated metal content of the crops and plant toxicity. The following restrictions (with a built in safety factor) are designed to control this potential problem.

The following table lists criteria for the metal content in soils:

TABLE 11.15 – CRITERIA FOR METAL CONTENT IN SOILS		
METAL	MAXIMUM ACCEPTABLE METAL ADDITION TO SOIL (kg/ha)	MAXIMUM ACCEPTABLE METAL CONTENT IN SLUDGED SOILS (µg/g)
As	14	14
Cd	1.6	1.6
Co	30	20
Cr	210	120
Cu	150	100
Hg	0.8	0.5
Mo	4	4
Ni	32	32
Pb	90	60
Se	2.4	1.6
Zn	330	220

11.10.4.4 *Maximum Acceptable Concentrations in Soil*

The sludge application rates and metal restrictions have been developed to prevent excessive metal accumulation in the soil. When evaluating sludge application programs the following shall be considered the maximum acceptable concentration in the soil.

TABLE 11.17 – MAXIMUM ACCEPTABLE CONCENTRATIONS IN SOIL	
METAL	MAXIMUM CONCENTRATION IN SOIL (mg/kg) – DRY WEIGHT
As	12
Cd	1.4
Co	20
Cr	64
Cu	63
Hg	0.5
Mo	4
Ni	32
Pb	60
Se	1.6
Zn	200

11.10.5 Sludge Application on Forested Land

Disposal of sludge on forested land is considerably less hazardous than on cropland in terms of heavy metal toxicity unless the land is to be converted to cropland. For the allowable sludge loading the regulatory agency should be consulted.

11.10.6 Management of Spreading Operation

11.10.6.1 Hauling Equipment

The sludge hauling equipment should be designed to prevent spillage, odour and other public nuisance.

11.10.6.2 Valve Control

The spreading tank truck should be provided with a control so that the discharge valve can be opened and closed by the driver while the vehicle is in motion. The spreading valve should be of the "fail-safe" type (i.e., self-closing) or an additional manual stand-by valve should be employed to prevent uncontrolled spreading or spillage.

11.10.6.3 Sludge Storage

Sufficient sludge storage capacity shall be provided for periods of inclement weather and equipment failure. The storage facilities shall be designed, located and operated so as to avoid nuisance conditions.

11.10.6.4 Spreading methods

The selection of spreading methods depends on the sludge characteristics, environmental factors and others. When control of odour nuisance and runoff is required, immediate incorporation of sludge after spreading or subsurface injection should be considered. When such a method is utilized, an adjustment in the reduced rate of ammonia loss into the atmosphere should be considered in the computation for nitrogen balance.

The sludge should be spread uniformly over the surface when tank truck spreading, ridge and furrow irrigation or other methods are used. Sewage sludge application should not be made during or immediately after rainfall.

Proposals for subsurface application of sludge shall include for review a description of the equipment program for application.

Spray systems, except for downward directed types, will not ordinarily be approved.

11.10.6.5 Boundary Demarcation

The boundaries of the site shall be marked (i.e., with stakes at corners) as to avoid confusion regarding the location of the site during sludge application. The markers should be maintained until the end of the current growing season.

11.10.6.6 Public Access

Public access to a land application site should be controlled by either positive barriers or remoteness of the site.

11.10.6.7 Monitoring and Reporting

The requirements of the regulatory agency on the monitoring and reporting of the sludge spreading operation should be followed. As a minimum, the producer of sludge should regularly collect and record information on the sludge and soil characteristics and the volume of sludge spread on a particular site.

11.10.6.8 Land Restrictions

No vegetable crops may be grown in soils during the calendar year of the most recent sewage sludge application.

No animal grazing may occur on lands during the calendar year of the most recent sewage sludge application.

11.11 SLUDGE STORAGE

11.11.1 General

Sludge storage facilities shall be provided at all mechanical treatment plants. Appropriate storage facilities may consist of any combination of drying beds, lagoons, separate tanks, additional volume in sludge stabilization units, pad area or other means to store either liquid or dried sludge.

The design shall provide for odour control in sludge storage tanks and lagoons including aeration, covering or other appropriate means.

11.11.2 Volume

Rational calculations justifying the number of days of storage to be provided shall be submitted and shall be based on the total sludge handling and disposal system. Sludge production values for stabilization processes should be justified in the basis of design. If the land application method of sludge disposal is the only means of disposal utilized at a treatment plant, storage shall be provided based on considerations including at least the following items:

a. Inclement weather effects on access to the application land;

b. Temperatures including frozen ground and stored sludge cake conditions;

c. Haul road restrictions including spring thawing conditions;

d. Area seasonal rainfall patterns;

e. Cropping practices on available land;

f. Potential for increased sludge volumes from industrial sources during the design life of the plant; and

g. Available area for expanding sludge storage.

A minimum range of 120 to 180 days storage should be provided for the design life of the plant unless a different period is approved by the regulatory agency.

11.12 SLUDGE DISPOSAL METHODS

When other sludge disposal methods, such as incineration, lagoons and landfill, are considered, pertinent requirements from the regulatory agency shall be followed.

11.12.1 Sanitary Landfill

Sanitary landfilling of sludge, either separately or along with municipal solid waste, can be an acceptable means of ultimate sludge disposal.

The sludge must be stabilized prior to landfilling and daily soil cover must be provided.

Site selection must conform to the guidelines and criteria adopted by the regulatory agency. Particular attention is drawn to surface and groundwater protection, availability to on-site cover material and conformation to land use planning.

The site must be operated and maintained in accordance with the guidelines and regulations of the regulatory agency for sanitary landfill operation. Particular attention is drawn to the leachate and runoff of the heavy metals, persistent organics, pathogens and nitrates.

11.12.2 Incineration

Sludge incineration can be achieved in a multiple-hearth furnace.

Particular attention is drawn to proper air pollution control of the stack gases to conform to the regulations of the regulatory agency.

Sludge dewatering is required prior to incineration.

11.12.3 Land Reclamation

Sewage sludge can be used to reclaim strip-mine spoils or other low-quality land. Particular attention is drawn to the potential for water contamination and excessive accumulation of trace elements.

11.12.4 Energy/Resource Recovery

Energy and resource recovery processes include (1) recovery and recycling of marketable constituents of sludge or sludge incinerator ash, (2) co-incineration of sludge with combustible solid waste to generate power or steam or (3) pyrolysis of sludge to produce useful by-products such as fuel gases, oils, tars or activated charcoal. If such techniques are used, a detailed description of the process and design data shall accompany the plans.

11.13 SLUDGE TREATMENT ALTERNATIVES FOR PATHOGEN REDUCTION

11.13.1 General

The USEPA published a new regulation, 40 CFR-Part 503, in 1993. This regulation addresses the beneficial use and disposal of biosolids generated from the treatment of municipal sewage sludge. The regulation covers general provisions, land application, surface disposal, pathogen reduction, vector attraction reduction, and incineration. The section of the regulation that may impact Atlantic Canada is that, as a result of the classification of digested sludge into Class A or Class B and the corresponding restrictions placed on their disposal, processes to further reduce pathogens (PFRP's) are being developed and marketed.

The purpose of this section will be to describe some of the sludge digestion methods and PFRP's that have become popular in the US as a result of Rule 503. These methods may have application in Atlantic Canada wherever a need exists for a high quality end product due to restrictions that may exist for final disposal.

11.13.2 Processes to Further Reduce Pathogens (PFRP)

Unstabilized sludge contains putrescible organic substances, as well as pathogenic forms of bacteria, viruses, worm eggs, and the like. Sludge treatment processes that are classified as processes to further reduce pathogens must reduce both the organics and pathogens to set levels. PFRP alternatives include composting, heat drying, heat treatment, autothermal thermophilic aerobic digestion, irradiation, and pasteurization. The most applicable of the above processes will be described here.

11.13.2.1 Composting

Composting is a process in which organic material undergoes biological degradation to a stable end product. Sludge that has been composted properly is a sanitary, nuisance-free, humus-like material. Approximately 20 to 30 percent of the volatile solids are converted to carbon dioxide and water. As the organic material in the sludge decomposes, the compost heats to temperatures in the pasteurization range of 50 to 70^{0}C, and enteric pathogenic organisms are destroyed. A properly composted sludge may be used as a soil conditioner in agricultural or horticultural applications or for final disposal, subject to any limitations based on constituents in the sludge.

Most composting operations consist of the following basic steps: (1) mixing dewatered sludge with an amendment and/or a bulking agent: (2) aerating the compost pile either by the addition of air, by mechanical turning, or by both; (3) recovery of the bulking agent (if practicable); (4) further curing and storage; and (5) final disposal. An amendment is an organic material added to the feed substrate, primarily to reduce the bulk weight and increase the air voids for proper aeration. Amendments can also be used to increase the quantity of degradable organics in the mixture. Commonly used amendments are sawdust, straw, recycled compost and rice hulls. A bulking agent is an organic or inorganic material used to provide structural support and to increase the porosity of the mixture for effective aeration. Wood chips are the most commonly used bulking agents and can be recovered and reused. Aeration is required not only to supply oxygen, but to control the composting temperature and remove excess moisture.

Three major types of composting systems used are the aerated static pile, windrow, and in-vessel (enclosed mechanical) systems.

11.13.2.1.1 Aerated Static Pile

The aerated static pile system consists of a grid of aeration or exhaust piping over which a mixture of dewatered sludge and bulking agent is placed. In a typical static pile system, the bulking agent consists of wood chips, which are mixed with the dewatered sludge by a pug mill type or rotating drum mixer or by movable equipment such as a front-end loader. Material is composted for 21 to 28 days and is typically cured for another 30 days of longer. Typical pile heights are about 2 to 2.5 m. A layer of screened compost is often placed on top of the pile for insulation. Disposable corrugated plastic drainage pipes commonly used for air supply and each individual pile is recommended to have an individual blower for more effective aeration control. Screening of the cured compost is usually done to reduce the quantity of the end product requiring ultimate disposal and to recover the bulking agent. For improved process and odour control, many new facilities cover or enclose all or significant portions of the system.

11.13.2.1.2 Windrow

In a windrow system, the mixing and screening operations are similar to those for the aerated static pile operation. Windrows are constructed from 1 to 2 m high and 2 to 4.3 m at the base. The rows are turned and mixed periodically during the composting period. Supplemental mechanical aeration is used in some applications. Under typical operating conditions, the windrows are turned a minimum of five times while the temperature is maintained at or above 55^{0}C. Turning of the windrows is often accompanied by the release of offensive odours. The composting period is about 21 to 28 d. In recent years, specialized equipment has been developed to mix the sludge and the bulking agent and to turn the composting windrows. Some windrow operations are covered or enclosed, similar to aerated static piles.

11.13.2.1.3 In-Vessel Composting Systems

In-vessel composting is accomplished inside an enclosed container or vessel. Mechanical systems are designed to minimize odours and process time by controlling conditions such as air flow, temperature, and oxygen concentration.

11.13.2.1.4 Design Considerations

The factors that must be considered in the design of a composting system are presented in the following table:

TABLE 11.17 - DESIGN CONSIDERATIONS FOR AEROBIC SLUDGE COMPOSTING PROCESSES

ITEM	COMMENT
Type of Sludge	Both untreated and digested sludge can be composted successfully. Untreated sludge has a greater potential for odors, particularly for windrow systems. Untreated sludge has more energy available, will degrade more readily, and has a higher oxygen demand.
Amendments and Bulking Agents	Amendment and bulking agent characteristics, such as moisture content, particle size, and available carbon, affect the process and quality of the product. Bulking agents should be readily available. Wood chips, sawdust, recycled compost, and straw have been used.
Carbon : Nitrogen Ratio	The initial C:N ratio should be in the range of 25:1 to 35:1 by weight. Carbon should be checked to ensure it is easily biodegradable.
Volatile Solids	The volatile solids of the composting mix should be greater than 50 percent.
Air Requirements	Air with at least 50 percent of the oxygen remaining should reach all parts of the composting material for optimum results, especially in mechanical systems.
Moisture Content	Moisture content of the composting mixture should not be greater than 60 percent for static pile and windrow composting and not greater than 65 percent for in-vessel composting.
pH	pH of the composting mixture should generally be in the range of 6 to 9.
Temperature	The optimum temperature for biological stabilization is between 45 and 55 ^oC. For best results, the temperature should be maintained between 50 and 55 ^oC for the first few days and between 55 and 60 ^oC for the remainder of the composting period. If the temperatures are allowed to increase beyond 60 ^oC for a significant period of time, biological activity will be reduced.
Mixing and Turning	To prevent drying, caking, and air channelling, material in the process of being composted should be mixed or turned on a regular schedule as required. Frequency of mixing or turning will depend on the type of composting operation.
Heavy Metals and Trace Organics	Heavy metals and trace organics in the sludge and finished compost should be monitored to ensure that the concentrations do not exceed the applicable regulations for end use of the product.
Site Constraints	Factors to be considered in selecting a site include available area, access, proximity to treatment plant and other land uses, climatic conditions, and availability of buffer zone.

11.13.2.1.5 Co-Composting with Solid Wastes

Co-composting of sludge and municipal solid wastes may not require sludge dewatering. Feed sludges may have a solids content ranging from 5 to 12 percent. A 2 to 1 mixture of solid wastes to sludge is recommended as a minimum. The solid wastes should be presorted and pulverized in a hammermill prior to mixing with sludge.

11.13.2.2 Autothermal Thermophilic Aerobic Digestion (ATAD)

11.13.2.2.1 General

With an adequate supply of oxygen, microorganisms, nutrients, and biodegradable organic material, autothermal aerobic digestion can degrade complex organic substances into end products including carbon dioxide and water. Some of the energy released by microbial degradation is used to form new cellular material; much of it is released as heat. Typical biological heat production values reported or assumed range from 14,190 to 14,650 kJ/kg O_2. The carbonaceous oxygen requirements vary, but are often considered to be 1.42 kg O_2/kg volatile suspended solids (VSS) oxidized. In autothermal thermophilic aerobic digestion, the heat released by the digestion process is the major heat source used to achieve the desired operating temperature.

Autothermal conditions result from an adequately thickened sludge feed, a suitably insulated reactor, good mixing, and heat loss to an acceptable level.

11.13.2.2.2 Sludge Feed Source and Thickening Requirements

Autothermal thermophilic aerobic digestion (ATAD) processes thicken the sludge prior to digestion to minimize the size of the digestion tanks and to limit the energy requirements for mixing and heating.

A sludge in an ATAD system is adequate to support process temperature requirements if it is thickened to 4-6% TSS, of which at least 2.5% is mostly biodegradable volatile solids. Gravity thickening can usually achieve this concentration. Some plants also successfully co-thicken the waste-activated sludge with the primary solids and thereby avoid the need for a separate sludge thickener.

Sludge form plants without primary clarifiers and with activated sludge food-to-mass ratio (F/M) loadings as low as 0.1 to 0.15 kg BOD_5/kg TVSS still seems to be suitable for ATAD.

11.13.2.2.3 Detention Time

To satisfy the process requirements for destruction of pathogens and total organic solids, the design hydraulic detention time for ATAD systems is established at 5 to 6 d (2.5 to 3 d per reactor). Sixty percent of the volatile solids destruction occurs in the first reactor.

11.13.2.2.4 Feed Cycle and Isolated Reaction Time

Batch Feeding is Established by design intent and is reflected in the sizing of the sludge feed pump(s). The feed sludge pumping system is sized to deliver the daily thickened sludge volume to the reactor in less than 1 hour. Since all the sludge is pumped within a 1 hour period, the reactors are isolated for the remaining 23 hours during each day. This undisturbed reaction time is considered an important factor in attaining a high degree of pathogen destruction.

11.13.2.2.5 Aeration and Mixing

Typical design ranges for empirical aeration/mixing parameters include:

Specific power:	85-105 W/m^3 of active reactor volume
Air Input:	4 m^3/m^3·h of active reactor volume
Energy requirement:	9-15 kWh/m^3 of sludge throughput

11.13.2.2.6 Temperature and pH

An average temperature of 55^{0}C in the second reactor is used for design purposes. The temperature in the second reactor can exceed 55^{0}C. However, to prevent resolubilization of organics, the temperature should not exceed 65^{0}C.

The design areas that most affect operating temperature include the efficiency of the aeration system, reactor insulation, foam management in the reactor, and sludge prethickening.

Generally, process pH does not have to be controlled by special design considerations. The thermophilic operating temperatures of the reactors suppress nitrification in the process. Consequently, the pH depressions that could occur in a nitrifying environment are not experienced. With a feed sludge pH of 6.5, pH values in the first reactor are typically near 7.2 and may approach 8.0 in the second reactor.

11.13.2.2.7 Foam Control

The foam layer in the treatment reactor plays an important role in the ATAD process, though this role has not been fully evaluated. The foam layer appears to improve oxygen utilization, enhance the biological activity, and provide insulation, but it retards the amount of air entering the reactor. The amount of foam should be optimized and not eliminated.

Treatment reactors are sized to accommodate about 0.5 to 1.0 m of freeboard, which is partially used as volume for foam development and control. Control consists of densifying the foam (i.e., breaking up the large foam bubbles) to form a compact layer floating above the liquid surface of the reactor.

11.13.2.2.8 Post Thickening / Dewatering

In general, the gravity thickening performance of the hot effluent sludge is poor immediately after treatment, due to the thermal convection currents that occur in a thickening tank. If the sludge is allowed to cool in the post-thickening/storage tank or additional heat exchangers exist that cool the sludge down, thickening performance is usually satisfactory. Old Imhoff tanks have been very suitable for post-thickening. Values of 6-9% TS are typically achieved.

11.13.2.2.9 Detailed Design Manual

The following sources contain detailed design information for natural wastewater treatment systems:

U.S. Environmental Protection Agency: *Autothermal Thermophilic Aerobic Digestion of Municipal Wastewater Sludge*, EPA 625/ 10-90-007, Washington, DC, 1990.

11.13.2.3 Heat Drying

Heat drying of sludge involves the supply of auxiliary heat to mechanical drying processes in order to increase the vapour holding capacity of the ambient air and to provide the latent heat necessary for evaporation. Temperatures of greater than 80 °C are required in this process.

11.13.2.4 Heat Treatment

Heat treatment is a continuous process in which sludge is heated in a pressure vessel to temperatures up to 260 °C for approximately 30 minutes. This serves as both a stabilization process and a conditioning process. It conditions the sludge by rendering the solids capable of being dewatered without the use of chemicals. When the sludge is subjected to the high temperatures and pressures, the thermal activity releases bound water and results in the coagulation of solids. In addition, hydrolysis of proteinaceous materials occurs, resulting in cell destruction and release of soluble organic compounds and ammonia nitrogen.

11.13.3 Restrictions for Sludge Utilization on Land

Sludge that has been treated by a PFRP may have less restrictions placed on its disposal if it can be shown that the levels of pathogens and VSS have been reduced to the satisfaction of the regulatory authority.

APPENDIX A OPERATORS CERTIFICATION

A.1 PLANT CLASSIFICATION

A.1.1 General

It is necessary to classify facilities according to their "degree of difficulty to operate". All municipal wastewater facilities within Atlantic Canada be classified by the appropriate regulatory agency. A facility Classification Certificate may then be issued for each facility. Classification is mandatory in Nova Scotia.

The regulatory agency, upon application of a predesignated form, should classify all wastewater collection systems and treatment plants. The Classification shall take due regard to size, type and character of the wastewater to be treated, and other physical conditions affected by such treatment plants and distribution systems and according to the skill, knowledge, and experience required of an operator.

A.1.2 Classification System

Facilities shall be classified in one of four classes designated as Class I, II, III, or IV according to complexity of operation (with Class IV being the highest). Classification of treatment plants shall be based on a point system in accordance with Table 1.1 and 1.2. Classification of collection and distribution systems shall be based upon population served.

A.1.3 Classification Changes

Classification of any facility may be changed at the discretion of the regulatory agencies by reason of changes in any condition or circumstance upon which the original classification was predated. Due notice of any such change shall be given to the owner of the facility.

A.1.4 Classification Certificate

On satisfactory fulfilment of the requirements provided herein and based on the approval of the appropriate regulatory agency, a suitable certificate to the applicant designating the plant or system classification shall be issued.

Certificates of classification shall be permanent unless revoked for cause or replaced by one of a higher class.

TABLE 1.1 - FACILITY CLASSIFICATION SYSTEM

Facility	Units	I	II	III	IV
Wastewater Collection (WWC*)	Pop Served	1500 or less	1501 - 15,000	15,001 - 50,000	50,000 & greater
Wastewater Treatment (WWT)	Range of Points	30 or less	31 - 55	56 - 75	76 & greater

* Simple "in-line" treatment (such as booster pumping or preventive chlorination or odour control) is considered an integral part of a collection system.

TABLE 1.2 - POINT SYSTEM CLASSIFICATION OF WASTEWATER TREATMENT PLANTS (WWT)

Item	Points
Size Maximum population equivalent (PE) served, peak day	 1 pt. per 10,000 PE or part (Max 10).
Design flow (avg. day) or peak month's flow (avg. day), whichever is larger	1 pt. per MGD or part (Max 10).
Effluent Discharge Receiving stream (sensitivity) Land disposal-evaporation Subsurface disposal	 2-6 * 2 4
Variation in Raw Wastes (slight to extreme)	0-6 **
Pretreatment Screening, comminution Grit removal Plant pumping of main flow	 3 3 3
Primary Treatment Primary clarifiers Combined sedimentation digestion Chemical addition (except chlor., enz.)	 5 5 4
Secondary Treatment Trickling filter w/sec. clarifiers Activated sludge w/sec. clarifiers (including ext. aeration and oxidation ditches) Stabilization ponds without aeration Aerated lagoon	 10 15 5 8
Advanced Water Treatment Polishing pond Chemical/physical - without secondary Chemical/physical - following secondary Biological or chemical/biological Ion exchange Reverse osmosis, electrodialysis Chemical recovery, carbon regeneration Oxygen generation on-site	 2 15 10 12 10 15 4 5
Solids Handling Thickening Anaerobic digestion Aerobic digestion Evaporative sludge drying Mechanical dewatering Solids reduction(1) Incineration (2) Wet oxidation	 5 10 6 2 8 12 12
Disinfection Chlorination or comparable On-site generation or disinfection	 5 5
Laboratory Control by Plant Personnel Bacteriological (complexity) Chemical/physical (complexity)	 3-10 *** 1-10 ****

* These items are based on judgement and should be given the higher number of points for greater complexity. (See Point System Guidelines (WWT)).

POINT SYSTEM GUIDELINES (WWT)

* **Effluent Discharge**

Receiving stream sensitivity

The key concept is the degree of dilution provided under low flow conditions. Suggested point values are:

Table 1.2 - **POINT SYSTEM CLASSIFICATION OF WASTEWATER TREATMENT PLANTS (WWT) Cont'd**		
	Items	Points
	"Effluent limited segment in EPA terminology	
	secondary treatment is adequate	2
	More than secondary treatment is required	4
	"Water quality limited segment" in EPA terminology: stream conditions are very critical (dry run, for example) and a very high degree of treatment is required	6
**	**Variation in Raw Wastes (slight to extreme)**	
	The key concept is frequency and/or intensity of deviation of excessive variation from normal or typical fluctuations; such deviation can be in terms of strength, toxicity, shock loads, I and I, etc. Suggested point values are:	
	Variations do not exceed those normally or typically expected	0
	Recurring deviations or excessive variations of approximately 100% in strength and/or flow	3
	Recurring deviations or excessive variations of approximately 200% in strength and/or flow	6
***	**Laboratory Control by Plant Personnel**	
	Bacteriological/biological (complexity). The key concept is to credit bacti/bio lab work done on-site by plant personnel. Suggested point values are:	
	Lab work done outside the plant	0
	Membrane filter procedures	3
	Use of fermentation tubes or any dilution method	5
	Biological identification	7
	Virus studies or similarly complex work conducted on-site	10
****	**Chemical/Physical (complexity)**	
	The key concept is to credit chemical/physical lab work done on-site by plant personnel. Suggested point values are:	
	Lab work done outside the plant	0
	Push-button or visual methods for simple tests such as pH, settleable solids - up to	3
	Additional procedures such as DO, BOD, titration, solids, volatile content - up to	5
	More advanced determinations such as COD, gas analysis, specific constituents - up to	7
	Highly sophisticated instrumentation such as atomic absorption and gas chromatography	10

A.2 OPERATOR CERTIFICATION

A.2.1 General

Owners of municipal wastewater facilities may be required to have one or more operators certified at the same level as the facility classification or higher. This requirement will be mandatory in Nova Scotia as of January 1, 1997, for Class I facilities; as of January 1, 1998, for Class II facilities; as of January 1,1999, for Class III facilities; and as of January 1, 2000, for Class IV facilities. In New Brunswick, Newfoundland, and Prince Edward Island, certification requirements are being administered under a voluntary certification program. If and when operator certification becomes mandatory in these provinces, levels achieved under the voluntary program will still be valid.

A.2.2 Operator Pre-Requisites

The minimum prerequisites for operator certification at the various levels are shown in Table 1.3. The specific requirements for education/experience for each of the levels of certification are further defined in the sections following.

TABLE 1.3 - OPERATOR CERTIFICATION PRE-REQUISITES			
Operator Class		**WWT**	**WWC**
Op-in-trng	Education Experience	12 (a)	12 (a)
I	Education Experience	12 1	12 1
II	Education Experience	12 3	12 3
III	Education Experience DRC	14 4 (2)	14 4
IV	Education Experience DRC	16 4 (2)	16 6
WWT - Wastewater Treatment WWC - Wastewater Collection DRC - Direct Responsible Charge			

(a) Three (3) months operating experience or completion of an approved basic training course. It is recognized that the position operator-in-training (OIT) is not a legally required position. It is included here to illustrate a method of encouraging new entrants in the field to enter into the certification program.

A.2.3 Experience Requirements

"Operating experience" is defined as time spent at a plant or system in satisfactory performance of operation duties.

"Direct responsible charge" (DRC) is defined as follows:

(a) In smaller facilities where shift operation is not required, DRC experience is:

1) Active, daily, on-site charge, and performance of operation duties in the same or next lower certification class.

(b) In large facilities where shift operation is required, DRC experience is defined as both:

1) Active, daily, on-site technical direction, and supervision of operation duties in the same or next lower certification class; and/or

2) Active, daily, on-site charge of an operating shift, or a major segment of a system or facility in the same or next lower certification class.

A.2.4 DRC Requirements

"Class I or II" - For Class I or Class II certification, DRC experience is not required.

"Class III - For Class III certification, one-half of the operating experience requirement must be DRC experience gained:

(a) in Class II, if in a top supervisory position, and as specified in the definition of DRC experience, or

or

(b) in Class II or higher, in on-site charge, and as specified in the definition of DRC experience.

"Class IV" - For Class IV certification, one-half of the operating experience requirements must be DRC experience gained:

(a) in Class III, if in a top supervisory position, and as specified in the definition of DRC experience, or

(b) in Class III or higher, in on-site charge, and as specified in the definition of DRC experience.

"Collection and Distribution" - DRC experience is not required for wastewater collection.

A.2.5 Substitutions

"Class I" - No substitution for experience requirement for Class I is permitted.

"Class II, III or IV" - Substitutions may be made for required experience in Classes II, III and IV, but with the limitation that 50 percent of any stated experience requirement (both operating and DRC) must be met by actual on-site operating experience in a plant or system.

A.2.6 Formal Education Substituted for Experience

"School" - High school education cannot be credited for substitutional value toward any experience requirement, high school education is in itself a basic requirement for certification at any level.

"Post-High School" - Approved relevant formal academic education at the post-high school or college level may be substituted for experience requirement (either operating or DRC) on a year-for-year basis, subject to the 50 percent limitation previously described.

"Note" - Education applied in substitution for an experience requirement cannot also be applied to the education requirement.

A.2.7 Experience Substituted for Formal Education

Substitutions may be made for required formal education, subject to the following criteria:

"Grades 1-12" - 1 year of operating experience (operating or DRC) may be substituted for 2 years of grade school, without limitation.

"Class III" - A maximum of 1 year of DRC experience in a Class II (or higher) position may be substituted for 1 year of post-secondary formal education requirement for Class II certification.

"Class IV" - A maximum of 1 year of DRC experience in a Class III (or higher) position may be substituted for 1 year of post-secondary formal education requirement for Class IB certification.

"Note 1" - Experience applied in substitution for an education requirement cannot also be applied to the experience requirement.

A.2.8 Examinations

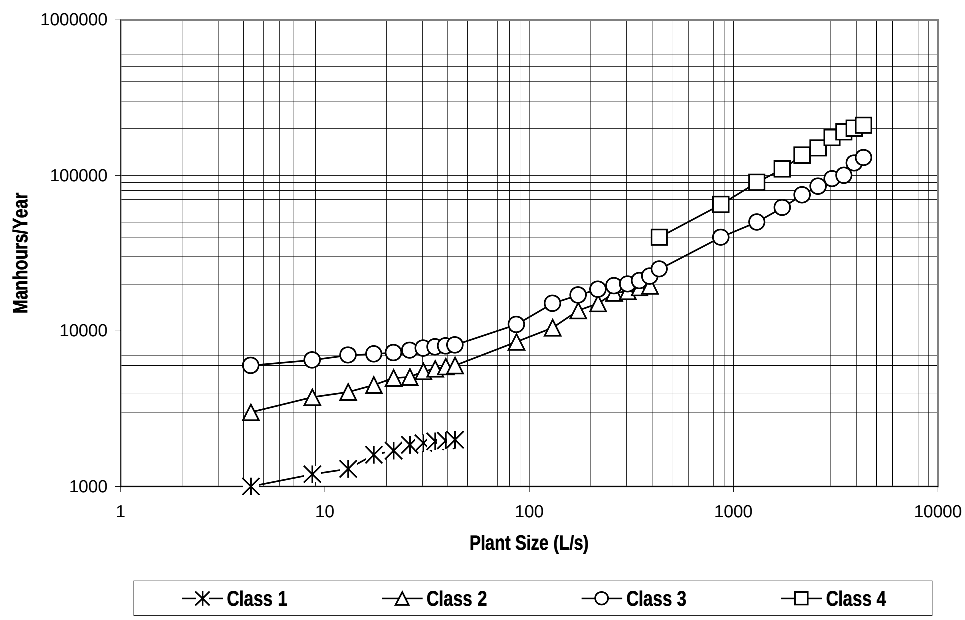
Once qualified under the education and experience requirements, an operator must pass an exam approved by the regulatory agencies in order to be certified.

B.1 OPERATION AND MAINTENANCE MANHOURS

Figure 2.1 outlines overall manpower requirements for each class of wastewater treatment facility over a wide range of average design flows. The information presented is to assist those seeking to project future wastewater treatment plant staffing requirements as a basis for planning of manpower training programs. The data can also be used as planning a guide for staffing requirements for individual conventional treatment plants, provided recognition is given to the "average" nature of the estimating data, and judgement is applied regarding specific local circumstances.

Figure 2.1

MANPOWER REQUIREMENTS vs. PLANT SIZE



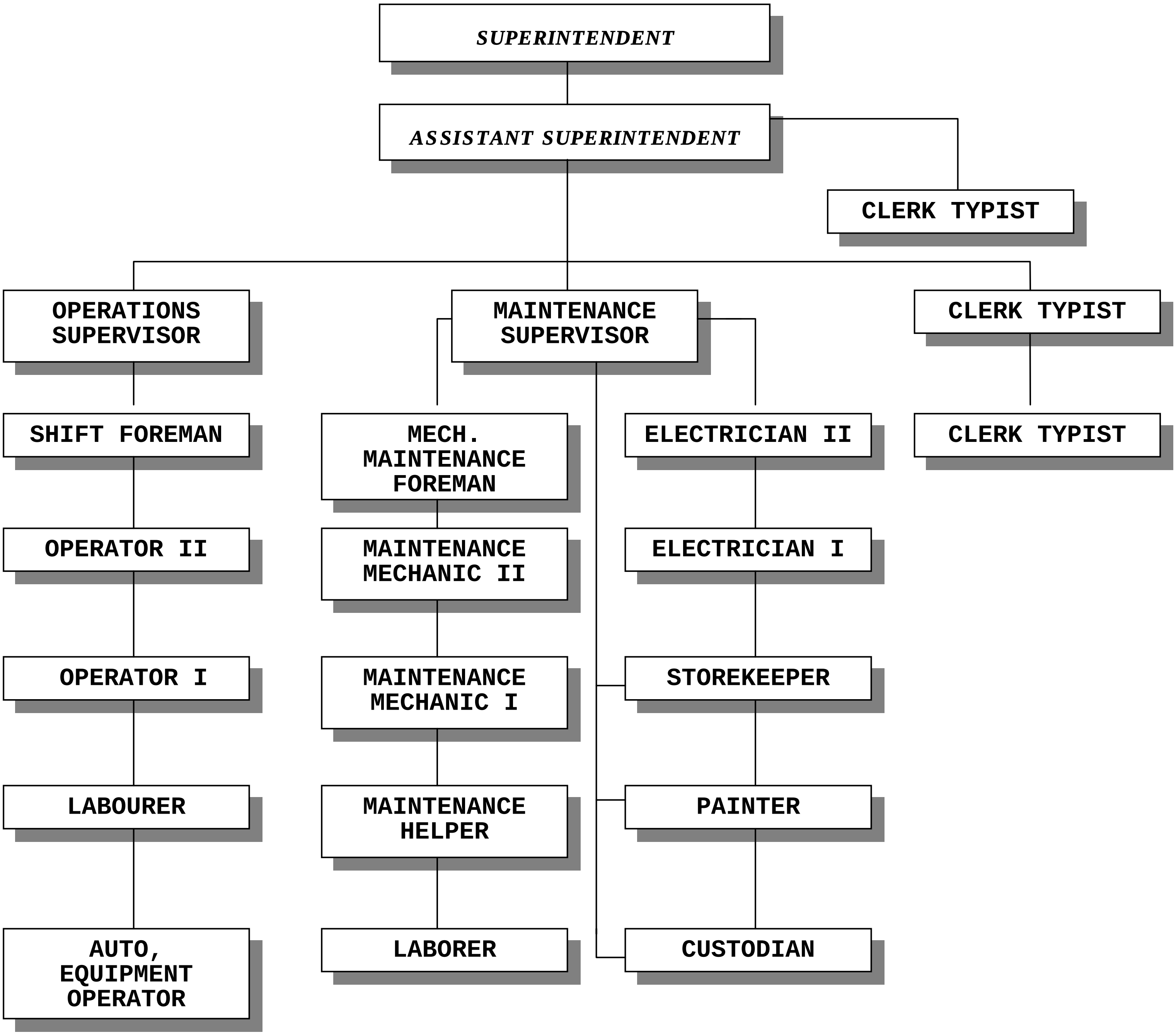
B.2 REQUISITE SKILLS

Individual wastewater treatment personnel may generally be classed into one of the following groups:

(a) Supervisory personnel;
(b) Operating personnel;
(c) Maintenance personnel; or
(d) Laboratory technical.

Figure 2.2 presents an organizational chart for a hypothetical wastewater treatment plant, and outlines these four general classes. This figure also illustrates the relative positions of personnel of differing responsibilities.

FIGURE 2.2
ORGANIZATIONAL CHART
CONVENTIONAL WASTEWATER TREATMENT PLANT



The following skill requirements are minimal for successful performance of specific required duties. These are only a guide and additional requirements for the particular plant location should be analyzed.

- Supervisory Personnel (level of ability depends on size and type of plant - at least high school education or equivalent, should display better than average ability to:

  1. Use and manipulate basic arithmetic and geometry.
  2. Think in terms of general chemistry and physical sciences.
  3. Understand biological and biochemical actions.
  4. Grasp meaning of written communications.
  5. Express thoughts clearly and effectively, both verbally and in writing.

  In addition, supervisory personnel are often responsible for:

  1. Public relations
  2. Bookkeeping
  3. Analysis and presentation of data
  4. Budget requests
  5. Report writing
  6. Personnel
  7. Safety educational program
  8. Contracts, specifications and codes
  9. Estimates and costs
  10. Plant Library

- Laboratory Technicians - require training in laboratory procedures and mathematics

- Operating Personnel - require training in:

  1. Fundamentals of wastewater treatment processes, including chemistry and biology.

- Maintenance Personnel - must be familiar with and capable of:

  1. Mechanical repairs
  2. Electrical and electronic repairs.

B.2.1 Additional Skills Requirement for Collection and Treating System Personnel

In addition to these skills, the Association of Boards of Certification (ABC) skill requirements for collection and treating system personnel has been updated with permission from the Association of Boards of Certification.

B.2.1.1 Introduction

As part of the development of its certification exams, the Association of Boards of Certification (ABC) conducted a job analysis of wastewater treatment operators during 1980 and 1981. The purpose of the job analysis was to identify the essential job tasks performed by wastewater treatment operators and the capabilities required to competently perform these job tasks. The results of this job analysis provided ABC with the foundation for the development of valid wastewater treatment certification exams. These exams were offered by ABC for the first time in 1982.

In 1997, ABC updated the results of its previous job analysis by conducting a national survey of wastewater treatment operators. A new job analysis was conducted to re-evaluate the tasks performed by wastewater treatment operators. The results of the job analysis determine the content of the wastewater treatment certification exams. This includes evaluating existing questions and writing new questions for the certification exams.

The Need-to-Know Criteria was developed from the results of ABC's 1997 wastewater treatment operator job analysis. The information in this document reflects the essential job tasks performed by operators and their requisite capabilities. This document is intended to be used by certification programs and trainers to help prepare operators for entry into the profession.

B.2.1.2 How the Job Analysis was Done

Committee Meetings

A six-member job analysis committee was formed to provide technical assistance in the development of the wastewater treatment operator job analysis. During their first meeting, this committee developed the list of the important job tasks performed by wastewater treatment operators. The committee also verified the technical accuracy, clarity, and comprehensiveness of the job tasks. A second committee meeting was held to identify the capabilities (i.e., knowledge, skills, and abilities) required to perform the job tasks identified during the previous committee meeting. Identification of capabilities was done on a task by task basis, so that a link was established between each task statement and requisite capability. This process resulted in a final list of 288 job tasks and 124 capabilities.

Task Inventory

A task inventory was developed from the data collected during the committee meetings. The inventory included 8-point rating scales for frequency of performance and seriousness of inadequate or incorrect performance. These two rating scales were used because they provide useful information (i.e., how critical each task is and how frequently each task is performed) pertaining to certification.

The task inventory also included a background information section where demographic data such as gender, age, ethnic origin, educational level attained, work experience, and certification level were collected. Space was provided at the end of the inventory for operators to list any important tasks performed on their job which were not included on the inventory and to make general comments.

The task inventory was sent to 524 certified wastewater treatment operators throughout the United States and Canada. Three hundred thirteen out of the 524 inventories mailed were returned for a response rate of 60%. 17% of the respondents were class I operators, 30% were class II operators, 26% were class III operators, and 27% were class IV operators.

Results

The mean, standard deviation, and the percentage of respondents performing each task statement were computed. The mean was used to determine the importance of items and the standard deviation was used to identify items with a wide variation in responses. The percentage of respondents performing each task statement was used to identify tasks and capabilities commonly performed by operators throughout the United States and Canada.

A criticality value of 2(mean seriousness rating) + mean frequency rating was calculated for each item on the inventory. This formula gives extra weight to the seriousness rating in determining critical items and was appropriate because it emphasized the purpose of certification—to provide competent operators.

B.2.1.3 Core Competencies

Tasks and their requisite capabilities performed by at least 50% of the respondents and with a criticality value of 13 or greater were designated as core competencies. They were the most important and commonly performed job tasks and capabilities. The core competencies were considered the essential tasks and capabilities for wastewater treatment operators.

Because the results reflect only those tasks performed by at least 50% of the respondents and with a criticality value of 13 or greater, some frequently performed tasks will be missing from the results. For example, a task may be performed every day but if the potential seriousness of inadequate or incorrect performance is negligible the task will not appear in the results. Because the purpose of certification is to protect the public, it was not reasonable to include tasks of negligible seriousness.

The following table lists the core competencies for wastewater treatment operators. This table is not broken down for each class level of operator. Therefore, there may be job tasks and capabilities that are common in one level and not in others. Because operators may move from one facility to another, they should have a basic understanding of the common job tasks performed at various facilities throughout the United States and Canada.

The core competencies in the table are clustered into eight job duties that are performed by each level of wastewater treatment operator: establish safety plans and apply safety procedures; monitor, evaluate, and adjust treatment processes; evaluate physical characteristics of wastestream; perform and interpret laboratory analyses; operate equipment; evaluate operation of equipment; perform preventive and corrective maintenance; and perform administrative duties. Following each job duty is a listing of the job tasks and capabilities that are associated with that duty.

B.2.1.4 Core Competencies for Wastewater Treatment Operators

Establish safety plans and apply safety procedures

Plans and procedures include:

- Blood borne pathogens
- Chemical hazard communication
- Confined space entry
- Electrical grounding
- Facility upset
- First-aid
- General safety and health
- Lifting
- Lock-out/tag-out
- Personal hygiene
- Personal protective equipment
- Respiratory protection
- Slips, trips, and falls
- Spill response
- Traffic control
- Transportation

Required capabilities:

- Ability to assess likelihood of disaster occurring
- Ability to communicate safety hazards verbally and in writing
- Ability to demonstrate safe work habits
- Ability to follow written procedures
- Ability to identify potential safety hazards
- Ability to recognize unsafe work conditions
- Ability to select and operate safety equipment
- Knowledge of emergency plans
- Knowledge of potential causes and impact of disasters on facility
- Knowledge of safety regulations

Monitor, Evaluate, and Adjust Processes

Processes include:

- Activated sludge
- Chemical addition
- Clarifiers
- Disinfection
- Grit removal
- Pumping of main flow
- Screens
- Solids handling

Required capabilities:

- Ability to adjust chemical feed rates, flow patterns, and process units
- Ability to calculate dosage rates
- Ability to confirm chemical strength
- Ability to evaluate, diagnose, and troubleshoot process units
- Ability to interpret Material Safety Data Sheets
- Ability to maintain processes in normal operating conditions
- Ability to measure and prepare chemicals
- Ability to perform basic math and process control calculations
- Knowledge of biological science
- Knowledge of general chemistry
- Knowledge of general electrical principles
- Knowledge of mechanical principles
- Knowledge of normal chemical range
- Knowledge of personal protective equipment
- Knowledge of principles of measurement

- Knowledge of proper application, handling, and storage of chemicals
- Knowledge of proper lifting procedures
- Knowledge of regulations
- Knowledge of wastewater treatment concepts and treatment processes

Evaluate Physical Characteristics of Wastestream

Characteristics Include:

- Color
- Flow pattern
- Foam
- Mixing pattern
- Odor
- Solids concentration
- Volume

Required capabilities:

- Ability to communicate observations verbally and in writing
- Ability to discriminate between normal and abnormal conditions
- Knowledge of normal characteristics of wastewater

Perform and Interpret Laboratory Analyses

Analyses Include:

- 5-day biochemical oxygen demand
- Ammonia
- Chlorine residual
- Coliform
- Dissolved oxygen
- pH
- Settleable solids
- Temperature
- Total suspended solids
- Volatile suspended solids

Required capabilities:

- Ability to calibrate instruments
- Ability to follow written procedures
- Ability to interpret Material Safety Data Sheets
- Ability to perform laboratory calculations
- Ability to recognize abnormal analytical results
- Knowledge of biological science
- Knowledge of general chemistry
- Knowledge of laboratory equipment and procedures
- Knowledge of normal characteristics of wastewater
- Knowledge of principles of measurement
- Knowledge of proper chemical handling and storage
- Knowledge of quality control and assurance practices
- Knowledge of safety regulations
- Knowledge of sampling procedures
- Knowledge of Standard Methods

Operate Equipment

Equipment Include:

- Backflow prevention devices
- Blowers and compressors
- Chemical feeders
- Computers
- Digesters
- Drives
- Electronic testing equipment
- Engines
- Generators
- Heavy vehicles
- Hydraulic equipment
- Instrumentation
- Motors
- Pneumatic equipment
- Pumps
- Valves

Required capabilities:

- Ability to adjust operation of equipment
- Ability to evaluate operation of equipment
- Ability to monitor operation of equipment
- Knowledge of electrical & mechanical principles
- Knowledge of function of tools
- Knowledge of safety regulations
- Knowledge of start-up and shut-down procedures
- Knowledge of wastewater treatment concepts

Evaluate Operation of Equipment

Characteristics Include:

- Read meters
- Read charts
- Read pressure gauges
- Check speed of equipment
- Measure temperature of equipment
- Inspect equipment for abnormal conditions

Required capabilities:

- Ability to discriminate between normal and abnormal conditions
- Ability to monitor and adjust equipment
- Ability to report findings
- Knowledge of electrical & mechanical principles
- Knowledge of process control instrumentation

Perform Preventive and Corrective Maintenance

Equipment Includes:

- Blowers and compressors
- Chemical feeders
- Generators
- Instrumentation
- Motors
- Pumps

Required capabilities:

- Ability to assign work to proper trade
- Ability to calibrate equipment
- Ability to diagnose and troubleshoot units
- Ability to differentiate between preventive and corrective maintenance
- Ability to discriminate between normal and abnormal conditions
- Ability to follow written procedures
- Ability to perform general maintenance
- Ability to record information

- Knowledge of electrical and mechanical principles
- Knowledge of facility operation and maintenance
- Knowledge of safety regulations
- Knowledge of start-up and shut-down procedures

Perform Administrative Duties

Tasks Include:

- Control employee work activities
- Establish recordkeeping systems for facility
- Plan and organize work activities
- Record information relating to facility performance
- Respond to complaints
- Write internal, state, and federal reports

Required capabilities:

- Ability to determine what information needs to be recorded
- Ability to evaluate employee & facility performance
- Ability to interpret and transcribe data
- Ability to organize information & follow written procedures
- Ability to perform basic math
- Ability to translate technical language into common terminology
- Knowledge of facility operation & maintenance
- Knowledge of monitoring & reporting requirements
- Knowledge of principles of general communication
- Knowledge of principles of management
- Knowledge of principles of public relations
- Knowledge of principles of supervision
- Knowledge of recordkeeping functions & policies
- Knowledge of regulations

B.2.1.5 Wastewater Treatment Certification Exams

The wastewater treatment certification exams evaluate an operator's knowledge of tasks related to the treatment and disposal of wastewater and are intended for the certification of wastewater treatment operators. The content of the exams was determined from the results of the job analysis. To successfully take an ABC exam, an operator must demonstrate knowledge of the core competencies in this document. Because certificates may be used to work in various treatment facilities, the exams may include technologies that are not used in each facility but are commonly used in many facilities.

Four levels of certification are offered by ABC, with class I being the lowest level and class IV the highest level. Each exam consists of 100 multiple-choice questions. The specifications for the exams are based on a weighting of the job analysis results so that they reflect the criticality of tasks performed on the job. The specifications list the percentage of questions on the exam that fall under each job duty. For example, the ABC class I exam consists of 24 questions relating to the job duty "establish safety plans and apply safety procedures" and its associated tasks and capabilities. For a list of tasks and capabilities associated with each job duty, please refer to the list of core competencies on the previous pages.

ABC Wastewater Treatment Exam Specifications

Job Duty	Class I	Class II	Class III	Class IV
Establish safety plans & apply safety procedures	24%	21%	20%	22%
Monitor/evaluate/adjust treatment processes	15%	24%	21%	22%
Evaluate physical characteristics of wastestream	8%	7%	6%	5%
Perform/interpret laboratory analyses	14%	14%	16%	17%
Operate equipment	15%	13%	14%	11%
Evaluate operation of equipment	7%	6%	6%	5%
Perform preventive/corrective maintenance	6%	4%	3%	3%
Perform administrative duties	11%	11%	14%	15%

B.2.1.6 *Suggested References*

The following manuals are recommended for operators interested in learning how to operate wastewater treatment plants.

<u>California State University, Sacramento</u>

- *Operation of Wastewater Treatment Plants*, Volumes 1 and 2
- *Advanced Waste Treatment*
- *Utility Management*

To order, contact Office of Water Programs, California State University, Sacramento, 6000 J Street, Sacramento, CA 95819-6025, phone: (916) 278-6142, fax: (916) 278-5959 or e-mail: wateroffice@csus.edu

<u>Water Environment Federation</u>

- *Operation of Municipal Wastewater Treatment Plants, Manual of Practice No. 11*
- *Design of Municipal Wastewater Treatment Plants, Manual of Practice No. 8*
- *WEF/ABC Certification Study Guide for Wastewater Treatment Personnel*

For more in-depth references on specific aspects of wastewater treatment, please contact the Water Environment Federation for a complete list of Manuals of Practice.

To order, contact Water Environment Federation, 601 Wythe Street, Alexandria, VA 22314-1994, phone: (800) 666-0206, fax: (703) 684-2492 or e-mail: pubs@wef.org.

B.3 JOB DESCRIPTIONS

Job descriptions for the types of personnel commonly employed for the operation and maintenance of conventional wastewater treatment systems are defined in the USEPA Manual "Estimating Costs and Manpower Requirements for Conventional Wastewater Treatment Facilities", contract No. 14-12-462.

A job description for a specific occupation may include details from several of the above categories depending upon the flexibility required. However, a good job description should include but is not necessarily limited to the following:

- List items or processes that an individual must operate.
- State if monitoring of gauges or meters is required.
- Discuss interpreting of any meter or gauge readings for process control actions.
- List any logs or records to be maintained.
- Outline any maintenance duties required.
- State any other title that an individual might carry.
- Discuss decision making requirements.
- State responsibilities and authority given to an individual in the job being described.
- List any report or budget functions that must be performed.
- Discuss any supervisory or inspection functions.

C.1 GENERAL

The requirements for treatment process control will depend on the size of plant and types of process employed. In general, treatment process control should provide safe and efficient manual and automatic operation of all parts of the plant, with minimal operator effort, and all automatic controls should be provided with manual back-up systems.

In making the decisions relating to treatment process control, the following factors should be considered:

- plant size;
- effluent requirements;
- plant process complexity;
- hours in day plant will be manned;
- potential chemical and energy savings with automation;
- reliability of primary devices for parameter measurement;
- preferred location for primary device;
- parameters with useful significance to process;
- equipment which should be controlled manually;
- equipment which should be remotely controlled;
- equipment which should be locally controlled;
- data requiring display at the control centre;
- indication, totalization and recording functions necessary to the overall process.

C.2 REMOTE CONTROL VS. LOCAL CONTROL

Where some parts of a plant may be operated or controlled from a remote location, local control stations should be provided and shall include the provision for preventing operation of the equipment from the remote location. Consideration should be given to providing communication via intercom between remote stations and the local stations. In some cases, the use of television equipment may be justified to provide scanning centres as well as process equipment. Decisions will have to be made by the designer as to which equipment will be controlled locally and which will be controlled from a remote location, and whether control will be automatic or manual.

C.2.1 Supervisory Control and Data Acquisition (SCADA)

At wastewater treatment plants, SCADA systems can be used to control and monitor wastewater collection systems. SCADA systems operate using modems over voice-grade phone lines, radio systems, direct burial cable, or cable TV. If radio-based telemetry systems are used, special attention should be given to the design and layout to eliminate any potential for radio frequency interference (RFI).

Interface to the plant control system ranges from a simple contact closure to a sophisticated digital link with a special protocol requiring special software to be written and supported by the plant control system vendor. Special software should be avoided because it may be difficult to get support from the vendor once the project has been accepted.

C.3 LABORATORY CONTROL

C.3.1 Parameters Requiring Measurement

For proper operation of larger sewage treatment plants, the following parameters should be measured (however for smaller plants some of the parameters could be omitted):

- sewage flow rates, including raw sewage, by-passed flows, and flows through plant subsections (flow trains);
- chlorine dosage and chlorine residual;
- sludge pumpage, including raw, digested sludges and activated sludge return;
- digester supernatant flows;
- chemical dosage;
- digester gas production and utilization;
- anaerobic digester temperature; and
- hazardous gas levels.

Auxiliary instrumentation is desirable to measure the following parameters:

- air flow;
- mixed liquor dissolved oxygen concentrations;
- sludge blanket levels;
- sludge concentrations.

C.3.2 Sampling

C.3.2.1 General

Quantity and quality data is required to effectively control the various unit operations such as pumping, sludge loading on digesters, digester heating, sludge disposal operations and chlorination feed rates. In addition, this data is required to distribute charges for treatment among the various municipal districts and industries involved. The recorded data will also be extremely helpful in the design of future treatment facilities as the plant is expanded. Sampling and testing of the treatment plant effluent will not only provide an indication of plant efficiency but will also ensure that the effluent quality is within acceptable guidelines and will facilitate the calculation of the effect of the effluent on the receiving waters.

C.3.2.2 Sampling Location Points

The location of appropriate sampling points must be established independently for each treatment plant as conditions vary from one plant to another. However, certain general principles are common to all plant sampling surveys and some of these principles are listed below as guidelines to establishing a sampling program:

- Samples should be taken at locations where the wastewater or sludge is as completely mixed as possible;
- Particles greater than one-quarter inch in diameter should be excluded when sampling;

- Any floating materials, growths, deposits, etc., which may have collected at a sampling location should not be included when sampling;
- If samples are to be kept for an hour or more prior to testing, they should be immersed in ice water to retard bacterial action;
- Proper sampling equipment should be provided and safety precautions should be exercised during all sampling;
- Consideration should be given to the relationship between the plant's daily flow variation and detention time through the units so that influent and effluent samples relate to the same waste.

C.3.2.3 Frequency of Sampling

The frequency of sampling will depend upon the variability of the waste stream under consideration as well as practical limitations associated with the treatment plant size, loading, staff and hours of supervision. However, continuing routine sampling to monitor plant performance and effluent quality should be undertaken on a regularly scheduled basis. More intensive sampling and testing may be required to assess unit operation performance and the effect of corrective action in the event of an upset.

It is important to point out that the size of a treatment plant is not necessarily indicative of the number and frequency of tests and analyses performed. Rather this should be determined by the seriousness of the possible effects of the treatment plant effluent on the receiving stream or body of water.

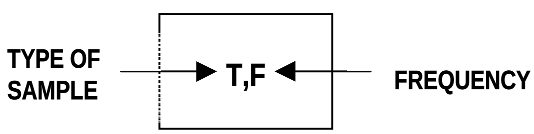
Figure 3.1 presents a sample format for a Laboratory Sampling Program.

FIGURE 3.1
SAMPLE LABORATORY TESTING PROGRAM

	SETTLEABLE SOLIDS	SUSPENDED SOLIDS	BOD	CHLORINE RESIDUAL	GREASE	TOTAL DISSOLVED SOLIDS	COLIFORM ORGANISMS	VOLATILE SUSPENDED SOLIDS	DISSOLVED OXYGEN	TOTAL SOLIDS	TOTAL VOLATILE SOLIDS	pH
RAW SEWAGE	C,D	C,D	C,D			C,W		C,D		C,W	C,W	
PRIMARY EFFLUENT	C,D	C,D	C,D					C,D		C,W	C,W	
SECONDARY EFFLUENT	C,D	C,D	C,D		C,W			C,D		C,W	C,W	
CHLORINE CONTACT TANK				G,E								
MIXED LIQUOR		C,D										
PLANT EFFLUENT			C,W	G, D2		C,W	C,W		G,D			
RAW SLUDGE										C,W	C,W	C,W
DIGESTED SLUDGE										G,W	G,W	G,W

TYPE OF SAMPLE
C: Composite Sample
G: Grab Sample

FREQUENCY
D: Daily
W: Weekly
D2: Twice daily
E: Every Four Hours



C.3.3 Tests and Procedures

Although wastewater treatment plants may vary in size and degree of treatment, there are specific basic tests that are applicable to any plant which provide information required for process control. The following list places in order of importance the samples and analyses required within these plants.

Influent or Raw Sewage

a) settleable solids
b) total solids
c) suspended solids

d) volatile suspended solids
e) BOD
f) COD
g) pH
h) phosphates
i) ammonia
j) total kjeldahl nitrogen (TKN)
k) nitrates
l) chlorides

Grit

a) moisture content
b) dry solids
c) volatile solids
d) sieve tests

Primary Effluent

a) total solids
b) suspended solids
c) volatile suspended solids
d) BOD
e) pH
f) COD
g) total phosphate
h) orthophosphate

Aeration Section

a) half-hour settling test of mixed liquor
b) suspended solids in mixed liquor
c) volatile suspended solids in mixed liquor
d) sludge volume index
e) dissolved oxygen
f) pH
g) solids in return and waste activated sludge
h) oxygen uptake rate

Secondary Effluent

a) total solids
b) suspended solids
c) volatile suspended solids
d) BOD
e) pH
f) COD
g) total phosphate
h) orthophosphate

Lagoon Contents

a) DO
b) temperature
c) pH

For lagoons and oxidation ponds it is most important that careful observation of the condition of the lagoon should be noted and recorded, particularly the presence of colour, algae or odours.

Chlorine Contact Tank

a) chlorine residual
b) fecal coliform bacterial count

Final Effluent

a) total solids
b) suspended solids
c) volatile suspended solids
d) BOD
e) chlorine residual
f) fecal coliform bacterial count
g) dissolved oxygen (DO)
h) pH
i) COD
j) total phosphate
k) orthophosphate
l) ammonia

Raw Sludge

a) pH
b) dry solids
c) volatile solids

Waste Activated Sludge Thickening

a) solids in feed sludge
b) solids in discharge sludge
c) suspended solids in filtrate or centrate
d) percent volatile in suspended solids of filtrate or centrate

Digested Sludge and Digester Supernatant

a) pH
b) total solids
c) volatile solids
d) volatile acids
e) alkalinity

Digester Gas

a) percent methane
b) gas production

Cake from Vacuum Filter or Centrifuge

a) total solids
b) volatile solids
c) phosphates
d) nitrates

Filtrate or Centrate

a) pH
b) total solids
c) suspended solids
d) volatile suspended solids

Incinerator Ash

a) dry solids
b) volatile solids

C.4 PROCESS CONTROL TECHNIQUES

There are two main types of process control techniques within a wastewater treatment plant. These include manual control and on-line control. Under the manual control system there is limited automatic control and the operator is responsible for decisions and actions. On-line control involves a multi-purpose computerized system with limited scope for modification or a dedicated purpose system with standard hardware and customized software.

Whether process control involves manual or on-line control, or a combination of both, the operation and maintenance manual shall fully describe specific process control techniques.

C.5 OWNER/OPERATOR RESPONSIBILITY

The owner/operator of a wastewater treatment or collection facility shall be responsible for the sampling and analysis requirements for the proper operational control of the facility. These requirements shall be in accordance with Operations Section 3, and shall ensure the proper control of day-to-day operations of the system.

C.6 REGULATORY AGENCIES' RESPONSIBILITY

The regulatory agencies are only responsible for compliance enforcement. They shall not be responsible for any aspect of process control at any wastewater treatment or collection facility.

C.7 REFERENCES

The following is a list of references which will assist operating staff in performing the necessary sampling, laboratory and control procedures to effectively operate a treatment system:

1. "Standard Methods for the Examination of Water and Sewage," APHA, AWWA, WPCF;
2. EPA publication 6003 "Methods for Chemical Analysis of Water and Wastes";

3. WPCF Manual of Practice No. 18, "Simplified Laboratory Procedures for Wastewater Examination";

4. WPCF Manual of Practice No. 11, "Operation of Wastewater Treatment Plants";

5. "Laboratory Procedures for Wastewater Treatment Plant Operators", New York State Department of Health;

6. "Manual of Instruction for Sewage Treatment Plant Operators", New York State Department of Health;

7. "Chemistry for Sanitary Engineers", Sawyer, McGraw-Hill;

8. "Methods for Chemical Analysis of Water and Wastewater", EPS Surveillance Report 5-AR-73-16;

9. EPA Publication 6001 "Handbook for Analytical Quality Control in Water and Wastewater Laboratories";

10. EPA Publication, "Procedures for Evaluating Performance of Wastewater Treatment Plants", Contract No. 68-01-0107;

11. EPA Publication, "Estimating Laboratory Needs for Municipal Facilities", Contract No. 68-01-0328.

12. "Manual on Wastewater Sampling Practice", The Canadian Institute on Pollution Control;

13. EPA Publication, "Performance Evaluation and Troubleshooting at Municipal Wastewater Treatment Facilities", Contract No. 68-01-4418;

14. EPA Publication, "Process Control Manual for Aerobic Biological Wastewater Treatment Facilities", EPA-430/9-77-006.

D.1 USE OF MANUALS

The purpose of an O & M Manual is to give treatment system personnel the proper understanding, techniques and references necessary to efficiently operate their facilities. The O & M Manual should help to ensure the performance record of a treatment system remains high. The manual should thus serve as a tool for operating and maintenance personnel of the plant.

D.2 RECOMMENDED FORMAT

D.2.1 General

The formats presented in this section are intended to be a flexible guide for the preparation of an O & M Manual for a wastewater treatment system and wastewater pumping stations and/or pipelines. They can be modified to fit the particular system at hand. It is anticipated that these formats can be used in most cases. If manual preparation follows these formats, the review process will be greatly accelerated.

Each of the twelve (12) chapters and the appendices in the suggested guide is addressed in the reference manual: "Consideration for Preparation of Operation and Maintenance Manuals" (US EPA 430/9-74-001). Detailed descriptions of the type information required in that respective chapter of the O & M Manual are given.

It should be remembered that the O & M Manual will provide assistance in developing standard operating procedures for each system. The adequacy of these procedures plays a major role in determining how well the system will operate. The O & M Manual should provide the necessary information to insure these standard operating procedures can be readily developed. Once an acceptable set of procedures has been established, the O & M Manual becomes a reference book for the entire treatment system.

D.2.2 Suggested Guide and Checklist for an Operation and Maintenance Manual for Municipal Wastewater Treatment Facilities

Chapter I - Introduction

Manual User Guide
Table of Contents

A. Operation and managerial responsibility

1. Operator responsibility

   a) General - outline responsibilities

      (1) Know proper operational procedures
      (2) Keep accurate records
      (3) Properly manage operating funds
      (4) Keep supervisors informed
      (5) Keep informed of current O & M practices.

b) List short courses available.

c) Provide suggested list of journals/periodicals related to municipal wastewater treatment.

2. Treatment system management responsibility - outline responsibilities.

a) Maintain efficient plant operation and maintenance

b) Maintain adequate records

c) Establish staff requirements, prepare job descriptions and assign personnel

d) Provide good working conditions

e) Establish operator training program

f) Provide incentives for employees

g) Maintain good public relations

h) Prepare budgets and reports

i) Plan for future facility needs

j) Develop standard operating procedures.

B. Type of treatment and treatment requirements/effluent limitations

1. Type of treatment - Describe major process

a) Primary
b) Secondary - RBC, trickling filter
c) Secondary - activated sludge
d) Other

2. Treatment requirements/effluent limitations - state whether monthly or yearly averages are used

a) Biochemical oxygen demand (BOD)
b) Suspended solids concentrations
c) pH
d) Other

C. Description of plant type and flow pattern

1. Plant type - Briefly describe individual units

a) Pretreatment
b) Primary treatment

c) Secondary treatment
d) Tertiary Treatment
e) Disinfection
f) Sludge handling

2. Flow Pattern

a) Include a basic flow diagram
b) Bypasses and alternate flow paths can generally be omitted from this introductory diagram.

Chapter II Permits and Standards

Table of Contents

A. Discharge permit and permit requirements

1. Give permit number
2. Give renewal date if applicable
3. List permit requirements
4. Include permit application guidelines
5. Copy of permit sections dealing with municipal wastewater discharge permits should be included

B. Reporting procedure for spills of raw or inadequately treated wastewater.

1. Include copies of permit sections requiring reporting of bypass/spill condition.

a) Discuss owner's responsibilities
b) Discuss penalties

2. Outline reporting procedure to include telephone numbers and sample report format.

C. Water Quality Standards

1. Include a copy of Provincial quality standards for receiving waters of treatment plant's effluent
2. Include a copy of Provincial receiving waters classification system.

Chapter III. Description, Operation and Control of Wastewater Treatment Facilities

Table of Contents

A. General - Each major wastewater treatment unit/process should be discussed separately with respect to the following considerations:

1. Description

   a) Provide a brief general description with each major treatment unit/process discussed.

      1) Pretreatment
      2) Primary sedimentation
      3) Biological process
      4) Secondary sedimentation
      5) Disinfection
      6) Other

   b) The description should physically trace the wastewater through the unit/process and comment on design efficiency.

2. Relationship to adjacent units

   a) Give type and function of any or all preceding units/processes as they relate to unit/process being considered.

   b) Give type and function of any or all following units/processes as they relate to unit/process being considered.

3. Classification and Control

   a) Classification - Briefly describe relation to similar units/processes

      1) Standard/conventional
      2) Modified
      3) Other

   b) Control - give methods of controlling unit/process

      1) Flow to plant
      2) Recirculation pumps
      3) Air supply
      4) Sludge return/wasting rates
      5) Other (physical and process controls)

4. Major components

   a) List all components within the unit/process
   b) List all major mechanical equipment items within the Unit/process
   c) Other

5. Common operating problems

   a) State problems that might occur in unit/process
   b) List probable causes
   c) Discuss control/prevention techniques

6. Laboratory Controls

   a) List tests and give expected ranges for test results
   b) Give relation between test results and treatment unit/process operation

7. Start-up - give start-up technique

B. Specific Plant Operation

1. Normal Operation

   a) Discuss the normal operation of each unit/process. This discussion should include the following information as it may apply to the particular unit/process

      1) Valve positions
      2) Sluice gate settings
      3) Weir elevations
      4) Sludge rake speeds
      5) Pump settings
      6) Recirculation rates
      7) MLSS concentrations
      8) Other

2. Alternate Operation

   a) List alternate modes of operation
   b) Provide discussion and schematics to illustrate alternate operations.

3. Emergency Operations and Failsafe Features

   a) Discuss emergency operating procedures for potential emergency conditions
   b) List failsafe features
   c) Describe operation of failsafe features.

Chapter IV Description, Operation and Control of Sludge Handling Facilities

Table of Contents

A. General - each major sludge handling unit/process should be discussed separately with respect to the following considerations:

1. Description

   a) Provide a brief general description with each major unit/process discussed.

      1) Concentration/thickening
      2) Digestion
      3) Conditioning
      4) Dewatering/drying
      5) Incineration
      6) Wet oxidation
      7) Disposal
      8) Other

   b) The description should physically trace the sludge through the unit/process and comment on how the character of the sludge is altered.

2. Relationship to adjacent units

   a) Describe type and function of any or all preceding units/processes as they relate to unit/process being considered

   b) Describe type and function of any or all following units/processes as they relate to process being considered.

3. Classification and control

   a) Classification - Describe relation to similar units/processes

      1) Standard/conventional
      2) Modified
      3) Other

   b) Control - Give methods of controlling unit/process

      1) Recirculation pumps
      2) Aerobic digestion air supply
      3) Conditioning chemicals
      4) Temperature
      5) Other

4. Major components

    a) List all components within the unit/process
    b) List all major mechanical equipment items within the unit/process
    c) Other

5. Common operating problems

    a) State problem that might occur in unit/process
    b) List probable causes
    c) Discuss control/prevention techniques

6. Laboratory controls

    a) List tests and give expected ranges for test results
    b) Give relation between test results and treatment process operation

7. Start-up - give start-up techniques

B. Specific Plant Operation

1. Normal operation

    a) Discuss the normal operation of each unit/process. This discussion should include the following information as it may apply to the particular unit/process

        1) Valve positions
        2) Heat requirements
        3) Sludge blanket depths
        4) Sludge pumping schedule
        5) Sludge collector/stirring speeds
        6) Vacuum filter hours of operation
        7) Other

2. Alternate Operation

    a) List alternate modes of operation
    b) Provide discussion and schematics to illustrate alternate operations

3. Emergency operations and failsafe features

    a) Discuss emergency operating procedures for potential emergency conditions
    b) List failsafe features
    c) Describe operation of failsafe features.

Chapter V - Personnel

Table of Contents

A. Manpower Requirements/Staff - List personnel required

1. Supervisors
2. Administrative
3. Operational
4. Maintenance

B. Qualifications

1. For each job title give:

   a) Training
   b) Experience
   c) Skills required
   d) Certificate required

C. Certification Program

1. Include copy of permit section regarding training courses required.

2. Discuss pertinent aspects of operator certification as they apply to the facility at hand.

Chapter VI. Laboratory Testing

Table of Contents

A. Purpose - to discuss purpose of laboratory testing

1. Essential to treatment process control
2. Provides an operating record for treatment system
3. Aids in problem analysis and prevention.

B. Sampling

1. Give grab sample definition
2. Give composite sample definition
3. Outline a sampling program for the treatment system

C. Laboratory References - List pertinent references

1. WPCF MOP No. 18, Simplified Laboratory Procedures for Wastewater Examination
2. Process Control Laboratory Course, WPCF and Environment Canada

3. Standard Methods for the Examination of Water and Sewage
4. Other

D. Interpretation of Laboratory Tests - give brief definition and sanitary engineering application for all tests

1. pH
2. Dissolved oxygen (DO)
3. Biochemical oxygen demand (BOD)
4. Settleable and suspended solids - discuss importance of solids balance
5. Chlorine residual
6. Other

E. Sample Laboratory Worksheets - give instructions for completing sample forms

1. Solids determinations
2. BOD determinations
3. Other

Chapter VII. Records

Table of Contents

A. Process Operations/Daily Operating Log - provide sample form and discuss features

1. Weather conditions
2. Facility influent flow
3. Recirculation rate
4. Grit removed
5. Sludge handling data
6. Status of secondary treatment process
7. Operators on duty
8. Complaints
9. Plant visitors
10. Power consumption
11. Chemicals used
12. Unusual conditions (operational and maintenance)
13. Routine operational duties

B. Laboratory - Comprehensive discussion of laboratory records should be included under laboratory controls chapter of the manual

C. Monthly Report to Provincial Agencies

1. Provide sample form
2. Give instructions for completing
3. Outline techniques for maximum utilization of forms to eliminate using any supplemental forms
4. Tell when and where to submit completed forms

D. Annual Report

1. Designate individual responsible for preparing report
2. State whether calendar or fiscal year summary
3. Give sample report format:
    a) Annual summary of operating data
    b) Annual summary of management data
    4. Provide coordinating instructions with financial arm of parent governmental body

E. Maintenance - Comprehensive discussion of maintenance records should be included under maintenance chapter of manual

F. Operating Costs and Record Keeping - list and discuss each major cost group and record keeping procedure for each

1. Labour:
    a) Operation
    b) Administration
    c) Maintenance

2. Utilities:
    a) Electricity
    b) Fuel oil
    c) Potable water
    d) Telephone
    e) Other

3. Chemicals (Process only):
    a) Lime
    b) Alum
    c) Chlorine
    d) Other

4. Supplies:
    a) Laboratory chemicals
    b) Cleaning materials
    c) Maintenance materials
    d) Other expendable items

G. Personnel Records

H. Emergency Conditions Record

1. Bypass report
2. Deteriorated effluent record
3. Other

Chapter VIII. Maintenance

Table of Contents

E. Maintenance Personnel

1. Outline maintenance staff
2. Review maintenance staff capabilities and limitations

F. Cost and Budgets for Maintenance Operations

1. Discuss importance of separation of maintenance costs:
   a) Preventive maintenance
   b) Corrective maintenance
   c) Major repairs or alterations

2. Suggest a cost accounting system for storeroom stock, special purchase items and man-hours
3. Other

G. Miscellaneous Maintenance Records

1. Provide sample preventive/corrective maintenance log
2. Give breakdown report format
3. Other

H. Housekeeping - discuss housekeeping activities

1. Yard work
2. Painting
3. General Cleaning
4. Other

I. Special Tools and Equipment

1. Outline tool room procedures:
   a) Tool inventory
   b) Tool check control system

2. Discuss use of tool boards:
   a) Special/frequently used tools
   b) Location of boards

3. Give maintenance skills required for all special tools

J. Lubrication

1. Give lubrication specifications
2. Provide interchangeable lubricants chart
3. Discuss use of color coded lubrication tags for all equipment
4. Give sample consumption/inventory records
5. Outline sample lubrication route

K. Major Equipment Information

1. List all major equipment items:
   a) Comminutors
   b) Grit chambers
   c) Sedimentation tanks
   d) Aerators
   e) Pumps
   f) Digesters
   g) Drying beds
   h) Lagoons
   i) Other

2. Outline basic maintenance considerations for all major electrical and mechanical equipment items
3. Outline procedure for ordering parts/components or new items

L. Warranty Provisions

1. List all guaranteed equipment
2. Give guarantee period for each piece of equipment
3. Discuss pertinent features of each guarantee

M. Contract Maintenance

1. Provide list of suggested contract jobs
2. Provide list of suggested contractors

Chapter IX. Emergency Operating and Response Program

Table of Contents

A. Give results of vulnerability analysis of system
B. List methods to reduce system vulnerability
C. List mutual aid agreements
D. Include emergency equipment inventory
E. Give method of preserving treatment system records
F. Include industrial waste inventory/monitoring system
G. Give coordinating instructions for local police and fire departments
H. Define responsibilities of treatment system personnel
I. Designate an emergency response center
J. List auxiliary personnel requirements
K. Provide a mechanism for ensuring plans updated periodically

Chapter X. Safety

Table of Contents

A. General

1. Management's responsibility - discuss responsibilities:
   a) Communicate safety information to employees
   b) Eliminate hazardous working conditions
   c) Motivate employees to be safety minded
   d) Other

2. Emergency telephone numbers - provide a list of all numbers:
   a) Hospital
   b) Fire station
   c) Ambulance Service
   d) Chlorine supplier
   e) Other

B. Sewers - discuss safety aspects of sewer maintenance

1. Work site protection
2. Gas testing equipment
3. Non-sparking tools
4. Other

C. Electrical Hazards

1. Discuss grounding of electric tools
2. Outline first aid for electric shock victim
3. Designate authorized personnel to perform electrical repairs
4. Other

D. Mechanical Equipment Hazards

1. Discuss equipment guards
2. Discuss noise level considerations
3. Designate authorized personnel to perform mechanical repairs
4. Other

E. Explosion and Fire Hazards

1. Discuss storage of flammable materials
2. Give type and location of fire extinguishers
3. Discuss use of flammable vapor detectors
4. Outline hazards associated with digester gases
5. Other

F. Bacterial Infection (Health Hazards)

1. State policy on tetanus shots
2. Outline personal hygiene considerations
3. State policy on care of cuts and other injuries
4. Other

G. Chlorine Hazards

1. Discuss cylinder handling
2. Outline procedure for testing for and responding to leaks
3. Describe self-contained breathing apparatus use
4. Other

H. Oxygen Deficiency and Noxious Gases

1. Outline noxious gas testing procedures
2. Discuss ventilating equipment
3. Provide tabulation of common gases encountered in wastewater treatment systems
4. Other

I. Laboratory Hazards

1. Discuss volatile materials handling
2. Describe protective clothing and devices
3. Discuss proper ventilation
4. Other

J. Safety Equipment - list safety equipment required

1. First aid kits
2. Fire extinguishers
3. Gas masks/air packs
4. Protective clothing and hard hats
5. Safety harnesses
6. Other

K. Process Chemical Handling - discuss procedures for all chemicals used

1. Alum
2. Lime
3. Ferric Chloride
4. Ferrous Sulfate

L. References - list pertinent safety references

1. WPCF MOP #1 - Safety in Wastewater Works
2. WPCF MOP #18 - Operations of Wastewater Treatment Plants
3. Chlorine Institute, Chlorine Manual

4. EPA Manual - Safety in the Design, Operation and Maintenance of

wastewater treatment Works, Contract No. 68-01-0324
5. Other

Chapter XI. Utilities

Table of Contents

A. General

1. Give name of utility company
2. List contact men within utility company:
   a) Routine contact
   b) Emergency contact
3. Discuss reliability of service
4. Give any cost information available

B. Electrical

1. Give voltage of service adjacent to facility
2. Give reduced voltage entering facility
3. Discuss stand-by power from second source

C. Telephone

1. Outline telephone communications system within treatment system
2. Discuss any alarm systems that utilize telephone wires

D. Natural Gas

1. Give cubic feet of gas per hour
2. Give normal operating pressure
3. Give size of gas line

E. Water

1. Give size of waterline
2. Give normal operating pressure
3. Discuss any backflow preventer prevention systems present

F. Fuel Oil

1. List capacities of storage tanks
2. Outline program to insure adequate supplies of fuel oil are always on hand
3. List potential suppliers

Chapter XII. Electrical System - describe the Electrical System

Table of Contents

Appendices

Table of Contents

1. Function
2. Type/size
3. Location
4. Identification

C. Sample Forms - provide as required

1. Daily Operating Log
2. Equipment Data Cards
3. Maintenance Work Order
4. Purchase Order
5. Accident Report Form
6. Provincial Reports
7. Other

D. Chemicals Used in Plant

1. List all chemicals
2. Give safety precautions and outline storage considerations in Safety Chapter of Manual
3. List suppliers
4. Provide reorder schedule

E. Chemicals Used in Laboratory

1. Give common name
2. Give chemical formula
3. List suppliers

F. Emergency Operating and Response Program - provide as required

1. Schematic diagrams
2. Sample forms

G. Detailed Design Criteria - tabulate criteria

1. Population served
2. Wastewater volume/strength:
   a) Present/future
   b) Domestic
   c) Industrial
3. Quantities of screenings, grit and sludge removed per thousand cubic meters of wastewater treated
4. Unit sizes and capacities
5. Hydraulic and organic loadings
6. Detention times
7. Pumping characteristics
8. Sludge treatment and disposal data

H. Equipment Suppliers

1. Give name
2. List equipment furnished
3. Give reference to where detail information on representatives can be found in manual

I. Manufacturer's Manuals

1. May be bound separately
2. Manuals should give adequate operating and maintenance instructions
3. Manuals should be indexed/cross-referenced

J. Sources for Service and Parts

1. List service organizations for all equipment

2. List local repair services:
   a) Meter repair
   b) Motor rewinding
   c) Other

3. List local parts sources:
   a) Plumbing wholesalers
   b) Electrical wholesalers
   c) Mill Supply Houses
   d) Other

K. As-Built Drawings

1. Ensure drawings are complete and accurate
2. Cross-reference with shop drawings

L. Approved Shop Drawings

1. Index adequately
2. Cross-reference with engineering drawings and construction specifications

M. Dimension Prints

1. Provide when necessary to show units relation to other units, adjacent walls, etc.
2. Use to tie shop drawings to engineering drawings

N. Construction Photos

1. Label and date all photos
2. Outline photo indexing system

O. Warranties and Bonds

1. Provide copies
2. Index properly

P. Copies of Provincial Reporting Forms - provide as required

1. Monthly Operating Report
2. Bypass Report
3. Chlorine Failure Report
4. Other

Q. Copies of Provincial Inspection Forms - provide as required

R. Infiltration Controls

1. Provide copy of existing ordinance
2. Provide model ordinance if none exists

S. Industrial Waste Controls

1. Provide copy of existing ordinance
2. Provide model ordinance if none exists

T. Piping Color Codes

1. List color for each piping system
2. State if directional flow arrows and/or labelling required

U. Painting

1. Give type of coating required for each unit
2. Give painting frequency schedule
3. Provide a copy of Water Pollution Control Federation, MOP-17, "Paints and Protective Coatings", (1969)

V. References to be maintained at treatment facility

1. MOP #1
2. MOP #11
3. Suggested references for detailed study of process utilized
4. Other

D.2.3 Suggested Guide and Checklist for an Operation and Maintenance Manual for Municipal Wastewater Pumping Stations and/or Pipelines

Chapter I - Introduction

Manual User Guide

Table of Contents

A. Operation and Managerial responsibility

1. Operator responsibility:

   a) General - outline responsibilities:
      (1) Know proper operational procedures
      (2) Keep accurate records
      (3) Properly manage operating funds
      (4) Keep supervisors informed
      (5) Keep informed of current O & M practices

   b) List short courses and operator schools available

   c) Provide suggested list of journals/periodicals related to municipal wastewater treatment

2. Treatment system management responsibility -outline responsibilities:

   a) Maintain efficient plant operation and maintenance

   b) Maintain adequate records

   c) Establish staff requirements, prepare job descriptions and assign personnel

   d) Provide good working conditions

   e) Establish operator training program

   f) Provide incentives for employees

   g) Maintain good public relations

   h) Prepare budgets and reports

   i) Plan for future facility needs

   j) Develop standard operating procedures

B. Description of pumping stations and/or pipeline type

1. Pumping station type - describe station type:

   a) Municipal wastewater
   b) Storm water runoff
   c) Industrial wastes
   d) Combined municipal and storm water
   e) Sludge
   f) Treated municipal wastewater

2. Pumping station classification - discuss how station is classified:
   a) Capacity (gpm, mgd)
   b) Energy source (Primary and Stand-by):
      (1) Electric
      (2) Diesel
      (3) Steam
      (4) Other
   c) Construction method

3. Discuss pumping station chlorination facilities

4. Pipeline types and sizes - describe pipeline:
   a) Asbestos-cement
   b) Brick masonry
   c) Clay
   d) Concrete
   e) Iron and Steel:
      (1) Cast iron
      (2) Ductile iron
      (3) Fabricated steel

5. Describe type of joint used

6. Discuss pipeline appurtenances and special structures:
   a) Manholes
   b) Check valves and relief overflows
   c) Siphons
   d) Flap gates
   e) Metering stations
   f) Air relief valves
   g) Other

Chapter II. Permits and Standards

Table of Contents

A. Permit and permit requirements

1. Give permit number
2. Give renewal date if applicable
3. List permit requirements
4. Include permit application guidelines
5. Copy of permit sections dealing with pumping station permits should be included

B. Reporting procedure for spills of raw or inadequately treated wastewater

1. Include copies of permit sections requiring reporting or bypass/spill condition:
   a) Discuss owner's responsibilities

b) Discuss penalties

2. Outline reporting procedure to include telephone numbers and sample report format

C. Water Quality Standards for adjacent water courses

1. Include copy of Provincial Quality Standards for any water courses adjacent to pumping stations or pipelines, where there is a potential for a spill of raw wastewater
2. Include copy of Provincial receiving waters classification system

Chapter III. Description, Operation and Control of Pumping Stations and/or Pipelines

Table of Contents

A. General

1. Pumping station description - provide a brief general description of the pumping station:
   a) Typical
   b) Package
   c) Pneumatic-ejector
   d) Other

2. Pipeline description - provide a brief general description of the pipeline:
   a) Gravity
   b) Force Main

3. Pumping station major components - list major components:
   a) Pumps
   b) Suction and discharge piping
   c) Wet Well
   d) Automatic Controls
   e) Other

4. Pipeline major components - list major components:
   a) Pipe
   b) Manholes
   c) Siphons
   d) Metering Stations
   e) Other

5. Pumping Station and/or Pipelines - common operating/maintenance problems:

   a) state problems
   b) list probable causes
   c) give control/prevention techniques

6. Pumping Station and/or Pipelines start-up - give start-up techniques

B. Specific Pumping Station and/or Pipeline Operation

1. Normal Operation:
   a) Discuss the normal operation of each type of pumping station and/or pipeline
      (1) Pump settings
      (2) Valve positions
      (3) Flow meter settings
      (4) Chlorination system
      (5) Other

2. Alternate Operation:
   a) List alternative modes of operation
   b) Provide discussion and schematics to illustrate alternate operation

3. Emergency Operations and Failsafe Features:
   a) Discuss emergency operating procedures for potential emergency conditions
   b) List failsafe features
   c) Describe operation of failsafe features

Chapter IV. Personnel

Table of Contents

A. Manpower requirements/staff - personnel required

1. Supervisors
2. Administrative
3. Operational
4. Maintenance

B. Qualifications

1. For each job title give:
   a) Training
   b) Experience
   c) Skills required
   d) License/certificate required

C. Certification Program

1. Include copy of permit section regarding training courses required
2. Discuss pertinent aspects of operator certification as apply to the facility at hand

Chapter V. Records

Table of Contents

A. Process Operations/Daily Operating Log - provide sample form and discuss features

1. Routine operational duties
2. Power consumption
3. Unusual conditions
4. Chemicals used
5. Other

B. Monthly Report to Provincial Agencies

1. Provide sample form
2. Give instructions for completing form
3. Outline techniques for maximum utilization of forms to eliminate using any supplemental forms
4. Tell when and where to submit completed form

C. Annual Report

1. Designate individual responsible for preparing report
2. State whether calendar or fiscal year summary
3. Give sample report format:
   a) Annual summary of operating data
   b) Annual summary of management data

4. Provide coordinating instructions with financial arm of parent Governmental body

D. Maintenance - comprehensive discussion of maintenance records should be included under maintenance chapter of the manual

E. Operating Costs and Record Keeping - list and discuss each major cost group and record keeping procedures for each

1. Labour:
   a) Operation
   b) Adminstration
   c) Maintenance

2. Utilities:
   a) Electricity
   b) Fuel oil
   c) Potable water
   d) Telephone
   e) Other

3. Chemicals:
    a) Chlorine
    b) Lime
    c) Other

4. Supplies:
    a) Cleaning materials
    b) Maintenance materials
    c) Other expendables

F. Personnel Records

G. Emergency Conditions Record

1. Bypass Report
2. Other

Chapter VI. Maintenance

Table of Contents

A. General

1. State purpose of maintenance system

2. Outline scope of recommended maintenance system

3. List basic features:
    a) Equipment record system
    b) Planning and scheduling
    c) Storeroom and inventory system
    d) Maintenance personnel
    e) Costs and budgets for maintenance operations

B. Equipment Record System

1. Describe equipment numbering system
2. Outline equipment catalog
3. Discuss the type information and equipment data which should be maintained
4. Provide instructions on preparing and fling information in the record system
5. Describe data card retrieval system
6. Provide completed equipment name-plate data cards for each item of equipment
7. Other

C. Planning and Scheduling

1. Provide guidelines for preventive maintenance and corrective maintenance tasks
2. Describe schedule chart board

3. Outline work order system:
    a) Provide sample forms
    b) Describe work order log
4. Discuss contract maintenance work
5. Other

D. Storeroom and Inventory System

1. Recommend spare parts/components to be maintained
2. Outline stockroom inventory procedures:
    a) Numbering system for all items
    b) Sample withdrawal slip
    c) Maximum/minimum quantities to be maintained
    d) Record system

3. Discuss purchase orders
4. Other

E. Maintenance Personnel

1. Outline maintenance staff
2. Review maintenance staff capabilities and limitations

F. Cost and Budgets for Maintenance Operations

1. Discuss importance of separation of maintenance costs:
    a) Preventive maintenance
    b) Corrective maintenance
    c) Major repairs or alterations
2. Suggest a cost accounting system for storeroom stock, special purchase items and man-hours
3. Other

G. Miscellaneous Maintenance Records

1. Provide sample preventive/corrective maintenance log
2. Give breakdown report format
3. Other

H. Housekeeping - discuss housekeeping activities

1. Yard work
2. Painting
3. General cleaning
4. Other

I. Special Tools and Equipment

1. Outline tool room procedures:
    a) Tool inventory
    b) Tool check control system

2. Discuss use of tool boards:
    a) Special/frequently used tools
    b) Location of boards
3. Give maintenance skills required for all special tools

J. Lubrication

1. Give lubrication specifications
2. Provide interchangeable lubricants chart
3. Discuss use of color coded lubrication tags for all equipment
4. Give sample consumption/inventory records
5. Outline sample lubrication route

K. Major Equipment Information

1. List all major equipment items
2. Outline basic maintenance considerations for all major electrical and mechanical equipment items

L. Warranty Provisions

1. List all guaranteed equipment
2. Give guarantee period for each piece of equipment
3. Discuss pertinent features of each guarantee

M. Contract Maintenance

1. Provide list of suggested contract jobs
2. Provide list of suggested contractors

Chapter VII. Emergency Operating and Response Program

Table of Contents

A. Give Results of Vulnerability Analysis of System
B. List Methods to Reduce System Vulnerability
C. List Mutual Aid Agreements
D. Include Emergency Equipment Inventory
E. Give Method of Preserving Treatment System Records
F. Include Industrial Waste Inventory/Monitoring System
G. Give Coordinating Instructions for local Police and Fire Departments
H. Define Responsibilities of Treatment System Personnel
I. Designate an Emergency Response Center
J. List Auxiliary Personnel Requirements
K. Provide a Mechanism for ensuring Plan is Updated periodically

Chapter VIII. Safety

Table of Contents

A. General

1. Management's responsibility - discuss responsibilities:
   a) Communicate safety information to employees
   b) Eliminate hazardous working conditions
   c) Motivate employees to be safety minded
   d) Other

2. Emergency telephone numbers - provide a list of all numbers:
   a) Hospital
   b) Fire station
   c) Ambulance service
   d) Chlorine supplier
   e) Other

B. Sewers - discuss safety aspects of sewer maintenance

1. Work site protection
2. Gas testing equipment
3. Nonsparking tools
4. Other

C. Electrical Hazards

1. Discuss grounding of electric tools
2. Outline first aid for electric shock victim
3. Designate authorized personnel to perform electrical repairs
4. Other

D. Mechanical Equipment Hazards

1. Discuss equipment guards
2. Discuss noise level considerations
3. Designate authorized personnel to perform mechanical repairs
4. Other

E. Explosion and Fire Hazards

1. Discuss storage of flammable materials
2. Give type and location of fire extinguishers
3. Discuss use of flammable vapor detectors
4. Other

F. Bacterial Infection (Health Hazards)

1. State policy on tetanus shots
2. Outline personal hygiene considerations
3. State policy on care of cuts and other injuries
4. Other

G. Chlorine Hazards

1. Discuss cylinder handling
2. Outline procedure for testing for and responding to leaks
3. Describe self-contained breathing apparatus use
4. Other

H. Oxygen Deficiency and Noxious Gases

1. Outline noxious gas testing procedures
2. Discuss ventilating equipment
3. Provide tabulation of common gases encountered in wastewater treatment systems
4. Other

I. Safety Equipment - list safety equipment required

1. First aid kits
2. Fire extinguishers
3. Gas masks/air packs
4. Protective clothing and hard hats
5. Safety Harnesses
6. Other

J. Process Chemical Handling - discuss procedures for all chemicals used

K. References - list pertinent safety references

1. WPCF MOP #1 - Safety in Wastewater Works
2. WPCF MOP #7 - Sewer Maintenance
3. Chlorine Institute, Chlorine Manual
4. EPA Manual - Safety in the Design, Operation and Maintenance of Wastewater Treatment Works, Contract No. 68-01-0324
5. Other

Chapter IX. Utilities

A. General

1. Give name of utility company
2. List contact men within utility company
   a) Routine contact
   b) Emergency contact
3. Discuss reliability of service
4. Give any cost information available

B. Electrical

1. Give voltage of service adjacent to facility

2. Give reduced voltage entering facility
3. Discuss stand-by power from a second source

C. Telephone

1. Outline telephone communications system within treatment system
2. Discuss any alarm systems that utilize telephone wires

D. Natural Gas

1. Give cubic feet of gas per hour
2. Give normal operating pressure
3. Give size of gas line

E. Water

1. Give size of waterline
2. Give normal operating pressure
3. Discuss any backflow preventer prevention systems present

F. Fuel Oil

1. List capacities of storage tanks
2. Outline program to insure adequate supplies of fuel oil are always on hand
3. List potential suppliers

Chapter X. Electrical System - describe the Electrical System

Table of Contents

A. General

1. Schematic drawings
2. Tables
3. Manufacturer's literature
4. Shop drawings
5. Designer's notes

B. Power Source

1. Give name of electrical utility company
2. Give characteristics of primary distribution line
3. Describe main transformer and state ownership
4. Discuss protective devices
5. Give maximum available short-circuit current at point(s) of service from utility company

C. Power Distribution System

1. Describe service entrance equipment
2. Describe motor control centers and control panels
3. Provide tabulations indicating power wiring from and loads fed by major electrical components

D. Control and Monitoring System

1. Provide tabulations of type controls present and process equipment involved
2. Provide schematic diagrams

E. Alternate Power Source

1. Describe power source
2. Describe any duplicate equipment in the power distribution system

Appendices

Table of Contents

A. Schematics - provide as required

1. Basic flow diagrams
2. Bypass piping diagrams
3. Hydraulic profile
4. Other

B. Valve Indices - describe all major valves

1. Function
2. Type/Size
3. Location
4. Identification

C. Sample Forms - provide as required

1. Daily Operating Log
2. Equipment Data Cards
3. Maintenance Work Order
4. Purchase Order
5. Accident Report Form
6. Provincial Reports
7. Other

D. Chemicals Used in System

1. List all chemicals
2. Give safety precautions and outline storage considerations in Safety Chapter of Manual
3. List suppliers
4. Provide reorder schedule

E. Emergency Operating and Response Program - provide as required

1. Schematic diagrams
2. Sample forms

F. Detailed Design Criteria - tabulate criteria

1. Population served
2. Wastewater volume
3. Line size and capacities
4. Pump sizes and capacities
5. Pumping characteristics
6. Other

G. Equipment Suppliers

1. Give name
2. List equipment furnished
3. Give reference to where detail information on representatives can be found in manual

H. Manufacturers' Manuals

1. May be bound separately
2. Manuals should give adequate operating and maintenance instructions
3. Manuals should indexed/cross-referenced

I. Sources for Service and Parts

1. List service organizations for all equipment

2. List local repair services:
   a) Meter repair
   b) Motor rewinding
   c) Other

3. List local parts sources:
   a) Plumbing wholesalers
   b) Electrical wholesalers
   c) Mill supply houses
   d) Other

J. As-Built Drawings

1. Ensure drawings are complete and accurate
2. Cross-reference with shop drawings

K. Approved Shop Drawings

1. Index adequately
2. Cross-reference with engineering drawings and construction specifications

L. Dimension Prints

1. Provide when necessary to show units relation to other units, adjacent walls, etc.
2. Use to tie shop drawings to engineering drawings

M. Construction Photos

1. Label and date all photos
2. Outline photo indexing system

N. Warranties and Bonds

1. Provide copies
2. Index properly

O. Copies of Provincial Reporting Forms - provide as required

1. Monthly Operating Report
2. Bypass Report
3. Chlorine Failure Report
4. Other

P. Copies of Provincial Inspection Forms - provide as required

Q. Infiltration Controls

1. Provide copy of existing ordinance
2. Provide model ordinance if none exists

R. Industrial Waste Controls

1. Provide copy of existing ordinance
2. Provide model ordinance if none exists

S. Piping Color Codes

1. List color for each piping system
2. State if directional flow arrows and/or labelling required

T. Painting

1. Give type of coating required for each unit
2. Give painting frequency schedule
3. Provide a copy of Water Pollution Control Federation, MOP-17, "Paints and Protective Coatings", (1969)

D.3 PREPARATION OF O & M MANUALS

D.3.1 Persons Responsible for Manual Development

Individuals responsible for O & M Manual development should obtain input from persons experienced in treatment system operations. This input, combined with the design engineer's expertise, is essential to any good manual. If possible, operations input should be obtained from persons with experience in the same processes as those described in the manual.

D.3.2 Equipment Information

Persons involved in the preparation of the O & M Manual should take necessary action to insure they obtain timely and accurate operations and maintenance information on all equipment items. These actions might simply be enforcement of existing requirements or adding sections to project specifications calling for submittal of preliminary O & M information prior to paying for equipment.

O & M Manual preparation requires timely and accurate information from suppliers of wastewater treatment equipment for incorporation in O & M Manuals. The information should be tailored for the specific equipment item supplied.

D.3.3 Manual Flexibility

O & M Manuals should possess the necessary flexibility to remain viable tools to operating personnel, in the event of changing treatment system operating and maintenance needs.

D.3.4 Writing Style

The key to an O & M Manual's ultimate success is the language used and the writing style. Persons preparing an O & M Manual must ensure that they obtain information from people actually experienced in plant operations and maintenance and translate the design engineer's concepts into a language form acceptable to operating personnel. The Manual must also consider the comprehension level of the end users.

PROVINCE OF NOVA SCOTIA SEWAGE TREATMENT PLANT EFFLUENT DISCHARGE POLICY

E.1 LEGISLATIVE AUTHORITY

Environmental Protection Act, Water Act, Health Act.

E.2 STATEMENT OF PRINCIPLES

This policy describes the level of treatment which will be required at new or upgraded municipal and private sewage treatment plants discharging into surface waters.

E.3 APPLICATION

This policy applies to any facility treating sanitary sewage regardless of the source.

E.3.1 Department's Requirement of Municipal and Private Sewage Treatment Systems

The Departments of the Environment and Health require that all municipalities and private sewage collection systems, treatment plants and outfalls be designed, constructed, located, operated and maintained so as to minimize pollution of the receiving waters and interference with other water uses.

E.3.2 Type of Treatment

The Departments recommend the following types of trreatment based on the size of the facility.

Table E.1

SIZE GALLONS/DAY	TYPE OF TREATMENT
0-2000	On-site in ground systems
2000-50,000	By order of preference 1. In ground systems 2. Seasonal discharge (i.e., lagoon) 3. STP* with land disposal (e.g., spray on forest) (*secondary 30/30 with no chlorination) 4. Small STP based on Table 1. Criteria other than that specified in Table 1 may be accepted when based on a receiving water study
>50,000	Treatment based on study

E.3.3 Standard Level of Treatment

The normal level of treatment shall be determined on the basis of the type of receiving water as documented in Table I.

E.3.4 Receiving Water Assessments

The Departments recommend that a receiving water assessment be undertaken for all sewage treatment systems. A receiving water assessment is mandatory where the proposed sewage treatment system is 50,000 US gal/day or greater or where in the opinion of the Regional Manager or the Manager of Municipal Wastes and Resources Recovery it is required on the basis of the sensitivity of the receiving water.

A receiving water study shall follow the format prescribed in “Nova Scotia Standards and Guidelines Manual for the Collection, Treatment, and Disposal of Sanitary Sewage”.

E.3.5 Deviations from the Standard Level of Treatment

A relaxation of the standard level of treatment will only be allowed on a case-by-case basis and in accordance with the level of treatment indicated as necessary in the receiving water assessment.

Under no circumstances will the level of treatment required by less than primary treatment for facilities discharging into a coastal environment and not less than secondary treatment when discharging to fresh water or inland water systems.

E.3.5.1 Higher than Standard Treatment

Higher than standard levels of treatment up to and including no discharge to surface waters may be imposed based on a site specific receiving water assessment.

E.3.6 Review of Treatment Plants

The level of treatment required for individual sewage treatment systems shall be subject to periodic review as necessary (especially when expansions of sewage treatment systems are contemplated).

E.3.7 Effluent Monitoring and Compliance

A monitoring program, including regular sampling of sewage treatment system effluent and recording of flows will be undertaken by the systems operating authority and/or owner.

This monitoring program should be carried out in compliance with the “Policy with Respect to Sampling and Analysis Requirements for Municipal and Private Sewage Treatment Works”.

E.3.8 Operator Certification

To ensure the facilities are properly maintained and operated, all facilities shall be under the direct supervision of a certified operator.

E.3.9 Effluent Disinfection

Effluent disinfection requirements will be established by the Nova Scotia Department of Health.

E.3.10 Deviation from Policies and Guidelines

Any deviation or relaxation from the policies listed above must receive the approval of the Director of the Resource Management and Pollution Control Division and the Director of Public Health Engineering.

TABLE E.2 EFFLUENT REQUIREMENTS

POINT OF DISCHARGE	FECAL COLIFORM	REQUIRED EFFLUENT QUALITY (A) (B) BOD_5/SS
Fresh water lakes low flow streams (5-10 times dilution)*	200/100 mls	5/5
Rivers and estuaries	1000/100 mls	20/20
Open coastline	5000/100 mls	30/30

A) Effluent quality may be stipulated as a result of a receiving water assessment. Under no circumstances will less than secondary treatment be acceptable for a fresh water discharge or primary for a coastal discharge. (The receiving water study may indicate that no discharge is permissible).

B) Nutrient removal may also be specifically required.

* <5 times dilution may require periods of no discharge.

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3. Antoine, R.L., "Fixed Biological Surfaces - Wastewater Treatment - The Rotating Biological Contactor", 1976.

4. Atlantic Canada Voluntary Certification Program.

5. British Columbia Ministry of the Environment, "Pollution Control Objectives for Municipal Type Waste Discharges in British Columbia", 1989.

6. Canadian Institute on Pollution Control, "Manual on Wastewater Sampling Practice", 1972.

7. Crites, R.W., "Design Criteria and Practice for Constructed Wetlands", Water Science and Technology, Vol. 29, No. 4, 1994.

8. Eckenfelder, W. Wesley, "Water Quality Engineering for Practicing Engineers", 1970.

9. Environment Canada, "Manual for Land Application of Treated Municipal Wastewater and Sludge", EPS 6-EP-84-1, March 1984.

10. Feurstein, Donald L., "Predesign Surveys and Monitoring of Waste Disposal Systems".

11. Ganczarczyk, Jerry J., "Activated Sludge Process - Theory and Practice", 1983.

12. Great Lakes - Upper Mississippi River Board of State Sanitary Engineers, "Recommended Standards for Sewage Works", 1978.

13. Great Lakes - Upper Mississippi River Board of State Public Health and Environmental Managers, "Recommended Standards for Wastewater Facilities", 1990.

14. Metcalf & Eddy Inc., "Wastewater Engineering : Treatment, Disposal, Reuse", 1991.

15. New Brunswick Department of Municipal Affairs and Environment, "Guidelines for the Collection and Treatment of Wastewater", September 1987.

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17. Nova Scotia Department of the Environment, "Manual of Wastewater - Interim Guidelines - Collection and Treatment of Municipal Wastewater", 1976.

18. Nova Scotia Department of the Environment, "Interim Report - Guidelines for the Handling and Disposal of Sewage Sludge"

19. Ontario Ministry of the Environment (Ontario MOE), "Water Management - Goals, Policies, Objectives and Implementation Procedures of the Ministry of Environment", May 1984.

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22. Ontario MOE, "Stream Water Quality Assessment Procedures Manual", March 1990.

23. Saskatchewan Environment and Public Safety, "Surface Water Quality Objectives", November 1988.

24. Saskatchewan Environment and Public Safety, "A Guide to Sewage Works Design", January 1989.

25. United States Environmental Protection Agency (USEPA), "Handbook for Stream Sampling for Waste Load Allocation Application", EPA/G25/6-86/013, 1988.

26. USEPA, "Considerations for Preparation of Operations and Maintenance Manuals", 1974.

27. USEPA, "Design Information on Rotating Biological Contactors", June 1984.

28. USEPA, "Aerobic Biological Wastewater Treatment Facilities", March 1977.

29. USEPA, "Process Design Manual for Sludge Treatment and Disposal".

30. USEPA, "Technical Guidance Manual for Providing Waste Load Allocations - Book IV Lakes and Impoundments - Chapter 1, 2, and 3".

31. USEPA, "Process Design Manual - Wastewater Treatment Facilities for Sewered Small Communities".

32. USEPA, "Technical Guidance Manual for Providing Waste Load Allocations - Book III Estuaries, Part 1 and 2", March 1990.

33. USEPA, "Technical Guidance Manual for Providing Waste Load Allocations - Book VI Design Conditions - Chapter 1 - Stream Flow Design for Steady State Modelling", September 1986.

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35. USEPA, "Technical Guidance Manual for Providing Waste Load Allocations - Simplified Analytical Method for Determining NPDES Effluent Limitations for POTW's Discharging into Low-Flow Streams", 1980.

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