Biological Wastewater Treatment in Warm Climate Regions

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VOLUME ONE

IWA
Publishing
LONDON • SEATTLE

DESA
UFMG

Published by IWA Publishing, Alliance House, 12 Caxton Street, London SW1H 0QS, UK

Telephone: +44 (0) 20 7654 5500; Fax: +44 (0) 20 7654 5555; Email: publications@iwap.co.uk
Website: **www.iwapublishing.com**

First published 2005
Reprinted 2006
© 2005 IWA Publishing

Copy-edited and typeset by TechBooks, India
Printed by TJ International, Padstow, UK

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British Library Cataloguing in Publication Data
A CIP catalogue record for this book is available from the British Library

Library of Congress Cataloguing-in-Publication Data
A catalogue record for this book is available from the Library of Congress

ISBN: 1 84339 002 7 (this volume and set)

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Preface

The implementation of wastewater treatment plants has been so far a challenge for most countries. Economical resources, political will, institutional strength and cultural background are important elements defining the trajectory of pollution control in many countries. Technological aspects are sometimes mentioned as being one of the reasons hindering further developments. However, as shown in this book, the vast array of available processes for the treatment of wastewater should be seen as an incentive, allowing the selection of the most appropriate solution in technical and economical terms for each community or catchment area. For almost all combinations of requirements in terms of effluent quality, land availability, construction and running costs, mechanisation level and operational simplicity there will be one or more suitable treatment processes.

Biological wastewater treatment is very much influenced by climate. Temperature plays a decisive role in some treatment processes, especially the natural-based and non-mechanised ones. *Warm temperatures* decrease land requirements, enhance conversion processes, increase removal efficiencies and make the utilisation of some treatment processes feasible. Some treatment processes, such as anaerobic reactors, may be utilised for diluted wastewater, such as domestic sewage, only in warm climate areas. Other processes, such as stabilisation ponds, may be applied in lower temperature regions, but occupying much larger areas and being subjected to a decrease in performance during winter. Other processes, such as activated sludge and aerobic biofilm reactors, are less dependent on temperature, as a result of the higher technological input and mechanisation level. The main purpose of the book is to present the technologies for urban wastewater treatment as applied to the specific condition of warm temperature, with the related implications in terms of design and operation. There is no strict definition for the range of temperatures that fall into this category, since the book always presents how to correct parameters, rates and coefficients for different temperatures. In this sense, subtropical and even temperate climate are also indirectly covered, although most of the focus lies on the tropical climate.

Another important point is that most warm climate regions are situated in *developing countries*. Therefore, the book casts a special view on the reality of these countries, in which simple, economical and sustainable solutions are strongly demanded. All technologies presented in the book may be applied in developing countries, but of course they imply different requirements in terms of energy, equipment and operational skills. Whenever possible, simple solutions, approaches and technologies are presented and recommended.

Considering the difficulty in covering all different alternatives for wastewater collection, the book concentrates on *off-site solutions*, implying collection and transportation of the wastewater to treatment plants. No on-site solutions, such as latrines and septic tanks, are analysed. Also, stronger focus is given to *separate sewerage systems*, although the basic concepts are still applicable to combined systems, especially under dry weather conditions. Furthermore, emphasis is given to *urban wastewater*, that is, mainly domestic sewage plus some additional small contribution from non-domestic sources, such as industries. Hence, the book is not directed specifically to industrial wastewater treatment, given the specificities of this type of effluent. Another specific view of the book is that it details *biological treatment processes*. No physical–chemical wastewater treatment processes are covered, although some physical operations, such as sedimentation and aeration, are dealt with since they are an integral part of some biological treatment processes.

The book's proposal is to present in a balanced way *theory and practice* of wastewater treatment, so that a conscious selection, design and operation of the wastewater treatment process may be practised. Theory is considered essential for the understanding of the working principles of wastewater treatment. Practice is associated with the direct application of the concepts for conception, design and operation. In order to ensure the practical and didactic view of the book, *371 illustrations*, *322 summary tables* and *117 examples* are included. All major wastewater treatment processes are covered by full and interlinked design examples which are built up throughout the book, from the determination of the wastewater characteristics, the impact of the discharge into rivers and lakes, the design of several wastewater treatment processes and the design of the sludge treatment and disposal units.

The 55 chapters are divided into 7 parts, namely: *(1) Introduction to wastewater characteristics, treatment and disposal; (2) Basic principles of wastewater treatment; (3) Stabilisation ponds; (4) Anaerobic reactors; (5) Activated sludge; (6) Aerobic biofilm reactors; and (7) Sludge treatment and disposal.*

Part 1 (*Introduction to wastewater characteristics, treatment and disposal*) presents an integrated view of water quality and wastewater treatment, analysing wastewater characteristics (flow and major constituents), the impact of the discharge into receiving water bodies and a general overview of wastewater treatment and sludge treatment and disposal. Part 1 is more introductory, and may be used as teaching material for undergraduate courses in civil engineering, environmental engineering, environmental sciences and related courses.

Part 2 (*Basic principles of wastewater treatment*) is also introductory, but at a higher level of detailing. The core of this part is the unit operations and processes associated with biological wastewater treatment. The major topics covered are: microbiology and ecology of wastewater treatment; reaction kinetics and reactor hydraulics; conversion of organic and inorganic matter; sedimentation; aeration. Part 2 may be used as part of postgraduate courses in civil engineering, environmental engineering, environmental sciences and related courses, either as part of disciplines on wastewater treatment or unit operations and processes.

Parts 3 to 6 are the central part of the book, being structured according to the major wastewater treatment processes (*stabilisation ponds, anaerobic reactors, activated sludge and aerobic biofilm reactors*). In each part, all major process technologies and variants are fully covered, including main concepts, working principles, expected removal efficiencies, design criteria, design examples, construction aspects and operational guidelines. Similarly to Part 2, Parts 3 to 6 can be used in postgraduate courses in civil engineering, environmental engineering, environmental sciences and related courses.

Part 7 (*Sludge treatment and disposal*) covers in detail sludge characteristics, production, treatment (thickening, dewatering, stabilisation, pathogen removal) and disposal (land application for agricultural purposes, sanitary landfills, landfarming and other methods). Environmental and public health issues are fully described. Possible academic uses for this part are same as those from Parts 3 to 6.

Besides being used as a *textbook* at academic institutions, it is believed that the book may be an important *reference for practicing professionals*, such as engineers, biologists, chemists and environmental scientists, acting in consulting companies, water authorities and environmental agencies.

The present book is based on a consolidated, integrated and updated version of a series of six books written by the authors in Brazil, covering the topics presented in the current book, with the same concern for didactic approach and balance between theory and practice. The huge success of the Brazilian books, used at most graduate and post-graduate courses at Brazilian universities, besides consulting companies and water and environmental agencies, was the driving force for the preparation of this international version.

In this version, the book aims at presenting consolidated technology based on worldwide experience available from international literature. However, it should be recognised that a significant input comes from the Brazilian experience, considering the background and working practice of all authors. Brazil is a large country with many geographical, climatic, economical, social and cultural contrasts, reflecting well the reality encountered in many countries in the world. Besides, it should be mentioned that Brazil is currently one of the leading countries in the world as regards the application of anaerobic technology to domestic sewage treatment, and the post-treatment of anaerobic effluents. Regarding this point, the authors would like to show their recognition for the Brazilian Research Programme on Basic Sanitation (PROSAB), which, through several years of intensive, applied, cooperative research has led to the consolidation of anaerobic treatment and aerobic/anaerobic post-treatment, which are currently widely applied in full-scale plants in Brazil. Consolidated results achieved by PROSAB are included in various parts of the book, representing invaluable and updated information applicable to warm climate regions.

Parts 1 to 5 were written by the two main authors. The book counted with the invaluable participation of Cleverson Vitorio Andreoli and Fernando Fernandes, who acted as editors for Part 7, and of several specialists, who acted as authors in the chapters in Parts 6 and 7. The authors of these chapters are: Aderlene Inês de Lara, Deize Dias Lopes, Dione Mari Morita, Eduardo Sabino Pegorini, Hilton Felício dos Santos, Marcelo Antonio Teixeira Pinto, Maurício Luduvice, Ricardo Franci Gonçalves, Sandra Márcia Cesário Pereira da Silva, Vanete Thomaz Soccol. Technical review of the English version of Part 7 was made by Hilton Felício dos Santos. Financial support for the translation of Part 7 was provided by SANEPAR (Paraná Water and Sanitation Company, Brazil).

Many colleagues, students and professionals contributed with useful suggestions, reviews and incentives for the Brazilian books that were the seed for this international version. It would be impossible to list all of them here, but our heartfelt appreciation is acknowledged.

The authors would like to express their recognition for the support provided by the Department of Sanitary and Environmental Engineering at the Federal University of Minas Gerais, Brazil, at which the two authors work. The department provided institutional and financial support for this international version, which is in line with the university's view of expanding and disseminating knowledge to society.

Finally, the authors would like to show their appreciation to IWA Publishing, for their incentive and patience in following the development of this book throughout the two years of hard work.

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Carlos Augusto de Lemos Chernicharo

To

Paulo and Margarida von Sperling,
Vanessa and Bruno Guerra de Moura von Sperling

and

Jair and Dinorah Chernicharo
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PART ONE

Introduction to wastewater characteristics, treatment and disposal

1

Introduction to water quality and water pollution

1.1 INTRODUCTION

Water, because of its properties as a solvent and its capacity to transport particles, incorporates in itself various impurities that characterise the *water quality*.

Water quality is a result of natural phenomena and the acts of human beings. Generally it can be said that *water quality is a function of land use in the catchment area*. This is due to the following factors:

- **Natural conditions**: even with the catchment area preserved in its natural condition, the surface water quality is affected by run off and infiltration resulting from rainfall. The impact of these is dependent on the contact of the water with particles, substances and impurities in the soil. Therefore, the incorporation of suspended solids (e.g. soil particles) or dissolved solids (e.g. ions originating from the dissolution of rocks) occurs even when the catchment area is totally preserved in its natural condition (e.g. occupation of the land with woods and forests). In this case, the soil protection and composition have a great influence.
- **Interference of human beings**: the interference of man manifests itself either in a concentrated form, such as in the discharge of domestic or industrial wastewater, or in a diffused form, such as in the application of fertilisers or pesticides onto the soil. Both contribute to the introduction of

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compounds into the water, thus affecting its quality. Therefore, the form in which human beings use and occupy the land has a direct implication in the water quality.

Figure 1.1 presents an example of possible interactions between land use and the presence of factors that modify the water quality in rivers and lakes. Water quality control is associated with a global planning at the whole catchment area level, and not individually, for each impacting source.

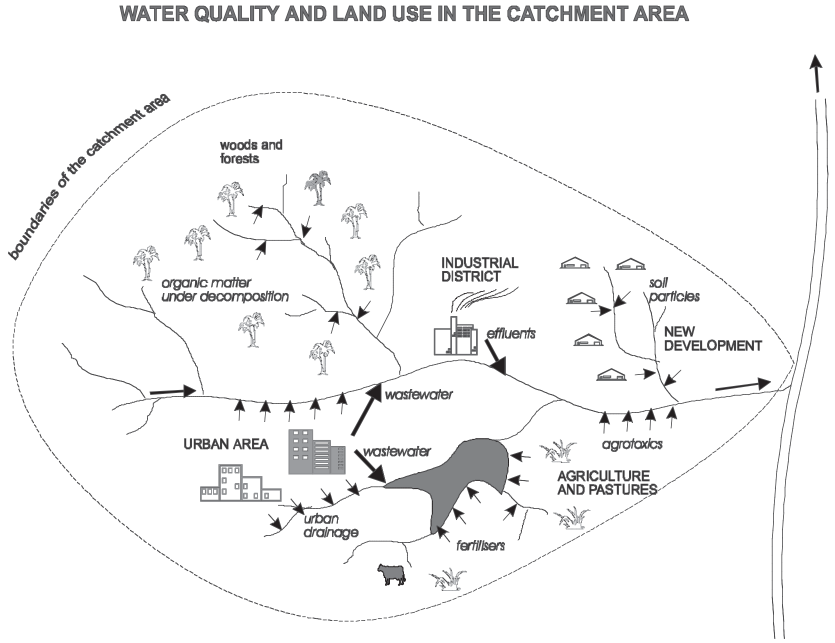


Figure 1.1. Examples in a catchment area of the interrelation between land use and water quality impacting agents

Separate from the above concept of *existing* water quality, there is the concept of the *desired* water quality. *The desired quality for a water is a function of its intended use*. There are various possible intended uses for a particular water, which are listed in Section 1.2. In summary:

- Existing water quality: function of the land use in the catchment area
- Desired water quality: function of the intended uses for the water

Within the focus of this book, the study of water quality is essential, not only to characterise the consequences of a certain polluting activity, but also to allow the selection of processes and methods that will allow compliance with the desired water uses.

1.2 USES OF WATER

The main water uses are:

- *domestic supply*
- *industrial supply*
- *irrigation*
- *animal supply*
- *preservation of aquatic life*
- *recreation and leisure*
- *breeding of aquatic species*
- *generation of electricity*
- *navigation*
- *landscape harmony*
- *dilution and transport of wastes*

In general terms, only the first two uses (domestic supply and industrial supply) are frequently associated with a prior water treatment, in view of their more demanding quality requirements.

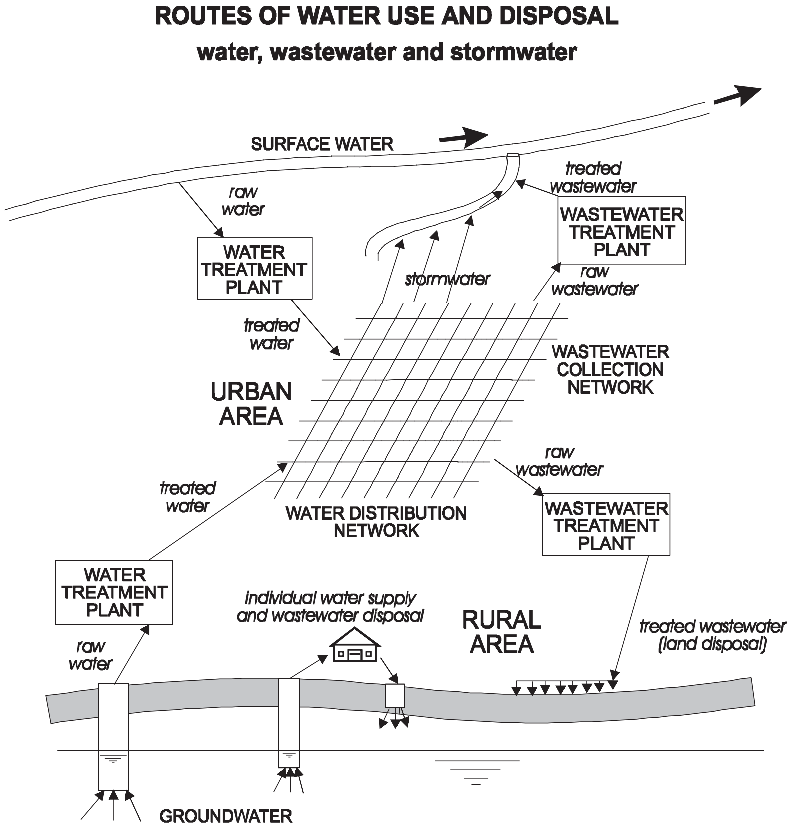
There is a direct relation between water use and its required quality. In the above list, the most demanding use can be considered domestic water supply, which requires the satisfaction of various quality criteria. Conversely, the less demanding uses are simple dilution and transportation of wastes, which do not have any specific requirements in terms of quality. However, it must be remembered that multiple uses are usually assigned to water bodies, resulting in the necessity of satisfying diverse quality criteria. Such is the case, for example, of reservoirs constructed for water supply, electricity generation, recreation, irrigation and others.

Besides the cycle of water on Earth (hydrological cycle), there are internal cycles, in which water remains in the liquid state, but has its characteristics modified as a result of its use. Figure 1.2 shows an example of typical routes of water use, composing partial cycles. In these cycles, the water quality is modified at each stage of its journey.

The management of these internal cycles is an essential role in environmental engineering, and includes the planning, design, construction and control of the works necessary for the maintenance of the desired water quality as a function of its intended uses. Therefore, the engineer or scientist must know how to ask for and interpret the results of water quality samples in the various points of the cycle. This book focuses mainly on the aspect of wastewater treatment, and the impact of the discharge of wastewater to receiving bodies is covered in Chapter 3.

1.3 WATER QUALITY REQUIREMENTS

Table 1.1 presents in a simplified way the association between the main quality requirements and the corresponding water uses. In cases of water bodies with multiple uses, the water quality must comply with the requirements of the various intended uses. The expression "free" in the table is different from "absolutely free". Zero levels of many contaminants cannot be guaranteed and in most cases are not necessary. The acceptable concentrations are based on risk analysis, a tool that is used for deriving quality guidelines and standards.



- *Raw water*. Initially, water is abstracted from the river, lake or water table, and has a certain quality.
- *Treated water*. After abstraction, water undergoes transformations during its treatment to be able to comply with its intended uses (e.g. public or industrial water supply).
- *Raw wastewater*. The water, after being used, undergoes new transformations in its quality and becomes a liquid waste.
- *Treated wastewater*. Aiming at removing its main pollutants, wastewater undergoes treatment before being discharged into the receiving body. Wastewater treatment is responsible for the new modification in the quality of the liquid.
- *Stormwater*. Rain water flows on the ground, incorporates some pollutants, and is collected at stormwater systems before being discharged into the receiving body.
- *Receiving body*. Stormwater and the effluent from the wastewater treatment plant reach the receiving body where water quality undergoes new modifications, as a result of dilution and self-purification mechanisms.

Figure 1.2. Routes of water use and disposal

Table 1.1. Association between water use and quality requirements

General use	Specific use	Required quality
Domestic supply	–	– Free from chemical substances harmful to health – Free from organisms harmful to health – Low aggressiveness and hardness – Aesthetically pleasant (low turbidity, colour, taste and odour; absence of macro-organisms)
Industrial supply	Water incorporated into the product (e.g. food, drinks, medicines)	– Free from chemical substances harmful to health – Free from organisms harmful to health – Aesthetically pleasant (low turbidity, colour, taste and odour; absence of macro-organisms)
	Water that enters into contact with the product	– Variable with the product
	Water that does not enter into contact with the product (e.g. refrigeration units, boilers)	– Low hardness – Low aggressiveness
Irrigation	Horticulture, products ingested raw or with skin	– Free from chemical substances harmful to health – Free from organisms harmful to health – Non-excessive salinity
	Other plantations	– Free from chemical substances harmful to the soil and plantations – Non-excessive salinity
Animal water supply	–	– Free from chemical substances harmful to animals health – Free from organisms harmful to animals health
Preservation of aquatic life	–	– Variable with the environmental requirements of the aquatic species to be preserved
Aquaculture	Animal breeding	– Free from chemical substances harmful to animals, workers and consumers health – Free from organisms harmful to animals, workers and consumers health – Availability of nutrients
	Vegetable growing	– Free from chemical substances toxic to vegetables and consumers – Availability of nutrients

(*continued*)

Table 1.1. (*continued*)

General use	Specific use	Required quality
Recreation and leisure	Primary contact (direct contact with the liquid medium – bathing; e.g.: swimming, water-skiing, surfing)	– Free from chemical substances harmful to health – Free from organisms harmful to health – Low levels of suspended solids and oils and grease
	Secondary contact (without direct contact with the liquid medium; e.g.: leisure navigation, fishing, contemplative viewing)	– Pleasant appearance
Energy generation	Hydroelectric power plants	– Low aggressiveness
	Nuclear or thermoelectric power plants (e.g. cooling towers)	– Low hardness
Transport	–	– Low presence of course material that could be dangerous to vessels
Waste dilution and transportation	–	–

1.4 WATER POLLUTION

Water pollution is the addition of substances or energy forms that directly or indirectly **alter** the nature of the water body in such a manner that **negatively affects** its legitimate **uses**.

This definition is essentially practical and, as a consequence, potentially controversial, because of the fact that it associates pollution with negative alterations and with water body uses, concepts that are attributed by human beings. However, this practical view is important, principally when analysing the control measures for pollution reduction

Table 1.2 lists the main pollutants and their source, together with the most representative effects. Chapter 2 covers in detail the main parameters, which characterise the quality of a wastewater (second column in the table). For domestic sewage, which is the main focus of this book, the main pollutants are: *suspended solids*, *biodegradable organic matter*, *nutrients* and *pathogenic organisms*. Their impact in the water body is analysed in detail in Chapter 3.

The solution to most of these problems, especially biodegradable organic matter and pathogens, has been reached in many **developed regions**, which are now concentrated on the removal of nutrients and micro-pollutants, together with substantial attention to the pollution caused by storm-water drainage. In **developing**

Table 1.2. Main pollutants, their source and effects

Pollutant	Main representative parameters	Source				Possible effect of the pollutant
		Wastewater		Stormwater		
		Domestic	Industrial	Urban	Agricultural and pasture	
Suspended solids	Total suspended solids	XXX	←→	XX	X	• Aesthetic problems • Sludge deposits • Pollutants adsorption • Protection of pathogens
Biodegradable organic matter	Biochemical oxygen demand	XXX	←→	XX	X	• Oxygen consumption • Death of fish • Septic conditions
Nutrients	Nitrogen Phosphorus	XXX	←→	XX	X	• Excessive algae growth • Toxicity to fish (ammonia) • Illnesses in new-born infants (nitrate) • Pollution of groundwater
Pathogens	Coliforms	XXX	←→	XX	X	• Water-borne diseases
Non-biodegradable organic matter	Pesticides Some detergents Others	X	←→	X	XX	• Toxicity (various) • Foam (detergents) • Reduction of oxygen transfer (detergents) • Non-biodegradability • Bad odours (e.g.: phenols)
Metals	Specific elements (As, Cd, Cr, Cu, Hg, Ni, Pb, Zn, etc.)	X	←→	X		• Toxicity • Inhibition of biological sewage treatment • Problems in agriculture use of sludge • Contamination of groundwater
Inorganic dissolved solids	Total dissolved solids Conductivity	XX	←→		X	• Excessive salinity – harm to plantations (irrigation) • Toxicity to plants (some ions) • Problems with soil permeability (sodium)

x: *small* xx: *medium* xxx: *high* ←→: variable *empty*: usually not important

regions, the basic pollution problems still need to be dealt with, and the whole array of pollutants needs to be tackled. However, because of scarcity of financial resources in these regions, priorities need to be set (as they have been, in the past, and continue to be, in the developed regions), and the gross pollution by organic matter and contamination by pathogens are likely to deserve higher attention. Naturally, each region has its own specificities, and these need to be taken into account when setting up priorities.

Also from the table, it is seen that it is very difficult to generalise industrial wastewater, because of its variability from process to process and from industry to industry.

In the table, it is also seen that there are two ways in which the pollutant could reach the receiving body (see Figure 1.3):

- *point-source pollution*
- *diffuse pollution*

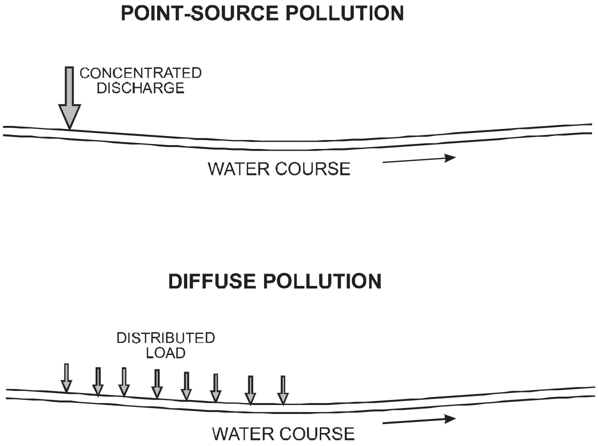


Figure 1.3. Point-source and diffuse pollution

In *point-source* pollution, the pollutants reach the water body in points concentrated in the space. Usually the discharge of domestic and industrial wastewater generates point-source pollution, since the discharges are through outfalls.

In *diffuse pollution,* the pollutants enter the water body distributed at various locations along its length. This is the typical case of storm water drainage, either in rural areas (no pipelines) or in urban areas (storm water collection system, with multiple discharges into the water body).

The focus of this book is the control of point-source pollution by means of wastewater treatment. In the developing regions, there is practically everything still to be done in terms of the control of point-source pollution originating from cities and industries.

2

Wastewater characteristics

2.1 WASTEWATER FLOWRATES

2.1.1 Introduction

Wastewater sewerage (collection, treatment and disposal) is accomplished by the following main alternatives (Figure 2.1):

- *Off-site sewerage*
    - Separate sewerage system
    - Combined sewerage system
- *On-site sewerage*

In various countries a **separate sewerage system** is adopted, which separates storm water from sewage, both being transported by independent pipeline systems. In this case, in principle, storm water does not contribute to the wastewater treatment plant (WWTP). In other countries, however, a **combined (unitary) sewerage system** is adopted, which directs sewage and storm water together into the same system (see Figure 2.1). In this case, the pipelines have a larger diameter, to transport not only the sewage flow, but mainly rainwater, and the design of the WWTP has to take into consideration the corresponding fraction of rainwater that is allowed to enter the treatment works. In countries with a warm climate, during the dry season, sewage flows slowly in these large diameter pipes, leading to long detention times which allow decomposition and generation of malodours. In this book, *focus is given to the separate sewerage system*, analysing only the three components listed above. However, the principles for the design of a combined sewerage system, based on dry-weather flow, are the same.

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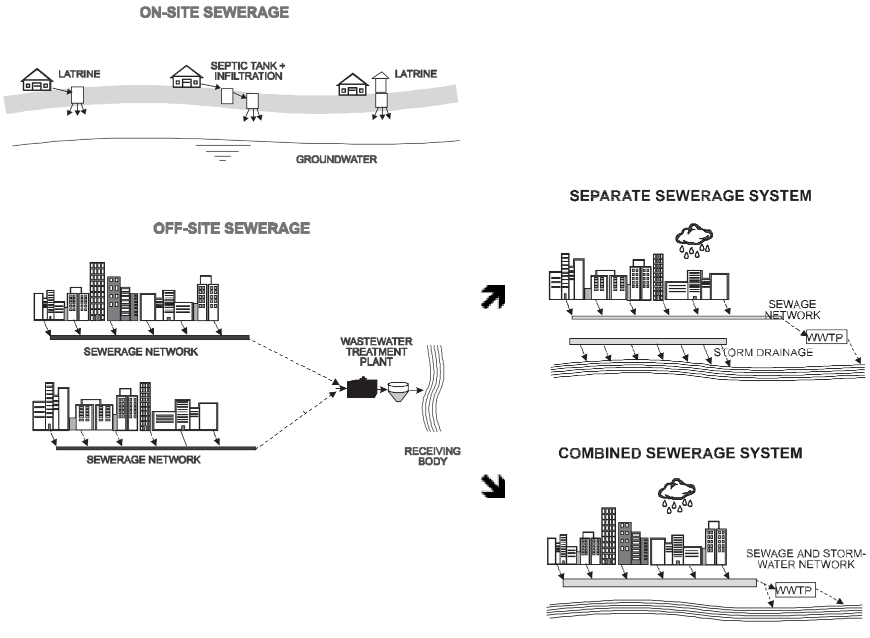


(a) Sewerage systems: on-site and off-site

(b) Off-site sewerage systems: separate and combined

Figure 2.1. Types of sewerage system

Similarly, the book concentrates on **off-site collection systems** (with a water-borne sewerage collection and transportation network) and does not cover the **on-site systems** (e.g. latrines and septic tanks). These are of great importance and in many cases the best alternative in various regions, being more applicable in locations with a low population density, like rural areas (even though they are also applied in various densely occupied locations, but frequently presenting problems of infiltration in the soil and the resulting contamination of the water table).

Urban wastewater that flows in an off-site sewerage system and contributes to a WWTP is originated from the following three main sources:

- *Domestic sewage* (including residences, institutions and commerce)
- *Infiltration*
- *Industrial effluents* (various origins and types of industry)

For the characterisation of both quantity and quality of the influent to the WWTP, it is necessary to separately analyse each of the three items.

2.1.2 Domestic wastewater flow

2.1.2.1 Preliminaries

The concept of domestic flow encompasses the sewage originating from homes, as well as commercial activities and institutions that are normally components of the locality. More expressive values originating from significant point sources must be computed separately and added to the global value.

Normally domestic sewage flow is calculated based on the water consumption in the respective locality. The water consumption is usually calculated as a function of the design population and of a value attributed for the average daily per capita water consumption.

It is important to observe that for the design and operation of the sewage treatment works it is not sufficient to consider only the average flow. It is also necessary to quantify the minimum and maximum flowrates, because of hydraulic and process reasons.

This Section describes the population-forecast studies, the estimates of water consumption and the production of wastewater, together with the variations in flow (minimum and maximum flow).

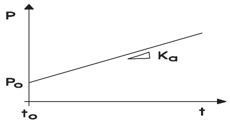
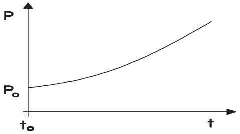
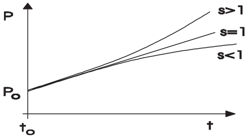
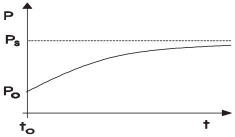
2.1.2.2 Population forecast

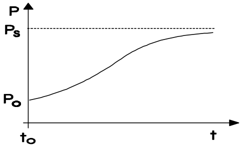
The population that contributes to the treatment plant is that situated inside the design area served by the sewerage system. However, the design population is only a certain fraction of the total population in this area, because maybe not all the population is connected to the sewerage system. This ratio (population served/total population) is called the **coverage index**. This index can be determined (current conditions) or estimated (future conditions), such as to allow the calculation of the design flow. In the final years of the planning horizon, it is expected that the coverage will be close to 100%, reflecting the improvement and expansion in the collection network. The coverage index is a function of the following aspects:

- *Physical, geographical or topographical conditions of the locality*. It is not always possible to serve all households with the sewerage system. Those not served must adopt other solutions besides the off-site water-borne sewerage system.
- *Adhesion index*. This is the ratio between the population actually connected to the system and the population potentially served by the sewerage system in the streets (not all households are connected to the available system, that is to say, not all adhere to the sewerage system). In some communities, it is compulsory to connect to the collection system, in case it passes in front of the house; in other communities, this is optional.
- *Implementation stages of the sewerage system*. In the initial operating years of the WWTP, maybe not all of the designed collection and transport system has been actually installed, and this affects the initial flow.

For the design of a sewage treatment works it is necessary to know the final population (population at the end of the planning horizon – see Chapter 6 for the concept of planning horizon) as well as the initial population and its evolution with time, in order to allow the definition of implementation stages.

Table 2.1. Population forecast. Methods based on mathematical formulas

Method	Description	Curve shape	Growth rate	Forecast formula	Coefficients (if regression analysis is not used)
Linear growth	Population growth follows a constant rate. Method used for short-term forecasts. Curve fitting can also be done through regression analysis.		$\frac{dP}{dt} = K_a$	$P_t = P_o + K_a.(t - t_o)$	$K_a = \frac{P_2 - P_0}{t_2 - t_0}$
Geometric growth	Population growth is a function of the existing population at every instant. Used for short-term forecasts. Curve fitting can also be done through regression analysis.		$\frac{dP}{dt} = K_g.P$	$P_t = P_o.e^{K_g.(t-t_0)}$ or $P_t = P_o.(1 + i)^{(t-t_0)}$	$K_g = \frac{\ln P_2 - \ln P_0}{t_2 - t_0}$ or $i = e^{K_g} - 1$
Multiplicative regression	Fitting of population growth by linear regression (logarithmic transformation of the equation) or non-linear regression.		–	$P_t = P_o + r.(t - t_o)^s$	r, s – regression analysis
Decreasing growth rate	Assumption that, as the town grows, the growth rate becomes lower. The population tends asymptotically to the saturation value. The coefficients can be also estimated by non-linear regression.		$\frac{dP}{dt} = K_d.(P_s - P)$	$P_t = P_0 + (P_s - P_o) \times \left[1 - e^{-K_d.(t-t_0)}\right]$	$P_s = \frac{2.P_o.P_1.P_2 - P_1^2.(P_o + P_2)}{P_o.P_2 - P_1^2}$ $K_d = \frac{-\ln[(P_s - P_2)/(P_s - P_o)]}{t_2 - t_o}$

Logistic growth	The population growth follows an S-shaped curve. The population tends asymptotically to a saturation value. The coefficients can be also estimated by non-linear regression. Required conditions: $P_o < P_1 < P_2$ and $P_0.P_2 < P_1^2$. The point of inflexion in the curve occurs at time $t = [t_o - \ln(c)/K_1]$ and with $P_t = P_s/2$.		$\frac{dP}{dt} = K_1.P.\left(\frac{(P_s - P)}{P_s}\right)$	$P_t = \frac{P_s}{1 + c.e^{K_1.(t-t_o)}}$	$P_s = \frac{2.P_o.P_1.P_2 - P_1^2.(P_o + P_2)}{P_o.P_2 - P_1^2}$ $c = (P_s - P_o)/P_o$ $K_1 = \frac{1}{t_2 - t_1}.\ln\left[\frac{P_o.(P_s - P_1)}{P_1.(P_s - P_o)}\right]$

Source: partly adapted from Qasim (1985)

- dP/dt = population growth rate as a function of time
- P_o, P_1, P_2 = population in the years t_o, t_1, t_2. The formulas for the decreasing and logistic growth rates require equally-spaced values in time if regression analysis is not employed (inhabitants)
- P_t = population estimated for year t (inhabitants); P_s = saturation population (inhabitants)
- K_a, K_g, K_d, K_l, i, c, r, s = coefficients (obtaining coefficients by regression analysis is preferable as all of the existing data series can be used, and not only P_o, P_1 e P_2)

The main methods or models used for population forecasts are (Fair et al, 1973; CETESB, 1978; Barnes et al, 1981; Qasim, 1985; Metcalf & Eddy, 1991):

- *linear (arithmetic) growth*
- *geometric growth*
- *multiplicative regression*
- *decreasing growth rate*
- *logistic growth*
- *graphical comparison between similar communities*
- *method of ratio and correlation*
- *prediction based on employment forecast or other utilities forecast*

Tables 2.1 and 2.2 list the main characteristics of the various methods. All of the methods presented in Table 2.1 can also be solved through statistical regression analysis (linear or non-linear). Such methods are found in many commercially available computer programs. Whenever possible it is always best to adopt a regression analysis that allows the incorporation of a largest historical data series instead of two or three 3 points, such as the algebraic methods presented in Table 2.1.

The results of the population forecast must be coherent with the population density in the area under analysis. The population density data are also useful in the computation of flows and loads resulting from a certain area or basin within the town. Typical population density values are presented in Table 2.3. Table 2.4 presents typical saturation population densities, in highly occupied metropolitan areas.

When making population forecasts, the following points must be taken into consideration:

- The population studies are normally very complex. All the variables (unfortunately not always quantifiable) that could interact in the specific locality under study must be analysed. Unexpected events can still occur, which can completely change the predicted trajectory of the population growth. This emphasises the need to establish a realistic value for the planning horizon and for the implementation stages of the WWTP.
- The mathematical sophistication associated with the determination of the coefficients of some forecast equations loses its meaning if it is not based on parallel information, often non-quantifiable, such as social, economical, geographical and historical aspects.
- The common sense of the analyst is very important in the choice of the forecast and in the interpretation of the results. Even though the choice of method is based on the best fit with census data, the extrapolation of the curve requires perception and caution.

Table 2.2. Population forecast based on indirect quantification methods

Method	Description
Graphical comparison	The method involves the graphical fitting of the past population under study. The population data of other similar but larger towns are plotted in a manner that the curves coincide at the current value of the population of the town under study. These curves are used as references for the forecast of the town under study.
Ratio and correlation	It is assumed that the town population follows the same trend of the region (physical or political region) in which it is inserted. Based on the census records, the ratio "town population / region population" is calculated and projected for future years. The town population is obtained from the population forecast for the region (made at a planning level by another body) and the calculated ratio.
Forecast of employment and utility services	The population is estimated using a job prediction (made by another body). Based on the past population data and people employed, the "job/population" ratio is calculated and projected for future years. The town population is calculated from the forecast of the number of jobs in the town. The procedure is similar to the ratio method. The same methodology can be adopted from the forecast of utility services, such as electricity, water, telephone, etc. The service utility companies normally undertake studies of forecast and expansion of their services with relative reliability.

Note: The forecast of the ratios can be done based on regression analysis
Source: Qasim (1985)

Table 2.3. Typical population densities as a function of land use

	Population density	
Land use	(inhab/ha)	(inhab/km^2)
Residential areas		
• single-family dwellings, large lots	12–36	1,200–3,600
• single-family dwellings, small lots	36–90	3,600–9,000
• multiple-family dwellings, small lots	90–250	9,000–25,000
Apartments	250–2,500	25,000–250,000
Commercial areas	36–75	3,600–7,500
Industrial areas	12–36	1,200–3,600
Total (excluding parks and other large-scale equipment)	25–125	2,500–12,500

Source: adapted from Fair et al (1973) and Qasim (1985) (rounded up values)

Table 2.4. Population densities and average street length per hectare, under saturation conditions, in highly occupied metropolitan areas

Land use	Saturation population density (inhab/ha)	Average street length (m/ha)
High standard residential areas, with standard lots of 800 m^2	100	150
Intermediate standard residential areas, with standard lots of 450 m^2	120	180
Popular residential areas, with standard lots of 250 m^2	150	200
Centrally-located mixed residential–commercial areas, with predominance of 3–4 storey buildings	300	150
Centrally-located residential areas, with predominance of 10–12 storey buildings	450	150
Mixed residential–commercial–industrial urban areas, with predominance of commerce and small industries	600	150
Centrally-located residential areas, with predominance of office buildings	1000	200

Average data from São Paulo Metropolitan Area, Brazil
Source: Alem Sobrinho and Tsutiya (1999)

Example 2.1

Based on the following census records, undertake the population forecast using the methods based on mathematical formulas (Table 2.1). Data:

Year	Population (inhabitants)
1980	10,585
1990	23,150
2000	40,000

Solution:

a) Nomenclature of the years and populations

According to Table 2.1, there is the following nomenclature:

$t_0 = 1980$ $P_0 = 10{,}585$ inhab
$t_1 = 1990$ $P_1 = 23{,}150$ inhab
$t_2 = 2000$ $P_2 = 40{,}000$ inhab

b) Linear (arithmetic) growth

$$K_a = \frac{P_2 - P_o}{t_2 - t_o} = \frac{40000 - 10585}{2000 - 1980} = 1470.8$$

$$P_t = P_o + K_a.(t - t_o) = 10585 + 1470.8 \times (t - 1980)$$

***Example 2.1* (*Continued*)**

For example, to calculate the population in the year 2005, t is substituted for 2005 in the above equation. For the year 2010, t = 2010, and so on.

c) Geometric growth

$$K_g = \frac{\ln P_2 - \ln P_o}{t_2 - t_o} = \frac{\ln 40000 - \ln 10585}{2000 - 1980} = 0.0665$$

$$P_t = P_0.e^{K_g.(t-t_0)} = 10585.e^{0.0665\times(t-1980)}$$

d) Decreasing growth rate

$$P_s = \frac{2.P_o.P_1.P_2 - P_1{}^2.(P_o + P_2)}{P_o.P_2 - P_1{}^2}$$

$$= \frac{2 \times 10585 \times 23150 \times 40000 - 23150^2 \times (10585 + 40000)}{10585 \times 40000 - 23150^2}$$

$$= 66709$$

The saturation population is, therefore, 66,709 inhabitants.

$$K_d = \frac{-\ln[(P_s - P_2)/(P_s - P_o)]}{t_2 - t_o}$$

$$= \frac{-\ln[66709 - 40000)/(66709 - 10585)]}{2000 - 1980} = 0.0371$$

$$P_t = P_O + (P_s - P_o).\left[1 - e^{-K_d.(t-t_o)}\right]$$

$$= 10585 + (66709 - 10585) \times \left(1 - e^{-0,0371\times(t-1980)}\right)$$

e) Logistic growth

$$P_s = \frac{2.P_o.P_1.P_2 - P_1{}^2.(P_o + P_2)}{P_o.P_2 - P_1{}^2}$$

$$= \frac{2 \times 10585 \times 23150 \times 40000 - 23150^2 \times (10585 + 40000)}{10585 \times 40000 - 23150^2}$$

$$= 66709$$

$$c = \frac{(P_s - P_o)}{P_o} = \frac{(66709 - 10585)}{10585} = 5.3022$$

$$K_1 = \frac{1}{t_2 - t_1}.\ln\left[\frac{P_o.(P_s - P_1)}{P_1.(P_s - P_o)}\right]$$

$$= \frac{1}{2000 - 1990}.\ln\left[\frac{10585 \times (66709 - 23150)}{23150 \times (66709 - 10585)}\right] = -0,1036$$

Example 2.1 (_Continued_)

$$P_t = \frac{P_s}{1 + c.e^{K_1.(t-t_0)}} = \frac{66709}{1 + 5.3022.e^{-0.1036\times(t-1980)}}$$

The inflexion in the S-shaped curve occurs at the following year and population:

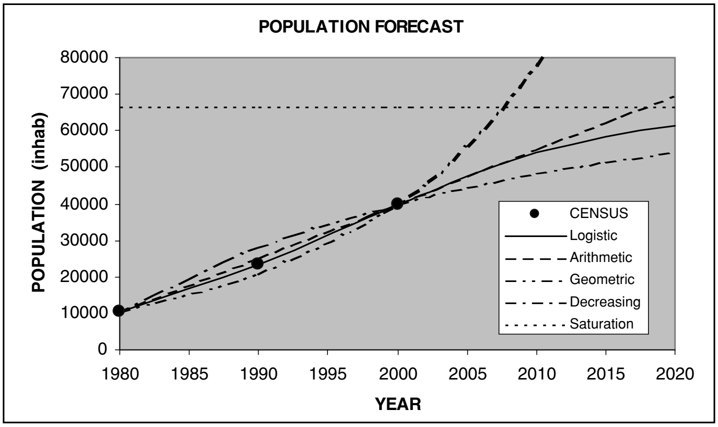
$$\text{Inflexion time} = t_o - \frac{\ln(c)}{K_1} = 1980 - \frac{\ln(5.3022)}{-0.1036} = 1996$$

$$\text{Population at inflexion} = \frac{P_s}{2} = \frac{66709}{2} = 33354 \text{ inhab}$$

Before inflexion (year 1996), population growth presented an increasing rate and, after it, a decreasing rate.

f) Results in table and graphic form

			Population forecast			
Nomenclature	Year	Actual population (census)	Linear	Geometric	Decreasing rate	Logistic
P_0	1980	10585	10585	10585	10585	10585
P_1	1990	23150	25293	20577	27992	23150
P_2	2000	40000	40000	40000	40000	40000
–	2005	–	47354	55770	44525	47725
–	2010	–	54708	77758	48284	53930
–	2015	–	62061	108414	51405	58457
–	2020	–	69415	151157	53998	61534



Population forecast. Census and estimated data

Example 2.1 (Continued)

From the graph and table, the following points specific for this data group can be seen:

- The census data (population of the years 1980 to 2000) present an increasing growth rate trend. Visually, it is seen that the decreasing rate model does not fit well.
- The geometric method leads to very high future estimates (that can turn out to be true or not, but that are far away from the other forecasts).
- The logistic and decreasing rate methods tend to the saturation population. (66,709 inhabitants, indicated on the graph)
- In all methods, the calculated population values for the years P_0 and P_2 are equal to the measured values.
- The population forecast as such is only from year 2000. The years with census data are plotted to permit the visual interpretation of the fit of the curves to the measured data (1980, 1990, 2000).
- The best-fit curve may be chosen from statistical criteria, which give an indication of the prediction error (usually based on the sum of the squared errors), where error or residual is the difference between the estimated and the observed data.
- Spreadsheets may be used, to find the value of the coefficients that lead to the minimum sum of the squared errors (e.g. solver tool in Excel®).

2.1.2.3 Average water consumption

As mentioned, the domestic flow is a function of the water consumption. Typical values of per capita water consumption for populations provided with household water connections are presented in Table 2.5.

These values can vary from locality to locality. Table 2.6 presents various factors that influence water consumption. The data listed in Table 2.5 are simply typical average values, being naturally subjected to all the variability resulting from the factors listed in Table 2.6.

Table 2.5. Typical ranges of per capita water consumption

Community size	Population range (inhabitants)	Per capita water consumption (L/inhab.d)
Rural settlement	<5,000	90–140
Village	5,000–10,000	100–160
Small town	10,000–50,000	110–180
Average town	50,000–250,000	120–220
Large city	>250,000	150–300

Note: in places with severe water shortages, these values may be smaller
Source: Adapted from CETESB (1977; 1978), Barnes et al (1981), Dahlhaus & Damrath (1982), Hosang & Bischof (1984)

Table 2.6. Factors that influence water consumption

Influencing factor	Comment
• Water availability	• In locations of water shortage consumption tends to be less
• Climate	• Warmer climates induce a greater water consumption
• Community size	• Larger cities generally present a larger per capita water consumption (to account for strong commercial and institutional activities)
• Economic level of the community	• A higher economic level is associated with a higher water consumption
• Level of industrialisation	• Industrialised locations present a higher consumption
• Metering of household consumption	• Metering inhibits greater consumption
• Water cost	• A higher cost reduces consumption
• Water pressure	• High pressure in the distribution system induces greater use and wastage
• System losses	• Losses in the water distribution network imply the necessity of a greater water production

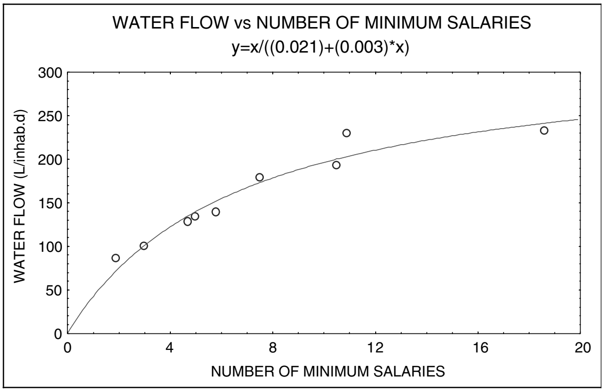


Figure 2.2. Per capita water consumption as a function of family salary in Belo Horizonte, Brazil

Campos and von Sperling (1996) observed, for predominantly residential sewage originating from nine sub-catchment areas in Belo Horizonte, Brazil, a strong relationship between per capita water consumption and average monthly family income (in number of minimum salaries) (Figure 2.2). Naturally the data are site specific and require caution in their extrapolation to other conditions.

Water consumption data from 45 municipalities in the State of Minas Gerais, Brazil (von Sperling et al, 2002), were investigated by the author. The State of Minas Gerais has many features in common with Brazil, as a whole, and many

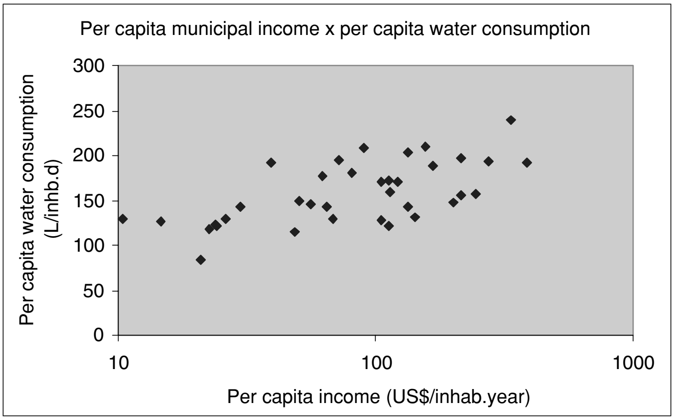


Figure 2.3. Relationship between per capita water consumption and per capita income. Data from the state of Minas Gerais, Brazil (von Sperling et al, 2002) (US$1.00 = R$2.50)

other developing countries, because it presents regions with high and low economic level, rainfall and temperature. The range of variation in the data was: per capita water consumption: 84 to 248 l/inhab.d; urban population: 4,000–2,300,000 inhabitants; average per capita income: US$8–1600 per inhabitant per year; mean yearly temperature: 20–26° C; mean yearly rainfall: 300–1750 mm/year. Figure 2.3 presents the relation of the per capita water consumption with per capita income, which was the clearer one. The analysis should be done only in terms of trends and average values, since the correlation coefficient was not high, as a result of a substantial scatter in the data.

Figure 2.4 presents the ranges of variation of the per capita water consumption as a function of the category of the per capita income and rainfall of the 45 municipalities (separation between low and high income: US$110/inhab.year, corresponding to the median of values; separation of high and low rainfall: 1350 mm/year, corresponding to the average value of the State of Minas Gerais). Naturally these values are region specific, but it is believed that a certain extrapolation of trends and ranges can be done, but always judiciously.

Table 2.7 shows ranges of per capita water consumption as a function of income and rainfall, based on the 25 and 75 percentiles presented in Figure 2.4.

Tables 2.8 and 2.9 show the ranges of average water consumption values for various commercial establishments and institutions. This information, which should only be used in the absence of more specific data, is particularly useful in the design of sewage treatment works for small areas, in which the contribution of individual important establishments could have an importance in the general flow calculations.

2.1.2.4 Average sewage flow

In general, the production of sewage corresponds approximately to the water consumption. However, the fraction of the sewage that enters the sewerage system can

Table 2.7. Ranges of water consumption values, based on 45 municipalities in the State of Minas Gerais, Brazil

	Ranges of per capita water consumption (L/inhab.d)	
Income	Low rainfall	High rainfall
Low	120–165	130–190
High	140–180	150–200

Notes:

- Ranges based on 25 and 75 percentile values from Fig. 2.4
- In larger towns (greater than 200,000 inhabitants), the per capita water consumption was on average approximately 10% higher than in smaller towns
- The ranges present usual values, and it is frequent to observe values outside them

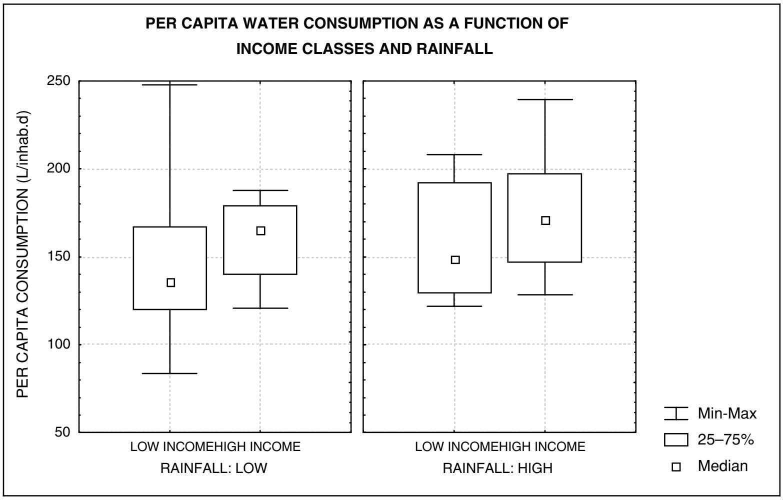


Figure 2.4. Box-and-whisker plot of the per capita water consumption values as a function of categories for per capita income and mean yearly rainfall (45 municipalities in the State of Minas Gerais, Brazil)

be different, due to the fact that part of the water consumed could be incorporated into the storm water system or infiltrate (e.g. watering of gardens and parks). Other influencing factors in a separate sewerage system are: (a) clandestine sewage connections to the storm water system, (b) clandestine connections of storm water into the separate sewerage system and (c) infiltration. The last point is covered separately in Section 2.1.3.

The fraction of the supplied water that enters the sewerage system in the form of sewage is called Return Coefficient (R = sewage flow/water flow). Typical values vary between 60% and 100%, and a value of 80% (R = 0.8) is usually adopted.

Table 2.8. Typical water consumption in some commercial establishments

Establishment	Unit	Flow range (L/unit.d)
Airport	Passenger	8–15
Accommodation (lodging house)	Resident	80–150
Public toilet	User	10–25
Bar	Customer	5–15
Cinema/theatre	Seat	2–10
Office	Employee	30–70
Hotel	Guest	100–200
	Employee	30–50
Industry (sanitary sewage only)	Employee	50–80
Snack bar	Customer	4–20
Laundry – commercial	Machine	2,000–4,000
Laundry – automatic	Machine	1,500–2,500
Shop	Toilet	1,000–2,000
	Employee	30–50
Department store	Toilet	1,600–2,400
	Employee	30–50
	m^2 of area	5–12
Petrol station	Vehicle attended	25–50
Restaurant	Meal	15–30
Shopping centre	Employee	30–50
	m^2 of area	4–10

Source: EPA (1977), Hosang and Bischof (1984), Tchobanoglous and Schroeder (1985), Qasim (1985), Metcalf & Eddy (1991), NBR-7229/93

Table 2.9. Typical water consumption in some institutional establishments

Establishment	Unit	Flow range (L/unit.d)
Rest home	Resident	200–450
	Employee	20–60
School	Student	50–100
– with cafeteria, gymnasium, showers	Student	40–80
– with cafeteria only	Student	20–60
– without cafeteria and gymnasium		
Hospital	Bed	300–1000
	Employee	20–60
Prison	Inmate	200–500
	Employee	20–60

Source: EPA (1977), Hosang and Bischof (1984), Tchobanoglous and Schroeder (1985), Qasim (1985), Metcalf & Eddy (1991)

The average domestic sewage flow calculation is given by:

$$\boxed{Q_{d_{av}} = \frac{Pop.L_{pcd}.R}{1000} \quad (m^3/d)} \tag{2.2}$$

$$\boxed{Q_{d_{av}} = \frac{Pop.L_{pcd}.R}{86400} \quad (L/s)} \tag{2.3}$$

where:

$Q_{d_{av}}$ = average domestic sewage flow (m^3/d or L/s)
L_{pcd} = per capita water consumption (L/inhab.d)
R = sewage flow/water flow return coefficient

It is important to notice that the water flow to be considered is the flow actually **consumed**, and not the flow produced by the water treatment works. The water flow produced is higher than that consumed due to unaccounted water losses in the distribution system, which can vary typically from 20 to 50%. Thus in a locality where the loss is 30%, for each 100 m^3 of water produced, 30 m^3 are unaccounted for and only 70 m^3 are consumed. Of this 70 m^3, around 80% (56 m^3/d) return in the form of sewage to the sewerage system.

2.1.2.5 Flow variations. Maximum and minimum flows

Water consumption and wastewater generation in a locality vary throughout the day (hourly variations), during the week (daily variations) and throughout the year (seasonal variations).

Figure 2.5 presents typical hourly influent flowrate variations in a WWTP. Two main peaks can be observed: a peak at the beginning of the morning (more pronounced) and a peak at the beginning of the evening (more distributed). The average daily flow corresponds to the line that separates equal areas, below and above the line.

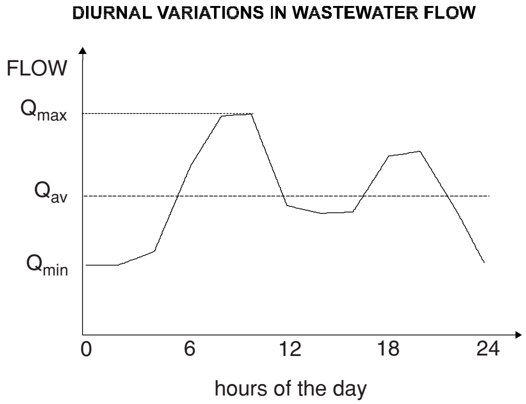


Figure 2.5. Typical hourly flow variations in the influent to a sewage treatment works

The following coefficients are frequently used to allow the estimation of minimum and maximum **water** flows:

- $K_1 = 1.2$ (peak coefficient for the day with the highest water consumption)
- $K_2 = 1.5$ (peak coefficient for the hour with the highest water consumption)
- $K_3 = 0.5$ (reduction coefficient for the hour with the lowest water consumption)

Table 2.10. Coefficient of hourly variation of sewage flow

Q_{max}/Q_{av}	Q_{min}/Q_{av}	Author	Reference
$1+(14/(4+P^{0.5}))$	–	Harmon	Qasim (1985)
$5P^{-0.16}$	$0.2P^{0.16}$	Gifft	Fair et al (1973)

Notes:
P = population, in thousands
Gifft's formula is indicated for $P < 200$ (population $< 200{,}000$ inhabitants)

Thus, the maximum and minimum water flows can be given by the formulas:

$$Q_{max} = Q_{av} \, . \, K_1 \, . \, K_2 = 1.8\, Q_{av} \qquad (2.4)$$

$$Q_{min} = Q_{av} \, . \, K_3 = 0.5\, Q_{av} \qquad (2.5)$$

If it is possible to carry out flow measurements, to establish the real flow variations, the actual data should be used in the design. The coefficients K_1, K_2 and K_3 are generalised, thus probably not allowing the accurate reproduction of the flow variations in the locality under analysis. Over- or underestimated values affect directly the technical and economical performance of the sewage works design.

When considering hourly variations of **wastewater** flow, it should be taken into consideration that the fluctuations are absorbed and reduced in amplitude along the sewerage system. It is easy to understand that the larger the network (or the population), the lower are the chances of peak flows to overlap simultaneously in the works entrance. Thus the residence time in the sewerage system has a large influence on the absorption of the peak flows. Based on this concept, some authors have developed formulas for correlating the coefficients of variation with population, or with average flow (Table 2.10). As an illustration, the following table presents the calculated coefficients for different populations.

	Q_{max}/Q_{av}		Q_{min}/Q_{av}
Population	Harmon	Gifft	Gifft
1,000	3.8	5.0	0.20
10,000	3.0	3.4	0.14
100,000	2.0	2.3	0.09
1,000,000	1.4	-	-

It can be observed that even the product of the coefficients K_1 and K_2 utilised for water supply, and frequently adopted as 1.8, could induce an underestimated ratio Q_{max}/Q_{av} for a wide population range.

Table 2.11. Approximate values of infiltration rates in sewerage systems

Pipe diameter	Type of joint	Groundwater level	Soil permeability	Infiltration coefficient L/s.km	Infiltration coefficient m^3/d.km
< 400 mm	Elastic	Below the pipes	Low	0.05	4
			High	0.10	9
		Above the pipes	Low	0.15	13
			High	0.30	26
	Non-elastic	Below the pipes	Low	0.05	4
			High	0.50	43
		Above the pipes	Low	0.50	43
			High	1.00	86
> 400 mm	–	–	–	1.00	86

Source: Crespo (1997)

2.1.3 Infiltration flow

Infiltration in a sewerage system occurs through defective pipes, connections, joints or manholes. The quantity of infiltrated water depends on various factors, such as the extension of the collection network, pipeline diameters, drainage area, soil type, water table depth, topography and population density (number of connections per unit area) (Metcalf & Eddy, 1991).

When no specific local data are available, infiltration rate is normally expressed in terms of flow per extension of the sewerage system or per area served. The values presented in Table 2.11 can be used as a first estimate, when no specific local data are available (Crespo, 1997).

Metcalf & Eddy (1991) present the infiltration coefficient as a function of the pipe diameter: **0.01 to 1.0 m^3/d.km per mm**. For instance, for a pipe diameter of 200 mm, the infiltration rate will range between 2 to 200 m^3/d.km.

The length of the network may be measured in the locality by using the map of the location of the sewerage system. In the absence of these data (for instance, for future populations), in preliminary studies of smaller localities, where the population density is usually less, values around 2.5 to 3.5 m of network per inhabitant may be adopted. In medium-size cities this value could be reduced to around 2.0 to 3.0 m/inhab and in densely populated regions, even smaller values may be reached (1.0 to 2.0 m/inhab or even lower). Figure 2.6, based on the 45 municipalities described in Section 2.1.2, presents the ranges of variation of per capita length of sewerage network for two population categories.

Based on the infiltration values per unit length and the per capita sewerage network length, per capita infiltration values may be estimated to range between **8 to 150 L/inhab.d**, excluding the extreme values. In areal terms, based on typical population densities (25 to 125 inhab/ha), infiltration rates between **0.2 and 20 m^3/d per ha** of drainage area (**20 to 2000 m^3/d.km^2**) are obtained. These ranges are very wide, and the designer should analyse carefully the prevailing conditions in

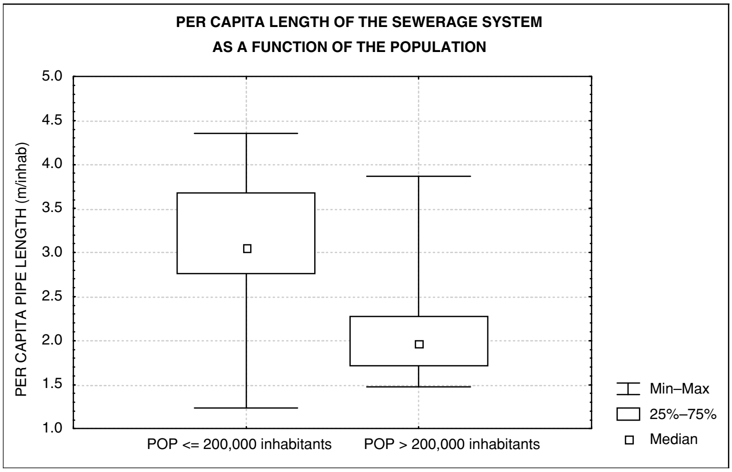


Figure 2.6. Box-and-whisker plot of the per capita length of sewerage network, as a function of two categories of population size (45 municipalities in Minas Gerais, Brazil)

the sewerage network in order to obtain narrower ranges, which could best represent the specific conditions in the community under analysis. The utilisation of good materials and construction procedures helps in reducing the infiltration rates.

In the calculation of the total influent flow to a WWTP, average infiltration values may be used for the computation of average and maximum influent flowrates. For minimum flow conditions, infiltration can be excluded, as a safety measure (in the case of minimum flow, the safety in a design is in the direction of establishing the lowest flow).

2.1.4 Industrial wastewater flow

Industrial wastewater flow is a function of the type and size of the industry, manufacturing process, level of recycling, existence of pre-treatment, etc. Even in the case of two industries that manufacture essentially the same product, the wastewater flows can diverge substantially.

If there are large industries contributing to the public sewerage system and subsequently to a WWTP, the adequate evaluation of their respective flows is of great importance. Industrial wastewater has a great influence in the planning and operation of a WWTP. Specific data must be obtained for each significant industry, through industrial surveys, thus allowing the supply of data of interest for the project. With relation to the water consumption and the generation of wastewater, the following information at least must be obtained for the main industries:

- *Water consumption*
    - Total volume consumed (per day or month)
    - Volume consumed in the various stages of the process

  - Internal recirculations
  - Water origin (public supply, wells, etc.)
  - Internal systems of water treatment
- *Wastewater production*
  - Total flow
  - Number of discharge points (with the corresponding industrial process associated with each point)
  - Discharge pattern (continuous or intermittent; duration and frequency) in each discharge point
  - Discharge destination (sewerage system, watercourse)
  - Occasional mixing of wastewater with domestic sewage and storm water

Additionally, whenever possible, effluent flow measurements must be carried out throughout the working day, to record the discharge pattern and variations.

In the event of having no specific information available for the industry, Table 2.12 can be used as a starting point to allow estimation of the probable effluent flow range. These values are presented in terms of water consumption per unit of product manufactured. For simplicity it can be assumed that sewage flow is equal to water consumption. It can be seen from the table that there is a great variety of consumption values for the same type of industry. If there are no specific data available for the industry in question, specific literature references relative to the industrial process in focus must be consulted. The table presented only gives a starting point for more superficial or general studies.

The daily discharge pattern for industrial wastewater does not follow the domestic flow variations, changing substantially from industry to industry. Industrial flow peaks do not necessarily coincide with the domestic peaks, that is to say, the total maximum flow (domestic + industrial) is normally less than the simple sum of the maximum flows.

2.2 WASTEWATER COMPOSITION

2.2.1 Quality parameters

Domestic sewage contains approximately 99.9% water. The remaining part includes organic and inorganic, suspended and dissolved solids, together with microorganisms. It is because of this 0.1% that water pollution takes place and the wastewater needs to be treated.

The composition of the wastewater is a function of the uses to which the water was submitted. These uses, and the form with which they were exercised, vary with climate, social and economic situation and population habits.

In the design of a WWTP, there is normally no interest in determining the various compounds that make up wastewater. This is due, not only to the difficulty in

Table 2.12. Specific average flows from some industries

Type	Activity	Unit	Water consumption per unit (m^3/unit) (*)
Food	Canned fruit and vegetables	1 tonne product	4–50
	Sweets	1 tonne product	5–25
	Sugar cane	1 tonne sugar	0.5 – 10.0
	Slaughter houses	1 cow or 2,5 pig	0.5–3.0
	Dairy (milk)	1000 L milk	1–10
	Dairy (cheese or butter)	1000 L milk	2–10
	Margarine	1 tonne margarine	20
	Brewery	1000 L beer	5–20
	Bakery	1 tonne bread	2–4
	Soft drinks	1000 L soft drinks	2–5
Textiles	Cotton	1 tonne product	120–750
	Wool	1 tonne product	500–600
	Rayon	1 tonne product	25–60
	Nylon	1 tonne product	100–150
	Polyester	1 tonne product	60–130
	Wool washing	1 tonne wool	20–70
	Dyeing	1 tonne product	20–60
Leather / tanneries	Tannery	1 tonne hide	20–40
	Shoe	1000 pairs of shoes	5
Pulp and paper	Pulp fabrication	1 tonne product	15–200
	Pulp bleaching	1 tonne product	80–200
	Paper fabrication	1 tonne product	30–250
	Pulp and paper integrated	1 tonne product	200–250
Chemical industries	Paint	1 employee	110 L/d
	Glass	1 tonne glass	3–30
	Soap	1 tonne soap	25–200
	Acid, base, salt	1 tonne chlorine	50
	Rubber	1 tonne product	100–150
	Synthetic rubber	1 tonne product	500
	Petroleum refinery	1 barrel (117 L)	0.2–0.4
	Detergent	1 tonne product	13
	Ammonia	1 tonne product	100–130
	Carbon dioxide	1 tonne product	60–90
	Petroleum	1 tonne product	7–30
	Lactose	1 tonne product	600–800
	Sulphur	1 tonne product	8–10
	Pharmaceutical products (vitamins)	1 tonne product	10–30
Manufacturing products	Precision mechanics, optical, electronic	1 employee	20–40 L/d
	Fine ceramic	1 employee	40 L/d
	Machine industry	1 employee	40 L/d
Metallurgy	Foundry	1 tonne pig iron	3–8
	Lamination	1 tonne product	8–50
	Forging	1 tonne product	80
	Electroplating	1 m^3 of solution	1–25
	Iron and steel plating industry	1 employee	60 L/d
Mining	Iron	1 m^3 mineral taken	16
	Coal	1 tonne coal	2–10

* Consumption in m^3 per unit produced or L/d per employee

Source: CETESB (1976), Downing (1978), Arceivala (1981), Hosang and Bischof (1984), Imhoff & Imhoff (1985), Metcalf & Eddy (1991), Derísio (1992)

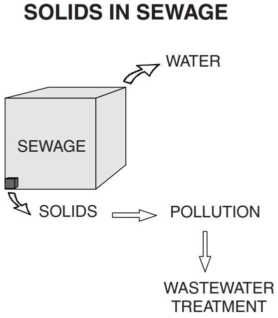


Figure 2.7. Solids in sewage

undertaking the various laboratory tests, but also to the fact that the results themselves cannot be directly utilised as elements in design and operation. Therefore, many times it is preferable to utilise indirect parameters that represent the character or the polluting potential of the wastewater in question. These parameters define the quality of the sewage, and can be divided into three categories: *physical*, *chemical* and *biological* parameters.

2.2.2 Main characteristics of wastewater

Tables 2.13, 2.14 and 2.15 present the main physical, chemical and biological characteristics of domestic sewage.

Table 2.13. Main physical characteristics of domestic sewage

Parameter	Description
Temperature	• Slightly higher than in drinking water • Variations according to the seasons of the years (more stable than the air temperature) • Influences microbial activity • Influences solubility of gases • Influences viscosity of the liquid
Colour	• Fresh sewage: slight grey • Septic sewage: dark grey or black
Odour	• Fresh sewage: oily odour, relatively unpleasant • Septic sewage: foul odour (unpleasant), due to hydrogen sulphide gas and other decomposition by-products • Industrial wastewater: characteristic odours
Turbidity	• Caused by a great variety of suspended solids • Fresher or more concentrated sewage: generally greater turbidity

Source: Adapted from Qasim (1985)

Table 2.14. Main chemical characteristics of domestic sewage

Parameter	Description
TOTAL SOLIDS	*Organic and inorganic; suspended and dissolved; settleable*
• *Suspended*	• Part of organic and inorganic solids that are non-filterable
• *Fixed*	• Mineral compounds, not oxidisable by heat, inert, which are part of the suspended solids
• *Volatile*	• Organic compounds, oxidisable by heat, which are part of the suspended solids
• *Dissolved*	• Part of organic and inorganic solids that are filterable. Normally considered having a dimension less than 10^{-3} μm.
• *Fixed*	• Mineral compounds of the dissolved solids.
• *Volatile*	• Organic compounds of the dissolved solids
• *Settleable*	• Part of organic and inorganic solids that settle in 1 hour in an Imhoff cone. Approximate indication of the settling in a sedimentation tank.
ORGANIC MATTER	*Heterogeneous mixture of various organic compounds. Main components: proteins, carbohydrates and lipids.*
Indirect determination	
• *BOD_5*	• Biochemical Oxygen Demand. Measured at 5 days and 20 °C. Associated with the biodegradable fraction of carbonaceous organic compounds. Measure of the oxygen consumed after 5 days by the microorganisms in the biochemical stabilisation of the organic matter.
• *COD*	• Chemical Oxygen Demand. Represents the quantity of oxygen required to chemically stabilise the carbonaceous organic matter. Uses strong oxidising agents under acidic conditions.
• *Ultimate BOD*	• Ultimate Biochemical Oxygen Demand. Represents the total oxygen consumed at the end of several days, by the microorganisms in the biochemical stabilisation of the organic matter.
Direct determination	
• *TOC*	• Total Organic Carbon. Direct measure of the carbonaceous organic matter. Determined through the conversion of organic carbon into carbon dioxide.
TOTAL NITROGEN	*Total nitrogen includes organic nitrogen, ammonia, nitrite and nitrate. It is an essential nutrient for microorganisms' growth in biological wastewater treatment. Organic nitrogen and ammonia together are called Total Kjeldahl Nitrogen (TKN).*
• *Organic nitrogen*	• Nitrogen in the form of proteins, aminoacids and urea.
• *Ammonia*	• Produced in the first stage of the decomposition of organic nitrogen.
• *Nitrite*	• Intermediate stage in the oxidation of ammonia. Practically absent in raw sewage.
• *Nitrate*	• Final product in the oxidation of ammonia. Practically absent in raw sewage.
TOTAL PHOSPHORUS	*Total phosphorus exists in organic and inorganic forms. It is an essential nutrient in biological wastewater treatment.*
• *Organic phosphorus*	• Combined with organic matter.
• *Inorganic phosphorus*	• Orthophosphates and polyphosphates.

(*Continued*)

Table 2.14 (*Continued*)

Parameter	Description
pH	*Indicator of the acidic or alkaline conditions of the wastewater. A solution is neutral at pH 7. Biological oxidation processes normally tend to reduce the pH.*
ALKALINITY	*Indicator of the buffer capacity of the medium (resistance to variations in pH). Caused by the presence of bicarbonate, carbonate and hydroxyl ions.*
CHLORIDES	*Originating from drinking water and human and industrial wastes.*
OILS AND GREASE	*Fraction of organic matter which is soluble in hexane. In domestic sewage, the sources are oils and fats used in food.*

Source: adapted from Arceivala (1981), Qasim (1985), Metcalf & Eddy (1991)

Table 2.15. Main organisms present in domestic sewage

Organism	Description
Bacteria	• Unicellular organisms • Present in various forms and sizes • Main organisms responsible for the stabilisation of organic matter • Some bacteria are pathogenic, causing mainly intestinal diseases
Archaea	• Similar to bacteria in size and basic cell components • Different from bacteria in their cell wall, cell material and RNA composition • Important in anaerobic processes
Algae	• Autotrophic photosynthetic organisms, containing chlorophyll • Important in the production of oxygen in water bodies and in some sewage treatment processes • In lakes and reservoirs they can proliferate in excess, deteriorating the water quality
Fungi	• Predominantly aerobic, multicellular, non-photosynthetic, heterotrophic organisms • Also of importance in the decomposition of organic matter • Can grow under low pH conditions
Protozoa	• Usually unicellular organisms without cell wall • Majority is aerobic or facultative • Feed themselves on bacteria, algae and other microorganisms • Essential in biological treatment to maintain an equilibrium between the various groups • Some are pathogenic
Viruses	• Parasitic organisms, formed by the association of genetic material (DNA or RNA) and a protein structure • Pathogenic and frequently difficult to remove in water or wastewater treatment
Helminths	• Higher-order animals • Helminth eggs present in sewage can cause illnesses

Note: algae are normally not present in untreated wastewater, but are present in the treated effluent from some processes (e.g. stabilisation ponds)
Source: Silva & Mara (1979), Tchobanoglous & Schroeder (1985), Metcalf & Eddy (1991), 2003

2.2.3 Main parameters defining the quality of wastewater

2.2.3.1 Preliminaries

The main parameters predominantly found in domestic sewage that deserve special consideration are:

- *solids*
- *indicators of organic matter*
- *nitrogen*
- *phosphorus*
- *indicators of faecal contamination*

2.2.3.2 Solids

All the contaminants of water, with the exception of dissolved gases, contribute to the solids load. In wastewater treatment, the solids can be classified according to (a) their size and state, (b) their chemical characteristics and (c) their settleability:

Solids in sewage

- *Classification by size and state*
  - Suspended solids (non-filterable)
  - Dissolved solids (filterable)
- *Classification by chemical characteristics*
  - Volatile solids (organic)
  - Fixed solids (inorganic)
- *Classification by settleability*
  - Settleable suspended solids
  - Non-settleable suspended solids

a) Classification by size

The division of solids by size is above all a practical division. For convention it can be said that particles of smaller dimensions capable of passing through a filter paper of a specific size correspond to the **dissolved solids**, while those with larger dimensions and retained by the filter are considered **suspended solids**. To be more precise, the terms *filterable* (=dissolved) solids and *non-filterable* (=suspended) solids are more adequate. In an intermediate range there are the colloidal solids, which are of importance in water treatment, but are difficult to identify by the simple method of paper filtration. Water analysis results based on typical filter papers show that the major part of colloidal solids is separated as filterable (dissolved) solids.

Sometimes the term *particulate* is used to indicate that the solids are present as suspended solids. In this context, expressions as particulate BOD, COD, phosphorus, etc. are used, to indicate that they are linked to suspended solids. In contrast, soluble BOD, COD and phosphorus are associated with dissolved solids.

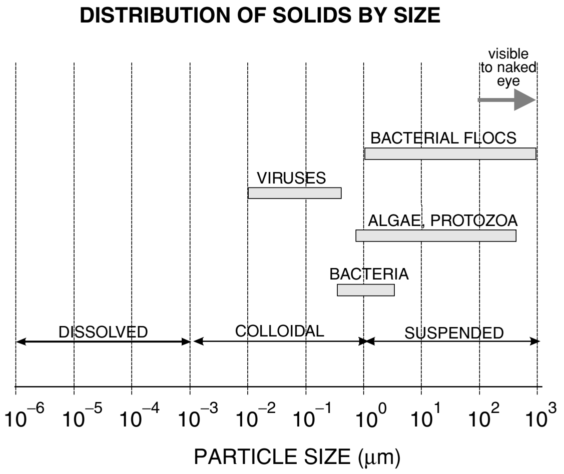


Figure 2.8. Classification and distribution of solids as a function of size

Figure 2.8 shows the distribution of particles by size. In a general manner, are considered dissolved solids those with a diameter of less than 10^{-3} µm, colloidal solids those with a diameter between 10^{-3} and 10^{0} µm and as suspended solids those with a diameter greater than 10^{0} µm.

b) Classification by chemical characteristics

If the solids are submitted to a high temperature (550 °C), the organic fraction is oxidised (volatilised), leaving after combustion only the inert fraction (unoxidised). The *volatile solids* represent an estimate of the *organic* matter in the solids, while the *non-volatile solids* (fixed) represent the *inorganic* or mineral matter. In summary:

↗ Volatile solids (organic matter)
Total solids
↘ Fixed solids (inorganic matter)

c) Classification by settleability

Settleable solids are considered those that are able to settle in a period of 1 hour. The volume of solids accumulated in the bottom of a recipient called an Imhoff Cone is measured and expressed as mL/L. The fraction that does not settle represents the non-settleable solids (usually not expressed in the results of the analysis).

Figure 2.9 shows the typical distribution between the various types of solids present in a raw sewage of average composition.

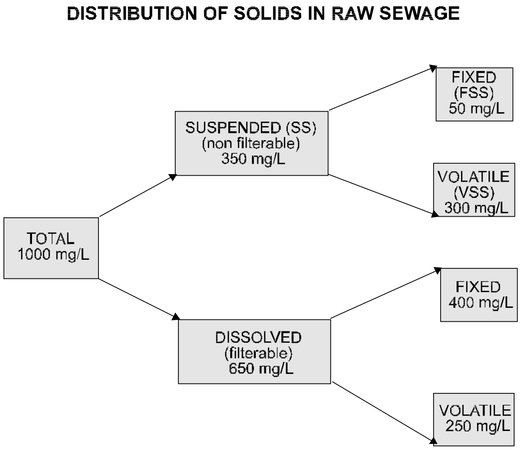


Figure 2.9. Approximate distribution of the solids in raw sewage (in terms of concentration)

2.2.3.3 Carbonaceous organic matter

The organic matter present in sewage is a characteristic of substantial importance, being the cause of one of the main water pollution problems: consumption of dissolved oxygen by the microorganisms in their metabolic processes of using and stabilising the organic matter. The organic substances present in sewage consist mainly of (Pessoa & Jordão, 1982):

- Protein compounds (≈ 40%)
- Carbohydrates (≈ 25 to ≈ 50%)
- Oils and grease (≈ 10%)
- Urea, surfactants, phenols, pesticides and others (lower quantity)

The carbonaceous organic matter (based on organic carbon) present in the influent sewage to a WWTP can be divided into the following main fractions:

Organic matter in sewage

- *classification: in terms of form and size*
  - Suspended (particulate)
  - Dissolved (soluble)
- *classification: in terms of biodegradability*
  - Inert
  - Biodegradable

In practical terms it is not usually necessary to classify organic matter in terms of proteins, fats, carbohydrates, etc. Besides, there is a great difficulty in determining in the laboratory the various components of organic matter in wastewater, in view of the multiple forms and compounds in which it can be present. As a result, direct or indirect methods can be adopted for the quantification of organic matter:

- *Indirect methods: measurement of oxygen consumption*
  - Biochemical Oxygen Demand (BOD)
  - Ultimate Biochemical Oxygen Demand (BOD_u)
  - Chemical Oxygen Demand (COD)
- *Direct methods: measurement of organic carbon*
  - Total Organic Carbon (TOC)

a) Biochemical Oxygen Demand (BOD)

The main ecological effect of organic pollution in a water body is the decrease in the level of dissolved oxygen. Similarly, in sewage treatment using aerobic processes, the adequate supply of oxygen is essential, so that the metabolic processes of the microorganisms can lead to the stabilisation of the organic matter. The basic idea is then to infer the “strength” of the pollution potential of a wastewater by the measurement of the oxygen consumption that it would cause, that is, an indirect quantification of the potential to generate an impact, and not the direct measurement of the impact in itself.

This quantification could be obtained through stoichiometric calculations based on the reactions of oxidation of the organic matter. If the substrate was, for example, glucose ($C_6H_{12}O_6$), the quantity of oxygen required to oxidise the given quantity of glucose could be calculated through the basic equation of respiration. This is the principle of the so-called Theoretical Oxygen Demand (TOD).

In practice, however, a large obstacle is present: the sewage has a great heterogeneity in its composition, and to try to establish all its constituents in order to calculate the oxygen demand based on the chemical oxidation reactions of each of them is totally impractical. Besides, to extrapolate the data to other conditions would not be possible.

The solution found was to measure in the laboratory the consumption of oxygen exerted by a standard volume of sewage or other liquid, in a pre-determined time. It was thus introduced the important concept of **Biochemical Oxygen Demand (BOD)**. The BOD represents the *quantity of oxygen required to stabilise, through biochemical processes, the carbonaceous organic matter*. It is an indirect indication, therefore, of the biodegradable organic carbon.

Complete stabilisation takes, in practical terms, various days (around 20 days or more for domestic sewage). This corresponds to the Ultimate Biochemical Oxygen Demand (BOD_u). However, to shorten the time for the laboratory test, and to allow a comparison of the various results, some standardisations were established:

- the determination is undertaken on the **5th day**. For typical domestic sewage, the oxygen consumption on the fifth day can be correlated with the final total consumption (BOD_u);

- the test is carried out at a temperature of **20°C**, since different temperatures interfere with the bacteria's metabolism, modifying the relation between BOD at 5 days and BOD Ultimate.

The **standard BOD** is expressed as $\mathbf{BOD_5^{20}}$. In this text, whenever the nomenclature BOD is used, implicitly the standard BOD is being assumed.

The BOD test can be understood in this simplified way: on the day of the sample collection, the concentration of dissolved oxygen (DO) in the sample is determined. Five days later, with the sample maintained in a closed bottle and incubated at 20°C, the new DO concentration is determined. This new DO concentration is lower due to the consumption of oxygen during the period. The difference in the DO level on the day zero and day 5 represents the oxygen consumed for the oxidation of the organic matter, being therefore, the BOD_5. Thus, for example, a sample from a water body presented the following results (see Figure 2.10):

DO on day 0: 7 mg/L
DO on day 5: 3 mg/L
$BOD_5 = 7 - 3 = 4$ mg/L

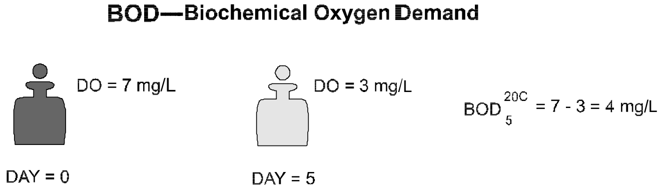


Figure 2.10. Example of the BOD_5^{20} concept

For sewage, some practical aspects require some adaptations. Sewage, having a large concentration of organic matter, consumes quickly (well before the five days) all the dissolved oxygen in the liquid medium. Thus, it is necessary to make dilutions in order to decrease the concentration of the organic matter, such that the oxygen consumption at 5 days is numerically less than the oxygen available in the sample (the sample is lost if, at day 5, the DO concentration is zero, because it will not be possible to know when the zero concentration was reached). Also it is usually necessary to introduce a seed, containing microorganisms, to allow a faster start of the decomposition process. To measure only the carbonaceous oxygen demand, an inhibitor for nitrification (nitrogenous oxygen demand, associated with the oxidation of ammonia to nitrate) can be added. Domestic sewage has a BOD in the region of 300 mg/L, or that is to say, 1 litre of sewage is associated with the consumption of approximately 300 mg of oxygen, in five days, in the process of the stabilisation of the carbonaceous organic matter.

The main **advantages** of the BOD test are related to the fact that the test allows:

- an approximate indication of the biodegradable fraction of the wastewater;
- an indication of the degradation rate of the wastewater;

- an indication of the oxygen consumption rate as a function of time;
- an approximate determination of the quantity of oxygen required for the biochemical stabilisation of the organic matter present.

However, the following **limitations** may be mentioned (Marais & Ekama, 1976):

- low levels of BOD_5 can be found in the case that the microorganisms responsible for the decomposition are not adapted to the waste;
- heavy metals and other toxic substances can kill or inhibit the microorganisms;
- the inhibition of the organisms responsible for the oxidation of ammonia is necessary, to avoid the interference of the oxygen consumption for nitrification (nitrogenous demand) with the carbonaceous demand;
- the ratio of BOD_u/BOD_5 varies with the wastewater;
- the ratio of BOD_u/BOD_5 varies, for the same wastewater, along the WWTP treatment line;
- the test takes five days, being not useful for operational control of a WWTP.

Despite of the limitations above, the BOD test continues to be extensively used, partly for historical reasons and partly because of the following points:

- the design criteria for many wastewater treatment processes are frequently expressed in terms of BOD;
- the legislation for effluent discharge in many countries, and the evaluation of the compliance with the discharge standards, is normally based on BOD.

Substantial research has been directed towards the substitution of BOD by other parameters. In the area of instrumentation, there are respirometric equipments that make automated measurements of the oxygen consumption, allowing a reduction in the period required for the test. However, universality has not yet been reached regarding the parameter or the methodology.

It is observed that the COD test is being more and more used for design, mathematical modelling and performance evaluation. However, the sanitary engineer must be familiar with the interpretation of the BOD and COD tests and know how to work with the complementary information that they both supply.

The present text utilises BOD in items in which the more consolidated international literature is based on BOD, and it uses COD in the items, usually more recent, in which the literature is based more on COD. In this way, it is easier to compare the design parameters presented in this book with international literature parameters.

b) Ultimate Biochemical Oxygen Demand (BOD_u)

The BOD_5 corresponds to the oxygen consumption exerted during the first 5 days. However, at the end of the fifth day the stabilisation of the organic material is still not complete, continuing, though at slower rates, for another period of weeks or days.

Table 2.16. Typical ranges for the BOD_u/BOD_5 ratio

Origin	BOD_u/BOD_5
High concentration sewage	1.1–1.5
Low concentration sewage	1.2–1.6
Primary effluent	1.2–1.6
Secondary effluent	1.5–3.0

Source: Calculated using the coefficients presented by Fair et al (1973) and Arceivala (1981)

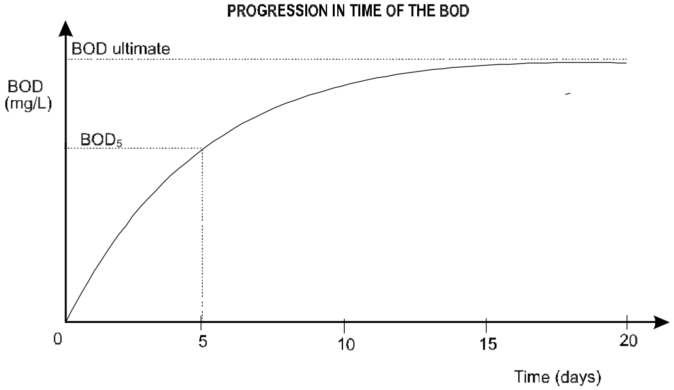


Figure 2.11. Progression in time of BOD in a sample, showing BOD_5 and BOD ultimate

After this, the oxygen consumption can be considered negligible. In this way the Ultimate Biochemical Oxygen Demand corresponds to the oxygen consumption until this time, after what there is no significant consumption, meaning that the organic matter has been practically all stabilised. Figure 2.11 shows the progression of BOD in time, in a sample analysed along various days.

For domestic sewage, it is considered, in practical terms, that after 20 days of the test the stabilisation is practically complete. Therefore the BOD_u can be determined at 20 days. The concept of the test is similar to the standard BOD of 5 days, varying only with the final period of determination of the dissolved oxygen concentration.

Table 2.16 presents typical ranges of the conversion factor for BOD_5 to BOD_u (domestic waste). Such a conversion is important, because various sewage treatment processes are designed using a BOD_u base. Chapter 3 shows how to proceed with this conversion using a specific formula.

Various authors adopt the ratio BOD_u/BOD_5 equal to **1.46**. This means that, in the case of having a BOD_5 of 300 mg/L, the BOD_u is assumed to be equal to $1.46 \times 300 = 438$ mg/L.

c) Chemical Oxygen Demand (COD)

The COD test measures the consumption of oxygen occurring as a result of the chemical oxidation of the organic matter. The value obtained is, therefore, an indirect indication of the level of organic matter present.

The main difference with the BOD test is clearly found in the nomenclature of both tests. The BOD relates itself with the *biochemical* oxidation of the organic matter, undertaken entirely by microorganisms. The COD corresponds to the *chemical* oxidation of the organic matter, obtained through a strong oxidant (potassium dichromate) in an acid medium.

The main **advantages** of the COD test are:

- the test takes only two to three hours;
- because of the quick response, the test can be used for operational control;
- the test results give an indication of the oxygen required for the stabilisation of the organic matter;
- the test allows establishment of stoichiometric relationships with oxygen;
- the test is not affected by nitrification, giving an indication of the oxidation of the carbonaceous organic matter only (and not of the nitrogenous oxygen demand).

The main **limitations** of the COD test are:

- in the COD test, both the biodegradable and the inert fractions of organic matter are oxidised. Therefore, the test may overestimate the oxygen to be consumed in the biological treatment of the wastewater;
- the test does not supply information about the consumption rate of the organic matter along the time;
- certain reduced inorganic constituents could be oxidised and interfere with the result.

For raw domestic sewage, the ratio COD/BOD_5 varies between 1.7 and 2.4. For industrial wastewater, however, this ratio can vary widely. Depending on the value of the ratio, conclusions can be drawn about the biodegradability of the wastewater and the treatment process to be employed (Braile & Cavalcanti, 1979):

- *Low COD/BOD_5 ratio* (less than 2.5 or 3.0):
  - the biodegradable fraction is high
  - good indication for biological treatment
- *Intermediate COD/BOD_5 ratio* (between 2.5 and 4.0):
  - the inert (non-biodegradable) fraction is not high
  - treatability studies to verify feasibility of biological treatment
- *High COD/BOD_5 ratio* (greater than 3.5 or 4.0):
  - the inert (non-biodegradable) fraction is high
  - possible indication for physical–chemical treatment

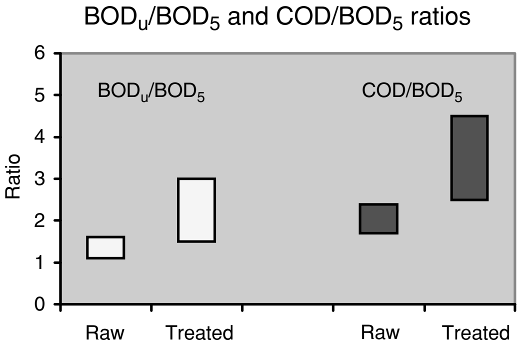


Figure 2.12. Ranges of values of the ratios BOD_u/BOD_5 and COD/BOD_5 for raw sewage and biologically treated sewage

The COD/BOD_5 ratio also varies as the wastewater passes along the various units of the treatment works. The tendency is for the ratio to increase, owing to the stepwise reduction of the biodegradable fraction, at the same time that the inert fraction remains approximately unchanged. In this way, the final effluent of the biological treatment has values of the COD/BOD_5 ratio usually higher than 3.0.

d) Total Organic Carbon (TOC)

In this test the organic carbon is **directly** measured, in an instrumental test, and not indirectly through the determination of the oxygen consumed, like the three tests above. The TOC test measures all the carbon released in the form of CO_2. To guarantee that the carbon being measured is really organic carbon, the inorganic forms of carbon (like CO_2, HCO_3^- etc) must be removed before the analysis or be corrected when calculated (Eckenfelder, 1980). The TOC test has been mostly used so far in research or in detailed evaluations of the characteristics of the liquid, due to the high costs of the equipment.

e) Relationship between the representative parameters of oxygen consumption

In samples of raw and treated domestic sewage, the usual ratios between the main representative parameters of oxygen consumption (BOD_u/BOD_5 and COD/BOD_5) are shown in Figure 2.12. The following comments can be made:

- The ratios can never be lower than 1.0.
- The ratios increase, from the condition of untreated to biologically treated wastewater.
- The higher the treatment efficiency, the higher the value of the ratio.

Table 2.18. Predominant forms of nitrogen in the water

Form	Formula	Oxidation state
Molecular nitrogen	N_2	0
Organic nitrogen	Variable	Variable
Free ammonia	NH_3	-3
Ammonium ion	NH_4^+	-3
Nitrite ion	NO_2^-	$+3$
Nitrate ion	NO_3^-	$+5$

2.2.3.4 Nitrogen

In its cycle in the biosphere, nitrogen alternates between various forms and oxidation states, resulting from various biochemical processes. In the aquatic medium, nitrogen can be found in the forms presented in Table 2.18.

Nitrogen is a component of great importance in terms of generation and control of the water pollution, principally for the following aspects:

- *Water pollution*
  - nitrogen is an essential nutrient for algae leading, under certain conditions, to the phenomenon of eutrophication of lakes and reservoirs;
  - nitrogen can lead to dissolved oxygen consumption in the receiving water body due to the processes of the conversion of ammonia to nitrite and this nitrite to nitrate;
  - nitrogen in the form of free ammonia is directly toxic to fish;
  - nitrogen in the form of nitrate is associated with illnesses such as methaemoglobinaemia
- *Sewage treatment*
  - nitrogen is an essential nutrient for the microorganisms responsible for sewage treatment;
  - nitrogen, in the processes of the conversion of ammonia to nitrite and nitrite to nitrate (*nitrification*), which can occur in a WWTP, leads to oxygen and alkalinity consumption;
  - nitrogen in the process of the conversion of nitrate to nitrogen gas (*denitrification*), which can take place in a WWTP, leads to (a) the economy of oxygen and alkalinity (when occurring in a controlled form) or (b) the deterioration in the settleability of the sludge (when not controlled).

The determination of the prevailing form of nitrogen in a water body can provide indications about the stage of pollution caused by an upstream discharge of sewage. If the pollution is recent, nitrogen is basically in the form of organic nitrogen or ammonia and, if not recent, in the form of nitrate (nitrite concentrations are normally low). In summary, the distinct forms can be seen in a generalised form presented in Table 2.19 (omitting other sources of nitrogen apart from sewage).

Table 2.19. Relative distribution of the forms of nitrogen under different conditions

Condition	Prevailing form of nitrogen
Raw wastewater	• Organic nitrogen • Ammonia
Recent pollution in a water course	• Organic nitrogen • Ammonia
Intermediate stage in the pollution of a water course	• Organic nitrogen • Ammonia • Nitrite (in lower concentrations) • Nitrate
Remote pollution in a water course	• Nitrate
Effluent from a treatment process without nitrification	• Ammonia
Effluent from a treatment process with nitrification	• Nitrate
Effluent from a treatment process with nitrification/denitrification	• Low concentrations of all forms of nitrogen

Note: organic nitrogen + ammonia = TKN (Total Kjeldahl Nitrogen)

In *raw domestic sewage*, the predominant forms are *organic nitrogen* and *ammonia*. Organic nitrogen corresponds to amina groups. Ammonia is mainly derived from urea, which is rapidly hydrolysed and rarely found in raw sewage. These two, together, are determined in the laboratory by the Kjeldahl method, leading to the *Total Kjeldahl Nitrogen* (**TKN**). Most of the TKN in domestic sewage has physiological origin. The other forms of nitrogen are usually of lesser importance in the influent to a WWTP. In summary:

- TKN = ammonia + organic nitrogen *(prevailing form in domestic sewage)*
- TN = TKN + NO_2^- + NO_3^- *(total nitrogen)*

The distribution of ammonia in the raw sewage can be represented schematically as shown in Figure 2.13. It is seen that the fraction of the oxidised nitrogen **NO_x** (nitrite + nitrate) is negligible in raw sewage. TKN can be further subdivided in a *soluble* fraction (dominated by ammonia) and a *particulate* fraction (associated with the organic suspended solids – nitrogen participates in the constitution of practically all forms of particulate organic matter in sewage).

Ammonia exists in solution in the form of the ion (NH_4^+) and in a free form, not ionised (NH_3), according to the following dynamic equilibrium:

$$\boxed{\begin{array}{ll} NH_3 + H^+ & \leftrightarrow NH_4^+ \\ \text{free ammonia} & \text{ionised ammonia} \end{array}} \qquad (2.6)$$

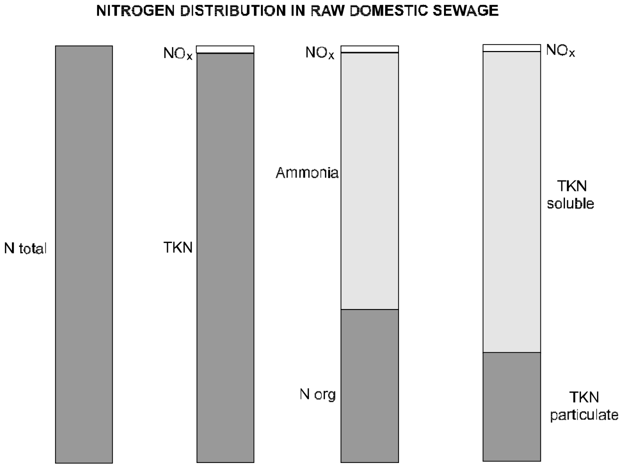


Figure 2.13. Distribution of nitrogen forms in untreated domestic sewage (adapted from IAWQ, 1995)

The relative distribution has the following values, as a function of the pH values.

Distribution between the forms of ammonia

- $pH < 8$ Practically all the ammonia is in the form of $NH_4{}^+$
- $pH = 9.5$ Approximately 50% NH_3 and 50% $NH_4{}^+$
- $pH > 11$ Practically all the ammonia in the form of NH_3

In this way it can be seen that, in the usual range of pH, near neutrality, the ammonia present is practically in the ionised form. This has important environmental consequences, because free ammonia is toxic to fish even in low concentrations. The temperature of the liquid also influences this distribution. At a temperature of 25 °C, the proportion of free ammonia relative to the total ammonia is approximately the double compared with a temperature of 15 °C.

The following equation allows the calculation of the proportion of free ammonia within total ammonia as a function of temperature and pH (Emerson et al, 1975):

$$\boxed{\frac{\text{Free NH}_3}{\text{Total ammonia}}(\%) = \left\{1 + 10^{0.09018+[2729.92/(T+273.20)]-p^H}\right\}^{-1} \times 100} \qquad (2.6)$$

where:

T = liquid temperature (°C)

Application of Equation 2.6 leads to the values of the ammonia distribution presented in Table 2.20 and illustrated in Figure 2.14.

Table 2.20. Proportion of free and ionised ammonia within total ammonia, as a function of pH and temperature

	T = 15 °C		T = 20 °C		T = 25 °C	
pH	% NH_3	% NH_4^+	% NH_3	% NH_4^+	% NH_3	% NH_4^+
6.50	0.09	99.91	0.13	99.87	0.18	99.82
7.00	0.27	99.73	0.40	99.60	0.57	99.43
7.50	0.86	99.14	1.24	98.76	1.77	98.23
8.00	2.67	97.33	3.82	96.18	5.38	94.62
8.50	7.97	92.03	11.16	88.84	15.25	84.75
9.00	21.50	78.50	28.43	71.57	36.27	63.73
9.50	46.41	53.59	55.68	44.32	64.28	35.72

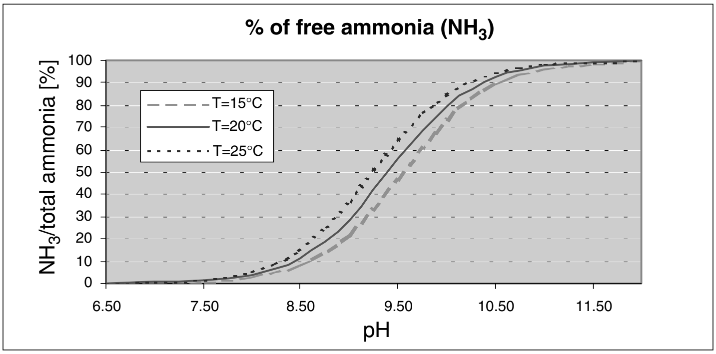


Figure 2.14. Percentage of free ammonia (NH_3) within total ammonia, as a function of pH and temperature

In a watercourse or in a WWTP, the ammonia can undergo subsequent transformations. In the process of **nitrification** the ammonia is oxidised to nitrite and the nitrite to nitrate. In the process of **denitrification** the nitrates are reduced to nitrogen gas. Chapter 35 details the biochemical processes involved and the implications in wastewater treatment.

2.2.3.5 *Phosphorus*

Total phosphorus in domestic sewage is present in the form of **phosphates**, according to the following distribution (IAWQ, 1995):

- **inorganic** (polyphosphates and orthophosphates) – main source from detergents and other household chemical products
- **organic** (bound to organic compounds) – physiological origin

Phosphorus in detergents is present, in raw sewage, in the form of soluble polyphosphates or, after hydrolysis, as orthophosphates. *Orthophosphates* are directly available for biological metabolism without requiring conversion to simpler

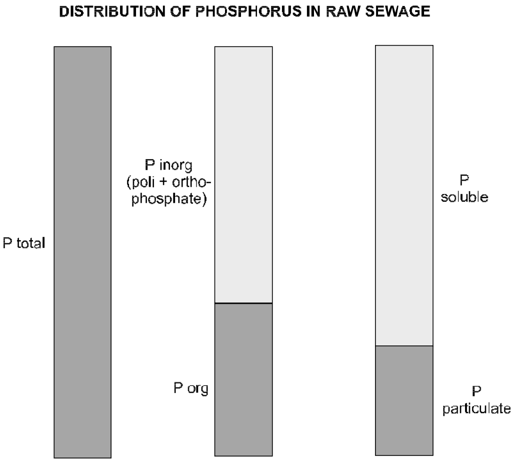


Figure 2.15. Distribution of phosphorus forms in untreated domestic sewage (IAWQ, 1995)

forms. The forms in which orthophosphates are present in the water are pH dependent, and include PO_4^{3-}, HPO_4^{2-}, $H_2PO_4^{-}$, H_3PO_4. In typical domestic sewage the prevailing form is HPO_4^{-2}. *Polyphosphates* are more complex molecules, with two or more phosphorus atoms. Polyphosphates are converted into orthophosphates by hydrolysis, which is a slow process, even though it takes place in the sewerage collection system itself. Mathematical models for wastewater treatment processes usually consider that both forms of phosphate are represented by orthophosphates since after hydrolysis they will effectively be present as such. Phosphorus in detergents can account for up to 50% of the total phosphorus present in domestic sewage.

Another way of fractionating phosphorus in wastewater is with respect to its form as solids (IAWQ, 1995):

- **soluble phosphorus** (predominantly inorganic) – mainly polyphosphates and orthophosphates (inorganic phosphorus), together with a small fraction corresponding to the phosphorus bound to the soluble organic matter in the wastewater
- **particulate phosphorus** (all organic) – bound to particulate organic matter in the wastewater

Figure 2.15 illustrates the fractionation of phosphorus in untreated domestic sewage.

The importance of phosphorus is associated with the following aspects:

- Phosphorus is an essential nutrient for the growth of the microorganisms responsible for the stabilisation of organic matter. Usually domestic sewage has sufficient levels of phosphorus, but a lack may occur in some industrial wastewaters;

- Phosphorus is an essential nutrient for the growth of algae, eventually leading, under certain conditions, to the eutrofication of lakes and reservoirs.

2.2.3.6 Pathogenic organisms and indicators of faecal contamination

a) Pathogenic organisms

The list of organisms of importance in water and wastewater quality was presented in Table 2.15. Most of these organisms play various essential roles, mainly related to the transformation of the constituents in the biogeochemical cycles. Biological wastewater treatment relies on these organisms, and this aspect is covered in many parts of this book.

Another important aspect in terms of the biological quality of a water or wastewater is that related to the disease transmission by pathogenic organisms. The major groups of pathogenic organisms are: (a) *bacteria*, (b) *viruses*, (c) *protozoans* and (d) *helminths*.

Water-related disease is defined as any significant or widespread adverse effects on human health, such as death, disability, illness or disorders, caused directly or indirectly by the condition, or changes in the quantity or quality of any water (Grabow, 2002). A useful way of classifying the water-related diseases is to group them according to the mechanism by which they are transmitted (*water borne, water hygiene, water based, water related*). Table 2.21 presents the main four categories, with a summary description and the main preventive strategies to be employed. Table 2.22 details the **faecal–oral transmission diseases** (*water borne and water hygiene*), with the main pathogenic agents and symptoms. Faecal–oral diseases are of special interest for the objectives and theme of this book, since they are associated with proper excreta and wastewater treatment and disposal.

The number of pathogens present in the sewage of a certain community varies substantially and depends on: (a) socio-economic status of the population; (b) health requirements; (c) geographic region; (d) presence of agroindustries; (e) type of treatment to which the sewage was submitted.

b) Indicator organisms

The detection of pathogenic organisms, mainly bacteria, protozoans and viruses, in a sample of water is difficult, because of their low concentrations. This would demand the examination of large volumes of the sample to detect the pathogenic organisms. The reasons are due to the following factors:

- in a population, only a certain fraction suffers from water-borne diseases;
- in the faeces of these inhabitants, the presence of pathogens may not occur in high proportions;

Table 2.21. Mechanisms of transmission of water-related infections

Mechanism	Description	Transmission mode	Main diseases	Preventive strategy
Water borne	• Ingestion of contaminated water	• Faecal–oral (pathogen in *water* is ingested by man or animal)	• Diarrhoeas and dysenteries (amoebic dysentery, balantidiasis, *Campylobacter* enteritis, cholera, *E. coli* diarrhoea, giardiasis, cryptosporidiosis, rotavirus diarrhoea, salmonellosis, bacillary dysentery, yersiniosis) • Enteric fevers (typhoid, paratyphoid) • Poliomyelitis • Hepatitis A • Leptospirosis • Ascariasis • Trichuriasis	• Protect and treat drinking water • Avoid use of contaminated water
Water hygiene	• Infections of the intestinal tract, due to low availability of water and poor hygiene	• Various faecal-oral routes (e.g. food, bad hygiene)	• Similar to above (water borne)	• Supply water in sufficient quantity • Promote personal, domestic and food hygiene
	• Infections of the skin or eyes, due to low availability of water and poor hygiene	• Lack of water and poor hygiene create conditions for their transmission • Non-faecal route (cannot be water borne)	• Infectious skin diseases (e.g. skin sepsis, scabies, fungal infections) • Infectious eye diseases (e.g. trachoma) • Others (e.g. louse-borne typhus)	

Water based	• Pathogen (helminth) spends part of its life cycle in a water snail or other aquatic animal	• Pathogen penetrates the skin or is ingested	• Schistosomiasis • Guinea worm • Clonorchiasis • Diphyllobothriasis • Fasciolopsiasis • Paragonimiasis • Others	• Avoid contact with contaminated water • Protect water sources from excreta • Adopt adequate solutions for excreta or wastewater disposal • Combat intermediate host
Water related	• Insects which breed in water or bite near water	• Insect bites man or animal	• Malaria • Sleeping sickness • Filariasis • River blindness • Mosquito-borne virus (e.g. yellow fever, dengue) • Others	• Combat insect vectors • Destroy breeding sites of insects • Avoid contact with breeding sites • Adopt individual protection (e.g. sprays, nets)

Source: Cairncross and Feachem (1990), Heller & Möller (1995), van Buuren et al (1995), Heller (1997)

Table 2.22. Main water-borne and water hygiene (faecal oral transmission) diseases, according to pathogenic organism

Organism	Disease	Causal agent	Symptoms / manifestation
Bacteria	Bacillary dysentery (shigellosis)	*Shigella dysenteriae*	Severe diarrhoea
	Campylobacter enteritis	*Campylobacter jejuni, Campylobacter coli*	Diarrhoea, abdominal pain, malaise, fever, nausea, vomiting
	Cholera	*Vibrio cholerae*	Extremely heavy diarrhoea, dehydration, high death rate
	Gastroenteritis	*Escherichia coli* – enteropathogenic	Diarrhoea
	Leptospirosis	*Leptospira* – various species	Jaundice, fever
	Paratyphoid fever	*Salmonella* – various species	Fever, diarrhoea, malaise, headache, spleen enlargement, involvement of lymphoid tissues and intestines
	Salmonella	*Salmonella* – various species	Fever, nausea, diarrhoea
	Typhoid fever	*Salmonella typhi*	High fever, diarrhoea, ulceration of small intestine
Protozoan	Amoebic dysentery	*Entamoeba histolytica*	Prolonged diarrhoea with bleeding, abscesses of the liver and small intestine
	Giardiasis	*Giardia lamblia*	Mild to severe diarrhoea, nausea, indigestion, flatulence
	Cryptosporidiosis	*Cryptosporidium*	Diarrhoea
	Balantidiasis	*Balantidium coli*	Diarrhoea, dysentery
Viruses	Infectious hepatitis	Hepatitis A virus	Jaundice, fever
	Respiratory disease	Adenovirus – various types	Respiratory illness
	Gastroenteritis	Enterovirus, Norwalk, rotavirus, etc. – various species	Mild to strong diarrhoea, vomiting
	Meningitis	Enterovirus	Fever, vomiting, neck stiffness
	Poliomyelitis (infantile paralysis)	*Poliomyelitis virus*	Paralysis, atrophy
Helminths	Ascariasis	*Ascaris lumbricoides*	Pulmonary manifestations, nutritional deficiency, obstruction of bowel or other organ
	Trichuriasis	*Trichuris trichiura*	Diarrhoea, bloody mucoid stools, rectal prolapse

Source: Benenson (1985), Tchobanoglous and Schroeder (1985), Metcalf & Eddy (1991)

- after discharge to the receiving body or sewerage system, there is still a high dilution of the contaminated waste;
- sensitivity and specificity of the tests for some pathogens;
- broad spectrum of pathogens.

In this sense, the final concentration of *pathogens* per unit volume in a water body may be considerably low, making detection through laboratory examination highly difficult.

This obstacle is overcome through the search for *indicator organisms of faecal contamination*. These organisms are predominantly non-pathogenic, but they give a satisfactory indication of whether the water is contaminated by human or animal faeces, and, therefore, of its potential to transmit diseases.

The organisms most commonly used with this objective are bacteria of the **coliform group**. The following are the main reasons for the use of the coliform group as indicators of faecal contamination:

- Coliforms are present in *large quantities in human faeces* (each individual excretes on average 10^{10} to 10^{11} cells per day) (Branco and Rocha, 1979). About 1/3 to 1/5 of the weight of human faeces consist of bacteria from the coliform group. All individuals eliminate coliforms, and not only those who are ill, as is the case with pathogenic organisms. Thus the probability that the coliforms will be detected after the sewage discharge is much higher than with pathogenic organisms.
- Coliforms present a *slightly higher resistance* in the water compared with the majority of enteric pathogenic bacteria. This characteristic is important, because they would not be good indicators of faecal contamination if they died faster than pathogenic organisms, and a sample without coliforms could still contain pathogens. On the other hand, if their mortality rate were much lower than that of pathogenic microorganisms, the coliforms would not be useful indicators, since their presence could unjustifiably make suspect a sample of purified water. These considerations apply mainly to pathogenic bacteria, since other microorganisms can present a higher resistance compared to coliforms.
- The *removal mechanisms* for coliforms from water bodies, water treatment plants and WWTP are the same mechanisms used for pathogenic bacteria. In this way the removal of pathogenic bacteria is usually associated with the removal of coliforms. Other pathogenic organisms (such as protozoan cysts and helminth eggs), however, can be removed by different mechanisms.
- The bacteriological techniques for coliform detection are *quick and economic* compared with those for pathogens.

The indicators of faecal contamination most commonly used are:

- *total coliforms (TC)*
- *faecal coliforms (FC)* or *thermotolerant coliforms*
- *Escherichia coli (EC)*

The group of **total coliforms** (**TC**) constitutes a large group of bacteria that have been isolated in water samples and in polluted and non polluted soils and plants, as well as from faeces from humans and other warm-blooded animals. This group was largely used in the past as an indicator, and continues to be used in some areas, although the difficulties associated with the occurrence of non-faecal bacteria are a problem (Thoman and Mueller, 1987). There is no quantifiable relation between TC and pathogenic microorganisms. The total coliforms could be understood in a simplified way as "environmental" coliforms, given their possible occurrence in non-contaminated water and soils, thus representing other free-living organisms, and not only the intestinal ones. For this reason, total coliforms should not be used as indicators of faecal contamination in surface waters. However, in the specific case of potable water supply, it is expected that treated water should not contain total coliforms. These, if found, could suggest inadequate treatment, post contamination or excess of nutrients in the treated water. Under these conditions, total coliforms could be used as indicators of the water treatment efficiency and of the integrity of the water distribution system (WHO, 1993).

Faecal coliforms (**FC**) are a group of bacteria predominantly originated from the intestinal tract of humans and other animals. This group encompasses the genus *Escherichia* and, to a lesser degree, species of *Klebsiella*, *Enterobacter* and *Citrobacter* (WHO, 1993). The test for FC is completed at a high temperature, aiming at suppressing bacteria of non-faecal origin (Thoman and Mueller, 1987). However, even under these conditions, the presence of non-faecal (free-living) bacteria is possible, although in lower numbers compared with the total coliforms test. As a result, even the test for faecal coliforms does not guarantee that the contamination is really faecal. For this reason, recently the faecal coliforms have been preferably denominated **thermotolerant coliforms**, because of the fact that they are resistant to the high temperatures of the test, but are not necessarily faecal. *Whenever in the present book reference is made to faecal coliforms (traditional in the literature and in the environmental legislation in various countries), it should be understood, implicitly, the more appropriate terminology of thermotolerant coliforms*.

Escherichia coli (**EC**) is the main bacterium of the faecal (thermotolerant) coliform group, being present in large numbers in the faeces from humans and animals. It is found in wastewater, treated effluents and natural waters and soils that are subject to recent contamination, whether from humans, agriculture, wild animals and birds (WHO, 1993). Its laboratory detection is very simple, principally by recent fluorogenic methods. Different from total and faecal coliforms, *E. coli* is the only that gives **guarantee of exclusively faecal contamination**. For this reason, there is a current tendency in using predominantly *E. coli* as indicator of faecal contamination. However, its detection **does not guarantee that the contamination is from human origin**, since *E. coli* can also be found in other animal faeces. There are some types of *E. coli* that are pathogenic, but this does not invalidate its concept as bacterial indicators of faecal contamination.

The detection of faecal contamination, **exclusively human**, requires the use of complementary biochemical tests, which are not usually undertaken in routine analysis.

Table 2.23. Application of total coliforms, thermotolerant coliforms and *E. coli* as indicators of faecal contamination

Item	Sample	Total coliform	Faecal (thermotolerant) coliforms	*E. coli*
Guarantee that the contamination is of faecal origin	Water bodies reasonably clean Water bodies polluted by sewage	Low Reasonable	Reasonable High	Total Total
Guarantee that the faecal contamination is exclusively human	Water bodies reasonably clean Water bodies polluted mainly by domestic sewage	None Reasonable	None High	None High
Proportion of *E. coli* in the total count of coliforms	Water bodies reasonably clean Water bodies polluted by domestic sewage Domestic sewage	Variable Reasonable to high Very high	Variable High Very high	– – –

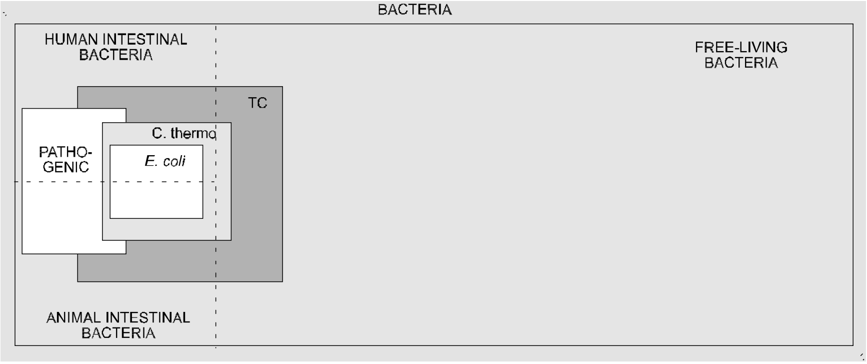


Figure 2.16. Schematic representation of bacteria and indicators of faecal contamination

Figure 2.16 illustrates the relative distribution of the indicator, pathogenic and other forms of bacteria. Table 2.23 synthesises the application of the three groups of indicators discussed above.

In **sewage**, *E. coli* is the predominant organism within the group of faecal (thermotolerant) coliforms, and the faecal (thermotolerant) coliforms are the predominant group within the total coliforms. For **water bodies**, when doing the interpretation of the tests for indicators of faecal contamination, it is very important to carry out a sanitary survey of the catchment area. This survey helps in establishing the origin of the faecal contamination (presence of domestic sewage discharges or wastes from animals), complementing the information supplied by the laboratory tests.

For the objectives of this book (wastewater treatment), the characterisation of the *faecal origin* is not so important, since it is already accepted that the wastewater will contain faecal matter and organisms. The indicator organisms are used, in this case, as *indicators of the pathogen removal **efficiency** in the wastewater treatment process*. The pathogenic organisms that can be represented are *bacteria* and *viruses*, since they are removed by the same mechanisms of the coliform bacteria. Protozoan cysts and helminth eggs, which are mainly removed by physical mechanisms, such as sedimentation and filtration, are not well represented by coliform bacteria as indicators of treatment efficiency.

There are various other indicator organisms proposed in the literature, each with its own advantages, disadvantages and applicability. Below some of these organisms are briefly discussed.

Faecal streptococci. The group of faecal streptococci comprises two main genera: *Enterococcus* and *Streptococcus.* The genus ***Enterococcus*** encompasses many species, the majority of them of faecal human origin; however, some species are from animal origin. All *Enterococcus* present high tolerance to adverse environmental conditions. The genus ***Streptococcus*** comprises the species *S. bovis* and *S. equinus*, which are abundant in animal faeces. Faecal streptococci seldom multiply in polluted waters, and are more resistant than *E. coli* and coliform bacteria (WHO, 1993). Because of these characteristics, they have been used as indicators for *bathing waters*. In the past, the ratio between the values of faecal coliforms and faecal streptococci (FC/FS ratio) was used to give an indication of the origin of the contamination, whether predominantly human or animal. High values of FC/FS would suggest predominantly human contamination, whereas low values of FC/FS would suggest predominantly animal contamination. More recent evidences indicate, however, that these relations are not applicable in a large number of situations, giving unreliable indications about the real origin or the contamination in various catchment areas.

Sulphite-reducing clostridia. *Clostridium perfringens* is the most representative species in this group, being normally present in faeces, although in much smaller numbers than *E. coli*. However, it is not exclusively of faecal origin and can be derived from other environmental sources. Clostridial spores can survive in water much longer than organisms of the coliform group and will also resist disinfection. Their presence in disinfected waters may indicate deficiencies in treatment and that disinfectant-resistant pathogens could have survived treatment. Because of its longevity, it is best regarded as indicating intermittent or remote contamination. However, false alarms may also result from its detection, which makes it of special value, but not particularly recommended for routine monitoring of water distribution systems (WHO, 1993).

Bacteriophages. For the indication of the presence of viruses, bacteriophages may be representative, owing to their similarities with the enteric human viruses. Bacteriophages are specific viruses that infect bacteria, for example the coliphages, which infect *E. coli*. Coliphages are not present in high numbers in fresh human or animal faeces, but may be abundant in sewage, owing to their fast reproduction rate resulting from the attack to bacterial cells (Mendonça, 2000). Their significance is as indicators of sewage contamination and, because of their greater persistence compared with bacterial indicators, as additional indicators of treatment efficiency or for groundwater protection.

Helminth eggs. For helminths, there are no substituting indicators, and helminth eggs are determined directly in laboratory tests. However, the eggs of nematodes, such as *Ascaris*, *Trichuris*, *Necator americanus* and *Ancilostoma duodenale* may be used as indicators of other helminths (cestodes, trematodes and other nematodes), which are removed in water and wastewater treatment by the same mechanism (e.g. sedimentation), being thus indicators of treatment efficiency. Helminth eggs are an important parameter when assessing the use of water or treated wastewater for irrigation, in which workers may have direct contact with contaminated water and consumers may eat the irrigated vegetable uncooked or unpeeled. Helminth eggs may be removed by physical operations, such as sedimentation, which takes place, for instance, in stabilisation ponds. Eggs may be viable or non-viable, and viability may be altered by specific disinfection processes.

This topic is under constant development, and the present text does not aim to go deeper into specific items, covering only the more general and simplified concepts.

2.2.4 Relationship between load and concentration

Before presenting the typical concentrations of the main pollutants in sewage, it is important to be clear about the concepts of per capita, load and constituent concentration.

Per capita load represents the average contribution of each individual (expressed in terms of pollutant mass) per unit time. A commonly used unit is grams per inhabitant per day (g/inhab.d). For example, when the BOD contribution is 54 g/inhab.d, it is equivalent to saying that every individual discharges 54 grams of BOD on average, per day.

The influent **load** to a WWTP corresponds to the quantity of pollutant (mass) per unit time. In this way, import relations are

$$\boxed{\textbf{load} = \textbf{population} \times \textbf{per capita load}} \tag{2.7}$$

$$\text{load (kg/d)} = \frac{\text{population (inhab)} \times \text{per capita load (g/inhab.d)}}{1000\,\text{(g/kg)}} \tag{2.8}$$

or

$$\boxed{\textbf{load} = \textbf{concentration} \times \textbf{flow}} \tag{2.9}$$

$$\text{load (kg/d)} = \frac{\text{concentration (g/m}^3) \times \text{flow (m}^3\text{/d)}}{1000\ \text{(g/kg)}} \tag{2.10}$$

Note: $g/m^3 = mg/L$

The **concentration** of a wastewater can be obtained through the rearrangement of the same dimensional relations:

$$\boxed{\textbf{concentration} = \textbf{load/flow}} \tag{2.11}$$

$$\text{concentration (g/m}^3) = \frac{\text{load (kg/d)} \times 1000 \text{ (g/kg)}}{\text{flow (m}^3\text{/d)}} \tag{2.12}$$

Example 2.2

Calculate the total nitrogen load in the influent to a WWTP, given that:

- concentration = 45 mgN/L
- flow = 50 L/s

Solution:

Expressing flow in m^3/d, :

$$Q = \frac{50 \text{ L/s} \times 86400 \text{ s/d}}{1000 \text{ L/m}^3}$$

The nitrogen load is:

$$\text{load} = \frac{45 \text{ g/m}^3 \times 4320 \text{ m}^3\text{/d}}{1000 \text{ g/kg}} = 194 \text{ kgN/d}$$

b) In the same works, calculate the total phosphorus concentration in the influent, given that the influent load is 40 kgP/d.

$$\text{concentration} = \frac{40 \text{ kg/d} \times 1000 \text{ g/kg}}{4320 \text{ m}^3\text{/d}} = 9.3 \text{ gP/m}^3 = 9.3 \text{ mgP/L}$$

2.2.5 Characteristics of domestic sewage

The typical quantitative physical–chemical characteristics of predominantly domestic sewage in developing countries can be found in a summarised form in Table 2.24.

Campos and von Sperling (1996) verified, for essentially domestic sewage in nine sub-catchment areas in the city of Belo Horizonte, Brazil, relationships between per capita BOD load and BOD concentration with the average family income.

Table 2.24. Physical–chemical characteristics of raw domestic sewage in developing countries

	Per capita load (g/inhab.d)		Concentration (mg/L, except pH)	
Parameter	Range	Typical	Range	Typical
TOTAL SOLIDS	120–220	180	700–1350	1100
Suspended	35–70	60	200–450	350
• *Fixed*	7–14	10	40–100	80
• *Volatile*	25–60	50	165–350	320
Dissolved	85–150	120	500–900	700
• *Fixed*	50–90	70	300–550	400
• *Volatile*	35–60	50	200–350	300
Settleable	–	–	10–20	15
ORGANIC MATTER				
BOD_5	40–60	50	250–400	300
COD	80–120	100	450–800	600
BOD ultimate	60–90	75	350–600	450
TOTAL NITROGEN	6.0–10.0	8.0	35–60	45
Organic nitrogen	2.5–4.0	3.5	15–25	20
Ammonia	3.5–6.0	4.5	20–35	25
Nitrite	≈ 0	≈ 0	≈ 0	≈ 0
Nitrate	0.0–0.3	≈ 0	0–2	≈ 0
PHOSPHORUS	0.7–2.5	1.0	4–15	7
Organic phosphorus	0.7–1.0	0.3	1–6	2
Inorganic phosphorus	0.5–1.5	0.7	3–9	5
pH	-	-	6.7–8.0	7.0
ALKALINITY	20–40	30	100–250	200
HEAVY METALS	≈ 0	≈ 0	≈ 0	≈ 0
TOXIC ORGANICS	≈ 0	≈ 0	≈ 0	≈ 0

Sources: Arceivala (1981), Jordão & Pessoa (1995), Qasim (1985), Metcalf & Eddy (1991), Cavalcanti et al (2001) and the author's experience.

The higher the income, the higher is the per capita BOD load and the lower is the BOD concentration (Figure 2.17). Family income is expressed as numbers of minimum salaries. The figures are presented in order to show the large influence of economic status, and not to allow direct calculations, since the economical data are region specific.

The typical biological characteristics of domestic sewage, in terms of pathogenic organisms, can be found in Table 2.25.

2.2.6 Characteristics of industrial wastewater

2.2.6.1 General concepts

The generalisation of typical industrial wastewater characteristics is difficult because of their wide variability from time to time and from industry to industry.

Table 2.25. Microorganisms present in raw domestic sewage in developing countries

Microorganisms	Per capita load (org/inhab.d)	Concentration (org/100 ml)
Total coliforms	10^{10}–10^{13}	10^{7}–10^{10}
Faecal (thermotolerant) coliforms	10^{9}–10^{12}	10^{6}–10^{9}
E. coli	10^{9}–10^{12}	10^{6}–10^{9}
Faecal streptococci	10^{7}–10^{10}	10^{4}–10^{7}
Protozoan cysts	$<10^{7}$	$<10^{4}$
Helminth eggs	10^{3}–10^{6}	10^{0}–10^{3}
Viruses	10^{5}–10^{7}	10^{2}–10^{4}

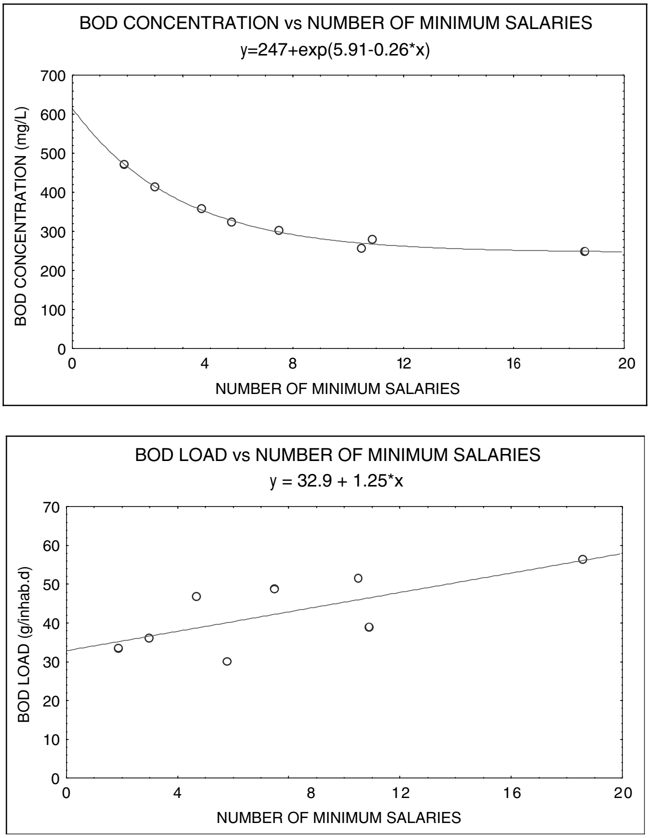


Figure 2.17. BOD concentration and per capita BOD load as a function of family income in domestic sewage from nine catchment areas in Belo Horizonte, Brazil (family income as number of minimum salaries; 1 minimum salary = US$80 at the time of the research)

The following concepts are important in terms of the biological treatment of industrial wastewater:

- **Biodegradability**: capacity of the wastewater to be stabilised through biochemical processes by microorganisms.
- **Treatability**: suitability of the waste to be treated by conventional or existing biological processes.
- **Biodegradable organic matter concentration**: BOD of the wastewater, which can be: (a) higher than in domestic sewage (predominantly biodegradable organic wastewater, treatable through biological processes), or (b) lower than in domestic sewage (predominately inorganic or unbiodegradable wastewater, in which there is less need for BOD removal, but in which the pollutional load can be expressed in terms of other quality parameters)
- **Nutrient availability**: biological wastewater treatment requires a balanced equilibrium between the nutrients C:N:P. This equilibrium is usually found in domestic sewage.
- **Toxicity**: certain industrial wastewaters have toxic or inhibitory constituents that can affect or render biological treatment unfeasible.

Figure 2.18 presents the main options for the treatment and discharge of industrial effluents.

The integration of industrial wastewater with domestic sewage in the public sewerage system, for subsequent combined treatment in a WWTP, may be an interesting alternative. Possible reasons for this alternative would be economy of scale, dilution of undesirable constituents, revenue for the sanitation company for transporting and treating the industrial wastewater, simplification for the industries. However, for this practice to be effective, it is necessary that *previous removal from the industrial effluent* is practised for the constituents that may pose one or more of the following problems:

- Safety risks and problems in the operation of the sewerage collection and interception system.
- Toxicity to the biological treatment.
- Toxicity to the sludge treatment and its final disposal.
- Persistence of contaminants in the effluent of the biological treatment, because of the fact that they have not been removed by the treatment.

The water and sanitation company, to receive the industrial wastewaters, must have specific standards for the discharge of industrial effluents into the public sewerage system.

If a pollutant leads to one of the above problems, the industry must pre-treat the wastewater, in order to place the effluent within the standards of the Sanitation Company for discharge into the public sewerage system.

The industry can opt for complete treatment and discharge the industrial effluent directly into the receiving water body. In this case, the effluents must comply with

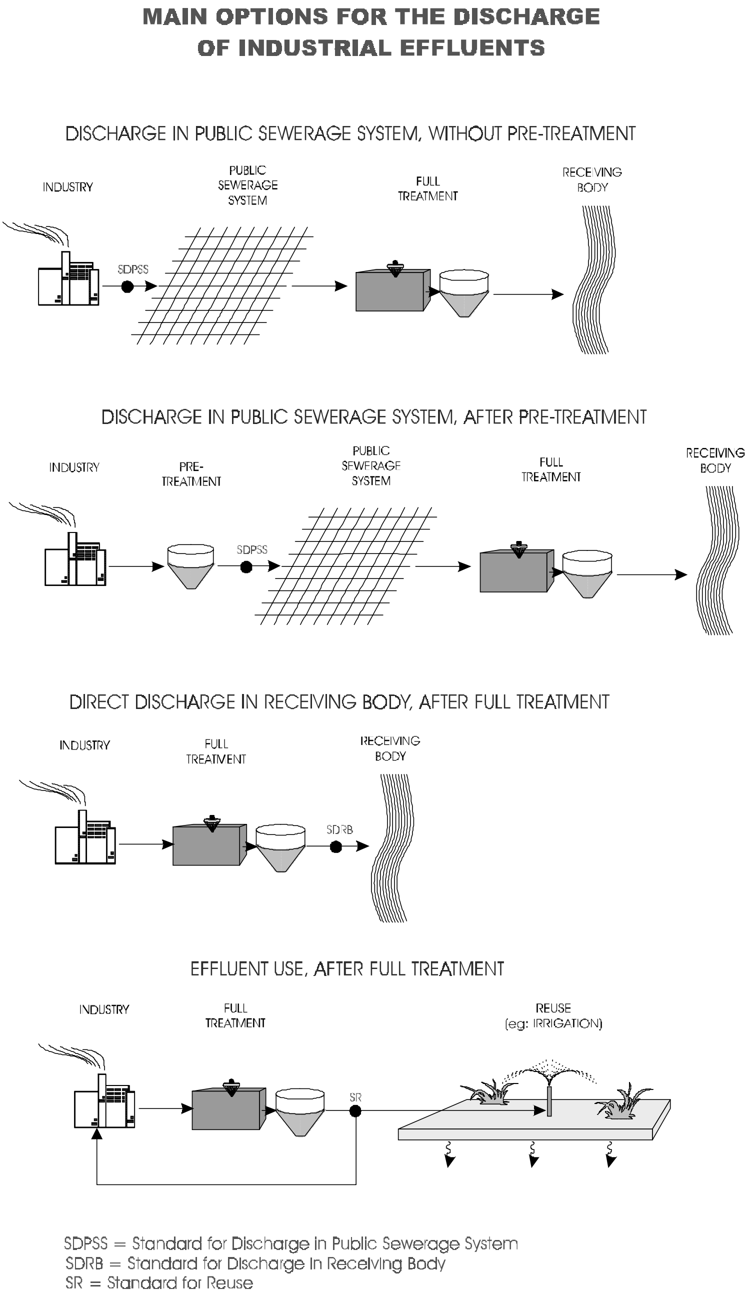


Figure 2.18. Main alternatives for the treatment and discharge of industrial effluents

the legislation established by the Environmental Agency for discharge to receiving bodies.

Another option is for the industry to completely treat the effluent and use it for other purposes (example: irrigation), or recycle it as process water along the production line. Naturally, public health implications need to be well addressed and guidelines or standards for reuse need to be satisfied.

2.2.6.2 Pollutants of importance in industrial wastewaters

Industrial effluents, depending on the type of the industrial process, can contain in greater or lesser degrees, the various pollutants described in Section 2.2.3, which are present in domestic sewage (suspended solids, biodegradable organic matter, nitrogen, phosphorus and pathogenic organisms). The present section covers other pollutants, which are not usually found in typical domestic sewage, but which can be of concern in industrial or municipal wastewaters containing a fraction of industrial effluents. The text is based on da Silva et al (2001).

a) Metals

In the present context, the main implications of metals are:

- Toxicity to human beings and other forms of plant or animal life, as a result of the discharge or disposal of wastewaters to receiving water bodies or land.
- Inhibition to the microorganisms responsible for the biological treatment of wastewater.

In spite of being widely used, the expression “heavy metal” does not have a sole definition, varying from branch to branch of science. From the environmental point of view of this book, heavy metals can be understood as those that, under certain concentrations and exposure time, offer risks to human health and the environment, impairing the activity of living organisms, including those responsible for the biological treatment of wastewater.

The main chemical elements that fit into this category are: Ag, As, Cd, Co, Cr, Cu, Hg, Ni, Pb, Sb, Se and Zn. These elements may be naturally found in soils or waters in variable concentrations, but lower than those ones considered toxic to different living organisms. Among these, As, Co, Cr, Cu, Se and Zn are essential to organisms in certain small quantities, while others have no function in biological metabolism, being toxic to plants and animals.

Most living organisms need only few metals, and in very small doses, characterising the concept of micronutrients, as zinc, magnesium, cobalt and iron. These metals become toxic and dangerous to human health when they exceed certain concentration thresholds. As for lead, mercury and cadmium, these are metals that do not exist naturally in any organism. They do not perform any nutritional or biochemical function in microorganisms, plants or animals. That is, the presence of these metals in living organisms is harmful at any concentration.

Table 2.26. Summary of the sources of contamination and the effects on human health by metals most frequently found in environment

Metal	Sources of contamination	Effects on health
Cadmium	Refined flours, cigarettes, odontological materials, steel industry, industrial gaseous effluents, fertilisers, pesticides, fungicides, coffee and tea treated with agrotoxics, ceramics, seafood, bone meal, welding, casting and refining of metals such as zinc, lead and copper. Cadmium derivatives are used in pigments and paintings, batteries, electroplating processes, accumulators, PVC stabilisers, nuclear reactors.	Carcinogenic, causes blood pressure rise and heart swelling. Immunity decreases. Prostate growth. Bone weakening. Joint pains. Anaemia. Pulmonary emphysema. Osteoporosis. Smell loss. Decrease in sexual performance.
Lead	Car batteries, paints, fuels, plants treated with agrotoxics, bovine liver, canned foods, cigarettes, pesticides, hair paint, lead-containing gas, newsprint and colour advertisements, fertilisers, cosmetics, air pollution.	Irritability and aggressiveness, indisposition, migraines, convulsions, fatigue, gum bleed, abdominal pains, nausea, muscular weakness, loss of memory, sleeplessness, nightmares, unspecific vascular cerebral accident, alterations of intelligence, osteoporosis, kidney illnesses, anaemia, coagulation problems. It affects the digestive and reproductive system and is a teratogenic agent (causes genetic mutation).
Mercury	Thermometers, pesticides and agrotoxics, dental alloy, water, mining, polishers, waxes, jewels, paints, sugar, contaminated tomato and fish, explosives, mercury fluorescent lamps, cosmetic products, production and delivery of petroleum by-products, salt electrolysis cells for chlorine production.	Depressive illness, fatigue, tremors, panic syndrome, paresthesias, lack of motor control, side walking, speech difficulties, loss of memory, loss of sexual performance, stomatitis, loose teeth, pain and paralysis in the edges, headache, anorexia in children, hallucination, vomiting, mastication difficulties, sweating, and pain sense loss.
Nickel	Kitchenware, nickel–cadmium batteries, jewels, cosmetics, hydrogenated oils, pottery works, cold permanent wave, welding.	Carcinogenic, may cause: contact dermatitis, gingivitis, skin rash, stomatitis, dizziness, joint pains, osteoporosis and chronic fatigue.
Zinc	Metallurgy (casting and refining), lead recycling industries.	Sense of sweetish taste and dryness in the throat, cough, weakness, panalgia, shivering, fever, nausea, vomiting.
Chromium	Leather tanning, electroplating.	Dermatitis, cutaneous ulcers, nose inflammation, lung cancer and perforation in the nose septum.

Table 2.26 (*Continued*)

Metal	Sources of contamination	Effects on health
Arsenic	Fuel oil, pesticides and herbicides, metallurgy, sea plants and animals.	Gastrointestinal disturbances, muscular and visceral spasms, nausea, diarrhoea, inflammation of mouth and throat, abdominal pains.
Aluminium	Water, processed cheese, white wheat flour, aluminium kitchenware, cosmetics, anti-acids, pesticides and antiperspirant, baker's yeast, salt.	Intestinal constipation, loss of energy, abdominal colics, infantile hyperactivity, loss of memory, learning difficulties, osteoporosis, rickets and convulsions. Related diseases: Alzheimer's and Parkinson's.
Barium	Polluted water, agrotoxics, pesticides and fertilisers.	Arterial hypertension, cardiovascular diseases, fatigue and discouragement.

Sources: http://www.rossetti.eti.br; http://www.greenpeace.org.br

In human beings, metals can produce several effects, resulting from their action on molecules, cells, tissues, organs and even the whole system. Besides, the presence of a metal might restrict the absorption of other nutrients essential to the activity of the organism. Metals, because they cannot be metabolised, remain in the organism and carry out their toxic effects, combining with one or more reactive groups, which may be indispensable for normal physiological functions. Depending on the material involved and on the intensity of the intoxication, the effect may range from a topic skin manifestation, pulmonary membrane or digestive tract, to mutagenic, teratogenic or carcinogenic effects, and even death. It is important to emphasise that synergistic effects also need to be taken into account. In most cases, synergistic effects might be far greater than the mere sum of the individual effects.

Although in general metals may be poisonous to plants and animals under the low concentrations in which they may be present in domestic wastewater, no chronic toxicity problems associated with their disposal have been reported. On the other hand, the same could not be said for industrial wastewaters and the resulting sludges (in which metals are concentrated).

Table 2.26 summarises the main sources of contamination from some metals, together with their effects on human health.

In wastewater treatment, limitations associated with metals are mainly related to the inhibition of, or toxicity to, microorganism growth and the incorporation of metals in the sludge. For a certain metal, the maximum allowable load needs to be determined, such that there are no problems with microorganism inhibition, deterioration of effluent quality and impairment to agricultural use of the sludge.

The discharge of a particular industrial wastewater into the public sewerage system will have a variable impact in the WWTP, depending on the dilution factor, the content and type of pollutant, and the treatment process employed. To analyse the impact, it is interesting to perform simulations and to apply a safety factor to the calculated limits. In this way, decisions may be made regarding the acceptance of the effluents into the system, and finally at the WWTP. If the estimated loads are lower than the acceptable limits, the discharge may be accepted. Conversely, if the limits are exceeded, pre-treatment may be required, or no further admissions to the public systems may be accepted. A check must be made on the system to verify whether the biological process is being inhibited, or whether the treated effluent and the sludge to be reused are outside the limits established by the environmental agency. The control must be centred on the industrial discharges, since domestic sewage may not be prevented to be discharged to the public network system.

b) Toxic and dangerous organic compounds

Like the section on metals, this text is also based on da Silva et al (2001). Toxic and dangerous organic compounds, even though they usually *do not represent a concern in domestic sewage*, may be of concern in municipal wastewaters that receive industrial effluents.

When wastewaters containing toxic organic compounds are disposed of in the receiving water body without adequate treatment, severe damage may occur, both to the aquatic life and to human beings, who use it as a source of water supply. Most of these compounds are very slowly biodegraded, persisting in the environment for a long period. These compounds are able to penetrate the food chain and, even if they are not detectable in the receiving body, they may be present in large quantities in the higher trophic levels, owing to their bioaccumulation characteristics. Another important fact is that, although some compounds do not pose serious health damages when ingested, their metabolites may be more toxic than the original products. Besides, since wastewaters have a complex composition and normally contain more than one organic pollutant, synergistic effects may take place (the combined effect may be higher than the sum of the individually exerted effects).

Several dangerous pollutants are volatile because of their low solubility, low molecular weight and high vapour pressure. Therefore, they may be transferred to the atmosphere in open units in the WWTP, such as aeration tanks, equalisation tanks and clarifiers, and also pumping stations. If adequate control means are not taken, their volatilisation represents a potential health risk to the population and workers who are frequently exposed to it. The structural integrity of the sewerage collection system is also affected, because many compounds are corrosive, inflammable and explosive (methanol, methyl-ethylketone, hexane, benzene, among others).

Other pollutants are adsorbed and concentrated in the biological flocs in the treatment process, and might cause inhibition to sludge digestion or generate sludge with dangerous characteristics which, if not adequately disposed of, could contaminate groundwater.

In some cases, the toxic pollutants are present in such low concentrations, that are not able to inhibit the biological process, but also are very hard to be removed.

Consequently, the treatment plant effluent may still contain these pollutants and, when discharged into the receiving body, may cause damages to the aquatic life and human beings.

There are relatively few data on the behaviour of these dangerous pollutants in WWTPs. The lack of knowledge of their physical, chemical and biochemical characteristics, as well as their inter-relationships in complex wastewaters, makes it extremely difficult to predict their treatability and destination during the treatment processes. More research is required for the identification of many compounds, understanding of their removal mechanisms and development of predictive models.

The main sources of organic compounds are: chemical and plastic industries, mechanical products, pharmaceutical industries, pesticide formulation, casthouses and steel industries, oil industry, laundries and lumber industries.

The most commonly found organic pollutants in industrial effluents are: phenol, methyl chloride, 1,1,1-trichloroethane, toluene, ethyl benzene, trichloroethylene, tetrachloroethylene, chloroform, bis-2-ethyl-hexyl phthalate, 2,4-dimethyl phenol, naphthalene, butylbenzylphthalate, acrolein, xylene, cresol, acetophenone, methyl-sobutyl-acetone, diphenylamine, aniline and ethyl acetate.

2.2.6.3 Population equivalent

Population equivalent (PE) is an important parameter for characterising industrial wastewaters. PE reflects the equivalence between the polluting potential of an industry (commonly in terms of biodegradable organic matter) and a certain population, which produces the same polluting load. For instance, when an industry is said to have a population equivalent of 20,000 habitants, it is the equivalent to saying that the BOD load of the industrial effluent corresponds to the load generated by a community with a population of 20,000 inhabitants. The formula for the calculation of population equivalent based on BOD is:

$$\boxed{\text{PE (population equivalent)} = \frac{\text{BOD load from industry (kg/d)}}{\text{per capita BOD load (kg/inhab.d)}}} \qquad (2.13)$$

In the case of adopting the value frequently used in the international literature for the per capita BOD load of 54 gBOD/inhab.d, PE may be calculated by:

$$\text{PE (population equivalent)} = \frac{\text{BOD load from industry (kg/d)}}{0.054\ \text{(kg/inhab.d)}} \qquad (2.14)$$

When reporting a value of population equivalent, it is important to make clear the per capita load used as a reference (54 gBOD/inhab.d or other value, more applicable to the region under analysis).

Example 2.3

Calculate the Population Equivalent (PE) of an industry that has the following data:

- flow = 120 m^3/d
- BOD concentration = 2000 mg/L

Solution:

The BOD load is:

$$\text{load} = \text{flow} \times \text{concentration} = \frac{120\,\text{m}^3/\text{d} \times 2000\ \text{g/m}^3}{1000\,\text{g/kg}} = 240\ \text{kgBOD/d}$$

The Population Equivalent is:

$$\text{PE} = \frac{\text{load}}{\text{per capita load}} = \frac{240\ \text{kg/d}}{0.054\ \text{kg/hab.d}} = 4{,}444\ \text{inhab}$$

Thus, the wastewater from this industry has a polluting potential (in terms of BOD) equivalent to a population of 4,444 inhabitants.

2.2.6.4 Characteristics of industrial wastewater

The characteristics of industrial wastewater vary essentially with the type of industry and with the type of industrial process used. Table 2.27 presents the main parameters that should be investigated for the characterisation of the effluents, as a function of the industry type. This table is only a general and initial guide, since there is always the possibility that the effluent from a certain industry has a parameter of importance not listed, or that a certain parameter in the table is not relevant to the industry in consideration.

The present book addresses mainly the treatment of predominantly domestic sewage. In this way, the main parameter of interest is the organic matter, represented by the BOD. Table 2.28 presents general information about the organic pollution generated by certain industries, including the population equivalent and the BOD loads per unit produced. Example 2.4 illustrates the use of the table for the estimation of the BOD in the industrial wastewater entering a WWTP.

Example 2.4

A slaughterhouse processes 30 heads of cattle and 50 pigs per day. Estimate the characteristics of the effluent.

Solution:

Using the table of industrial wastewater characteristics (Table 2.28), and adopting an average value of 3.0 kgBOD/cattle slaughtered (1 cow ≈ 2.5 pigs):

***Example 2.4* (*Continued*)**

a) BOD load produced

$$
\begin{aligned}
\text{–cows:}\quad & \frac{3\ \text{kgBOD}}{\text{cow}}.\frac{30\ \text{cow}}{\text{d}} = 90\ \text{kgDBO/d}\\
\text{–pigs:}\quad & \frac{3\ \text{kgDBO}}{2.5\ \text{pigs}}.\frac{50\ \text{pigs}}{\text{d}} = 60\ \text{kgDBO/d}\\
\text{–total:}\quad & 90 + 60 = 150\ \text{kgBOD/d}
\end{aligned}
$$

b) Population Equivalent (PE)

$$
\text{PE} = \frac{\text{BODload}}{\text{per capita BODload}} = \frac{150\,\text{kgDBO/d}}{0.054\,\text{kgDBO/inhab.d}} = 2.77\ \text{inhab}
$$

c) Wastewater flow

Using Table 2.28, and adopting an average value of 2.0 m^3/cattle slaughtered (or for 2.5 pigs slaughtered):

$$
\begin{aligned}
-\text{cows:}\quad & \frac{2.0\ \text{m}^3}{\text{cow}}.\frac{30\ \text{cow}}{\text{d}} = 60\ \text{m}^3/\text{d}\\
-\text{pigs:}\quad & \frac{2.0\ \text{m}^3}{2.5\ \text{pigs}}.\frac{50\ \text{pigs}}{\text{d}} = 40\ \text{m}^3/\text{d}\\
-\text{total:}\quad & 60 + 40 = 100\ \text{m}^3/\text{d}
\end{aligned}
$$

d) BOD concentration in the wastewater

$$
\begin{aligned}
\text{concentration} = \frac{\text{load}}{\text{flow}} = \frac{150\ \text{kg/d BOD}}{100\ \text{m}^3/\text{d}}.1000\ \text{g/kg} &= 1{,}500\ \text{g/m}^3\\
&= 1{,}500\ \text{mg/L}
\end{aligned}
$$

2.2.7 General example of the estimation of flows and pollutant loads

2.2.7.1 Problem configuration

Determine the characteristics of the sewage that is going to be generated by the following town, until year 20 of operation. The population forecast for the project produced the values presented in the table below.

The coverage (served population / total population) is 60% at the beginning of operation (year 0), reaching, as a target, the value of 100% from year 5.

Table 2.27. Main parameters of importance for industrial effluents as a function of the industry type

Type	Activity	BOD or COD	SS	Oils & Grease	Phenols	pH	CN^-	Metals
Food products	Sugar and alcohol	x	x		x	x		
	Meat and fish preservation	x	x			x		
	Dairies	x	x	x		x		
	Slaughter houses	x	x	x				
	Fruit and vegetable canning	x	x			x		
	Milling of grains	x	x					
Drinks	Soft drinks	x	x	x		x		
	Brewery	x	x	x		x		
Textiles	Cotton	x				x		
	Wool	x		x		x		
	Synthetics	x				x		
	Dyeing			x	x	x		x
Tanneries	Vegetable tanning	x	x	x		x		
	Chromium tanning	x	x	x		x		x
Farming	Breeding of animals in confined spaces	x	x					x
Paper	Process. of pulp-cellulose	x	x			x		x
	Manuf. pulp and paper	x	x			x		x
Non-metallic mineral products	Glass and mirrors		x	x		x		x
	Glass fibre	x	x	x	x			
	Cement		x	x		x		
	Ceramics		x	x				x

Rubber	Rubber articles	x	x	x		x		
	Tyres and tubes	x	x	x		x		
Chemical product	Chemical products (various)				x	x	x	x
	Photographic laboratory							x
	Paints and colouring agents							x
	Insecticides					x		x
	Disinfectants				x			x
Plastic	Plastics and resins	x	x		x	x		x
Perfume/soap	Cosmetics, detergents, soap	x		x				x
Mechanical	Production of metal pieces			x	x			
Metallurgy	Production of pig iron	x	x	x	x	x	x	x
	Steelworks		x	x		x	x	x
	Electroplating		x	x	x	x	x	x
Mining	Extraction activities		x			x		
Oil derivatives	Oils and lubricants	x		x	x	x		
	Asphalt works		x	x				
Electrical items	Electrical items						x	x
Wood	Saw mills		x					
Personal services	Laundries	x		x		x		

Table 2.28. Characteristics of the wastewater from some industries

Type	Activity	Unit of production	Specific wastewater flow (m^3/unit)	Specific BOD load (kg/unit)	BOD population equivalent [inhab/(unit/d)]	BOD concentration (mg/L)
Food	Canning (fruit/vegetables)	1 t processed	4–50	30	500	600–7,500
	Pea processing	1 t processed	13–18	16–20	85–400	300–1,350
	Tomato processing	1 t processed	4–8	1–4	50–185	450–1,600
	Carrot processing	1 t processed	11	18	160–390	800–1,900
	Potato processing	1 t processed	7.5–16	10–25	215–545	1,300–3,300
	Citrus fruit processing	1 t processed	9	3	55	320
	Chicken meat processing	1 t produced	15–60	4–30	70–1600	100–2400
	Beef processing	1 t processed	10–16	1–24	20–600	200–6,000
	Fish processing	1 t processed	5–35	3–55	300–2300	2,700–3,500
	Sweets / candies	1 t produced	5–25	2–8	40–150	200–1,000
	Sugar cane	1 t produced	0.5–10	2.5	50	250–5,000
	Dairy (without cheese)	1000 L milk	1–10	1–5	20–100	300–5,000
	Dairy (with cheese)	1000 L milk	2–10	5–40	100–800	500–8,000
	Margarine	1 t produced	20	30	500	1,500
	Slaughter house	1 cow / 2.5 pigs	0.5–3	0.5–5	10–100	1,000–5,000
	Yeast production	1 t produced	150	1100	21,000	7,500
Confined animal breeding	Pigs	live t.d	0.2	2	35–100	10,000–50,000
	Dairy cattle (milking room)	live t.d	0.02–0.08	0.05–0.10	1–2	370–2,300
	Cattle	live t.d	0.15	1.6	65–150	10,000–50,000
	Horses	live t.d	0.15	4–8	65–150	20,000–50,000
	Poultry	live t.d	0.38	0.9	15–20	2,000–3,000
Sugar–alcohol	Alcohol distillation	1 t cane processed	60	220	4,000	3,500
Drinks	Brewery	1 m^3 produced	5–20	8–20	150–350	500–4,000
	Soft drinks	1 m^3 produced	2–5	3–6	50–100	600–2,000
	Wine	1 m^3 produced	5	0.25	5	–

Textiles	Cotton	1 t produced	120–750	150	2,800	200–1,500
	Wool	1 t produced	500–600	300	5,600	500–600
	Rayon	1 t produced	25–60	30	550	500–1,200
	Nylon	1 t produced	100–150	45	800	350
	Polyester	1 t produced	60–130	185	3,700	1,500–3,000
	Wool washing	1 t produced	20–70	100–250	2,000–4,500	2,000–5,000
	Dyeing	1 t produced	20–60	100–200	2,000–3,500	2,000–5,000
	Textile bleaching	1 t produced	–	16	250–350	250–300
Leather and tanneries	Tanning	1 t hide processed	20–40	20–150	1,000–3,500	1,000–4,000
	Shoes	1000 pairs produced	5	15	300	3,000
Pulp and paper	Pulp	1 t produced	15–200	30	600	300
	Paper	1 t produced	30–270	10	100–300	
	Pulp and paper integrated	1 t produced	200–250	60–500	1,000–10,000	300–10,000
Chemical industry	Paint	1 employee	0.110	1	20	10
	Soap	1 t produced	25–200	50	1000	250–2,000
	Petroleum refinery	1 barrel (117 L)	0.2–0.4	0.05	1	120–250
	PVC	1 t produced	12.5	10	200	800
Non-metallic industry	Glass and by-products	1 t produced	50	–	–	–
	Cement (dry process)	1 t produced	5	–	–	–
Steelworks	Foundry	1 t pig iron produced	3–8	0.6–1.6	12–30	100–300
	Lamination	1 t produced	8–50	0.4–2.7	8–50	30–200

Note: data not filled in (-) means non-significant data or data not obtained; t = metric ton (1000 kg)
In various cases the water consumption is considered equal to the wastewater flow produced
Sources: CETESB (1976), Braile and Cavalcanti (1977), Arceivala (1981), Hosang & Bischof (1984), Salvador (1991), Wentzel (without date), Mattos (1998)

The length of the sewerage collection system is estimated to be 30 km for year 0, increasing to 55 km in year 5 (to sustain the increase in the coverage). From this year, it expands at a rate of 1 km per year.

The town has one dairy industry that processes around 5,000 litres of milk per day, for the production of milk, cheese and butter. There are provisions for expansion at year 10, when the production will be doubled.

Year	Total urban population (inhabitants)	Coverage (%)	Served population (inhabitants)	Length of the collection system (km)	Industrial production (litres of milk per day)
0	40,000	60	24,000	30	5,000
5	47,000	100	47,000	55	5,000
10	53,000	100	53,000	60	10,000
15	58,000	100	58,000	65	10,000
20	62,000	100	62,000	70	10,000

Owing to lack of time and other conditions during the design period, it was not possible to obtain samples for characterising the actual sewage composition. Assume adequate values for the missing variables and establish suitable hypotheses for the various parameters in the calculations.

2.2.7.2 Flow estimation

a) Domestic flow

- *Average flow*
  Assume:
  - per capita water consumption: $L_{pcd} = 160$ L/inhab.d (see Tables 2.5 and 2.7)
  - return coefficient (sewage flow/water flow): R = 0.8 (see Section 2.1.2.4)

  The average flow for year 0 is (Equation 2.2):

$$Qd_{av} = \frac{Pop.L_{pcd}.R}{1000} = \frac{24,000 \times 160 \times 0.8}{1000} = 3{,}072\ m^3/d\ (= 35.6\ L/s)$$

The flows for the other years are calculated in a similar way, changing only the population.

- *Maximum flow*

Adopting the Harmon formula (Table 2.10), the Q_{max}/Q_{av} ratio is calculated for the population of every year. For year 0:

$$\frac{Q_{max}}{Q_{av}} = 1 + \frac{14}{4+\sqrt{P}} = 1 + \frac{14}{4+\sqrt{24,000/1,000}} = 2.57$$

The values of Q_{max} are obtained by multiplying Q_{av} by the ratio Q_{max}/Q_{av}. Therefore, for year 0:

$$Q_{max} = 2.57 \times 35.6\ l/s = 91.5\ L/s$$

The ratios and flows for the other years are calculated in a similar manner, altering only the value P (population/1000).

- *Minimum flow*

Adopt a Q_{min}/Q_{av} equal to 0.5. The Q_{min} values are obtained by multiplication with the ratio Q_{min}/Q_{av}. Therefore, for year 0:

$$Q_{min} = 0.5 \times 35.6\ L/s = 17.8\ L/s$$

The ratios and the flows for the other years are calculated in a similar manner.

b) Infiltration flow

Adopt $Q_{inf} = 0.3$ L/s.km for the sewerage system. Consider the resulting flow value for each year occurring only in the average and maximum flows. For year 0:

$$Q_{inf} = 30\ km.\ 0.3\ L/s.km = 9.0\ L/s\ (= 778\ m^3/d)$$

The flows for the other years are calculated in a similar manner, remembering that, from year 5, for each year the sewerage system increases by 1 km.

c) Industrial wastewater flow

Adopt a value of 7 m^3 of wastewater produced per 1000 L of milk processed (water consumption being equal to wastewater production) (see Table 2.28).

Consider that for the years 0 and 5, 5,000 L of milk per day are processed and that for the years 10, 15 and 20, 10,000 L/d of milk are processed (given in the problem).

Assume that the maximum flow is 1.5 times the average flow and the minimum flow is 0.5 times the average flow.

For year 0:

- $Q_{av} = 5\ m^3\ \text{milk} \times 7\ m^3\ \text{wastewater}/m^3\ \text{milk} = 35\ m^3/d\ (= 0.4\ L/s)$
- $Q_{max} = 1.5 \times Q_{av} = 1.5 \times 0.4 = 0.6\ L/s$
- $Q_{min} = 0.5 \times Q_{av} = 0.5 \times 0.4 = 0.2\ L/s$

The flows for the other years are calculated in a similar manner.

d) Total flow

Total flow corresponds to the sum of the domestic, infiltration and industrial flows. Therefore for year 0, the total influent flow to the WWTP is:

Total flow = domestic flow + infiltration flow + industrial flow

- total average flow = 35.6 + 9.0 + 0.4 = 45.0 L/s (= 3,888 m^3/d)
- total maximum flow = 91.5 + 9.0 + 0.6 = 101.1 L/s (= 8,735 m^3/d)
- total minimum flow = 17.8 + 0.0 + 0.2 = 18.0 L/s (= 1,555 m^3/d)

The flows for the other years are calculated in a similar manner .

2.2.7.3 BOD load

a) Domestic BOD

Adopt a per capita BOD production of 50 $gBOD_5$/inhab.d (see Table 2.24)

For the population of year 0:

$$\text{Domestic BOD}_5\text{ load} = 50\text{ g/inhab.d} \times 24{,}000\text{ inhab.} = 1.2 \times 10^6\text{ g/d} = 1{,}200\text{ kg/d}$$

The loads for the other years are calculated in a similar manner.

b) Infiltration water BOD

Consider that the BOD is zero for infiltration water.

c) Industrial BOD

Adopt a value of 25 kg of BOD per 1000 L of milk processed (see Table 2.28).

Consider that for the years 0 and 5, 5,000 L of milk per day are processed and that for the years 10,15 and 20, 10,000 L/d of milk are processed (given in the problem).

For year 0:

$$\text{Industrial BOD}_5\text{ load} = 25\text{ kg/1000 L milk} \times 5{,}000\text{ L milk/d} = 125\text{ kg/d}$$

The loads for the other years are calculated in a similar manner.

d) Total BOD load

Total BOD load corresponds to the sum of the domestic, infiltration and industrial BOD. Therefore for year 0, the total BOD load is:

$$\text{Total BOD}_5\text{ load} = \text{domestic BOD}_5\text{ load} + \text{infiltration BOD}_5\text{ load} + \text{industrial BOD}_5\text{ load}$$

$$\text{Total BOD}_5\text{ load} = 1{,}200 + 0 + 125 = 1{,}325\text{ kg/d}$$

The total loads for the other years are calculated in a similar manner.

Table 2.29. Flows, loads and concentrations in the influent to the WWTP

	Data from the community			Wastewater flows (L/s)																			
				Domestic flow				Industrial flow			Total flow (L/s)				Average BOD load (kg/d)					BOD concentration (mg/L)			
Year	Served popul. (inhab.)	Length sewer. system (km)	Industrial product. (L milk/d)	Min.	Aver.	Max.	Infiltr	Min.	Aver.	Max.	Min.	Aver.	Max.	Total average flow (m^3/d)	Domestic	Infiltr.	Industrial	Total	Popul. equival. (industr.)	Domestic	Infiltr.	Industrial	Total
1	***2***	***3***	***4***	***5***	***6***	***7***	***8***	***9***	***10***	***11***	***12***	***13***	***14***	***15***	***16***	***17***	***18***	***19***	***20***	***21***	***22***	***23***	***24***
0	24,000	30	5,000	17.8	35.6	91.5	9.0	0.2	0.4	0.6	**18.0**	**45.0**	**101.1**	**3,888**	1,200	0	125	**1,325**	2,500	391	0	3,571	**341**
5	47,000	55	5,000	34.8	69.6	159.4	16.5	0.2	0.4	0.6	**35.0**	**86.5**	**176.5**	**7,477**	2,350	0	125	**2,475**	2,500	391	0	3,571	**331**
10	53,000	60	10,000	39.3	78.5	176.0	18.0	0.4	0.8	1.2	**39.7**	**97.3**	**195.2**	**8,409**	2,650	0	250	**2,900**	5,000	391	0	3,571	**345**
15	58,000	65	10,000	43.0	85.9	189.5	19.5	0.4	0.8	1.2	**43.4**	**106.2**	**210.2**	**9,179**	2,900	0	250	**3,150**	5,000	391	0	3,571	**343**
20	62,000	70	10,000	45.9	91.9	200.1	21.0	0.4	0.8	1.2	**46.3**	**113.7**	**222.4**	**9,820**	3,100	0	250	**3,350**	5,000	391	0	3,571	**341**

Calculations in each column:

Col 1: problem data
Col 2: problem data
Col 3: problem data
Col 4: problem data
Col 5 = *col 6* × 0.5
Col 6 = *col 2* × (160 L/inhab.d × 0.8) / 86400 s/d
Col 7 = *col 6* × (1+14/(4+(*col 2*/1000)$^{0.5}$))
Col 8 = *col 3* × 0.3 L/s.km
Col 9 = *col 10* × 0.5
Col 10 = *col 4* × 7 m^3 ww/m^3 milk × 1000 L/m^3 / 86,400 s/d
Col 11 = *col 10* × 1.5
Col 12 = *col 5* + *col 9*
Col 13 = *col 6* + *col 8* + *col 10*
Col 14 = *col 7* + *col 8* + *col 11*
Col 15 = *col 13* × 86400 s/d / 1000 L/m^3
Col 16 = *col 2* × 0.050 kg/inhab.d
Col 17 = 0
Col 18 = *col 4* × 25 kg/1000 L milk
Col 19 = *col 16* + *col 17* + *col 18*
Col 20 = *col 18* / 0.050 kg/inhab.d
Col 21 = *col 16* × 1000 L/m^3 × 1000 g/kg / (*col 6* × 86400 s/d)
Col 22 = *col 17* × 1000 L/m^3 × 1000 g/kg / (*col 8* × 86400 s/d)
Col 23 = *col 18* × 1000 L/m^3 × 1000 g/kg / (*col 10* × 86400 s/d)
Col 24 = (*col 19* / *col 15)* ×1000 g/kg

2.2.7.4 BOD concentration

The BOD concentration is given by the quotient between the BOD load and the wastewater flow (see Equation 2.11). The BOD concentration for the influent to the WWTP for the year 0 is:

$$\begin{aligned}\text{Concentration} &= \text{load/flow} = (1{,}325\ \text{kg/d}) \,/\, (3{,}888\ \text{m}^3\text{/d}) = 0.341\ \text{kg/m}^3 \\ &= 341\ \text{g/m}^3 = 341\ \text{mg/L}\end{aligned}$$

The BOD concentrations for the other years are calculated in a similar manner.

2.2.7.5 Presentation of results

Table 2.29 presents a summary of the various values determined following the proposed criteria. The table can be expanded to include other wastewater characteristics, such as suspended solids, nitrogen and phosphorus. The methodology to be used is the same as for BOD.

3

Impact of wastewater discharges to water bodies

3.1 INTRODUCTION

The present chapter covers basic aspects of water quality and water pollution, analysing in more detail three important topics related to the discharge of sewage to receiving water bodies (rivers, lakes and reservoirs):

- Pollution by organic matter (*dissolved oxygen consumption*)
- Contamination by pathogenic microorganisms (*bacterial die-off*)
- Pollution of lakes and reservoirs (*eutrophication*, caused by nitrogen and phosphorus)

In each of these main items, *causes*, *effects*, *control* and *modelling* of the pollution are discussed. Later in the chapter, water quality legislation is discussed, introducing the concepts of *discharge standard* and *quality standard for the water body*. The importance of the chapter is related to the planning of the removal efficiency and effluent quality to be achieved in the WWTP.

3.2 POLLUTION BY ORGANIC MATTER AND STREAM SELF PURIFICATION

3.2.1 Introduction

The present section covers one of the main problems of water pollution, largely solved in most developed regions but still of great importance in developing

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regions, that is, the consumption of dissolved oxygen (DO) after sewage discharge.

The introduction of *organic matter* into a water body results, indirectly, in the *consumption of dissolved oxygen*. This occurs as a result of the processes of the stabilisation of the organic matter undertaken by bacteria, which use the oxygen available in the liquid medium for their respiration. As expected, the decrease in the dissolved oxygen concentration in the water body has various implications from the environmental point of view.

The objective of this section is the study of the processes of consumption of dissolved oxygen and of stream self-purification, in which the water body recovers itself, through purely natural mechanisms. Both of these processes are analysed from an ecological viewpoint and, subsequently, more specifically through the mathematical representation of the DO profile in the water body.

In broader terms, the process of **self-purification** is associated with the *re-establishment of the equilibrium of the aquatic ecosystem, after the alterations induced by the effluent discharge*. Within a more specific point of view, the conversion of organic compounds into inert compounds, not deleterious from an ecological viewpoint, is an integral part of the process.

It should be understood that the concept of self-purification presents the same relativity as the concept of pollution (see Chapter 1). Water can be considered purified from one point of view, even if not fully purified in hygienic terms, presenting, for instance, pathogenic organisms. From a pragmatic approach, water could be considered purified when its characteristics are not conflicting anymore with their intended uses in each reach of the watercourse. This is because there is no absolute purification: the ecosystem reaches a new equilibrium, but under conditions that are different from before (upstream), owing to the increase in the concentrations of certain compounds and by-products resulting from the decomposition process. As a consequence, the aquatic community is different, even if at a new equilibrium state.

The knowledge and quantification of the self-purification phenomenon is very important, because of the following objectives:

- *To use the assimilation capacity of the rivers*. From a strictly ecological point of view, it could be argued that ecosystems should remain unaltered. However, from a pragmatic perspective, it can be considered that the capacity of a water body to assimilate discharges, without presenting environmental problems, is a natural resource that can be exploited. This realistic vision is of great importance in developing countries where the lack of financial resources justifies the use of the water course to complement, up to a certain point, the processes that occur in sewage treatment (provided this is done with parsimony and with well-defined and safe technical criteria).
- *To avoid effluent discharges above the assimilative capacity of the water body*. In this way, the assimilative capacity of the water body can be used up to a level that is acceptable and non-detrimental. Beyond this level, no further discharges could be allowed.

3.2.2 Ecological aspects of stream self purification

The ecosystem of a water body upstream of the discharge of untreated wastewater is usually in a state of equilibrium. Downstream of the discharge, the equilibrium between the communities is affected, resulting in an initial disorganisation followed by a subsequent tendency towards rearrangement.

In this sense, self-purification can be understood as a phenomenon of **ecological succession**. Along the river, there is a systematic sequence of replacements of a community by another, until a stable community is established, in equilibrium with the local conditions.

The presence or absence of pollution can be characterised by the concept of **species diversity**:

Ecosystem in natural conditions:
- High number of species
- Low number of individuals in each species

Ecosystem under disturbance:
- Low number of species
- High number of individuals in each species

A reduction in the diversity of the species is due to the fact that the *pollution is selective* for the species: only those that adapt to the new environmental conditions survive and, further, proliferate (resulting in a high number of individuals in these few species). The other species do not resist to the new environmental conditions and perish (leading to a reduction in the total number of species).

Because self-purification is a process that develops with time, and considering the dimension of the river as predominantly longitudinal, the stages of ecological succession can be associated with physically identifiable zones in the river. There are four main zones:

- *Degradation zone*
- *Active decomposition zone*
- *Recovery zone*
- *Clean water zone*

These zones occur downstream of the discharge of a predominantly biodegradable organic wastewater. It should be remembered that upstream of the discharge there is a clean water zone characterised by ecological equilibrium and good water quality. Figure 3.1 presents the trajectory along the four zones of the three main water quality parameters: organic matter, heterotrophic bacteria (feeding on organic matter) and dissolved oxygen.

DEGRADATION ZONE

Characteristic	Description
General characteristics	This zone starts soon after the discharge of wastewater to the water body. The main chemical characteristic is the high concentration of organic matter, still in a complex stage but potentially decomposable.
Aesthetic aspects	At the discharge point, water is turbid due to the solids present in the sewage. The sedimentation of the solids results in the formation of sludge banks.
Organic matter and dissolved oxygen	In this zone there is complete disorder, compared to the stable community that existed before. Decomposition of the organic matter, carried out by microorganisms, can have a slow start, depending on the adaptation of the microorganisms to the waste. Normally, in the case of predominantly organic wastewater, the microorganisms present in the wastewater itself are those responsible for the start of the decomposition. Because the decomposition can still be incipient, the oxygen consumption for the respiratory activities of the microorganisms can also be low, allowing sufficient dissolved oxygen for fish. After the adaptation of the microorganisms, the consumption rate of the organic matter becomes high, also implying a high rate of dissolved oxygen consumption.
Microorganisms	After the adaptation period, bacteria start to proliferate, with a massive predominance of aerobic forms, that is, those that depend on the oxygen available in the medium for their metabolic processes. Bacteria, having an abundance of food in the form of the organic matter introduced by the wastewater and also with sufficient oxygen for their respiration, have excellent conditions for development and reproduction. The quantity of organic matter is at a maximum at the discharge point and, due to the consumption by microorganisms, starts to decrease.
Decomposition by-products	There is an increase in the levels of carbon dioxide, one of the products of the microbial respiratory process. With the increase in CO_2 concentration, which is then converted into carbonic acid in water, water may become more acidic and pH may decrease.
Bottom sludge	Anaerobic conditions start to prevail in the sludge at the bottom, due to the difficulty in gas exchange with the atmosphere. As a consequence, there is a production of hydrogen sulphide, which is a potential generator of unpleasant smell.
Nitrogen	Complex nitrogen compounds are still present in high levels, although a large part undergoes conversion to ammonia.
Aquatic community	There is a substantial reduction in the number of living species, although the number of individuals in each one is extremely high, characterising a disturbed ecosystem. Less adapted forms disappear, while resistant and better-adapted forms prevail. The quantity of coliform bacteria is very high, when the discharge is associated with domestic sewage. Also occurring are protozoans that feed on the bacteria, besides fungi that feed on the organic matter. The presence of algae is rare because of the difficulty in light penetration, owing to the turbidity of the water. An evasion of hydras, sponges, crustaceans, molluscs and fish takes place.

ACTIVE DECOMPOSITION ZONE

Characteristics	Description
General characteristics	After the initial disturbance, the ecosystem begins to organise itself. Microorganisms, present in large numbers, actively decompose organic matter. The impact reaches the highest levels and water quality is at its worst state.
Aesthetic aspects	The strongest water colouration can still be observed, together with the dark deposits of sludge at the bottom.
Organic matter and dissolved oxygen	In this zone the dissolved oxygen reaches its lowest concentration. Depending on the magnitude of the discharge, dissolved oxygen may be completely consumed by the microorganisms. In this situation, anaerobic conditions occur in all the liquid bulk. Aerobic life disappears, giving way to predominantly anaerobic microorganisms.
Microorganisms	Bacteria begin to reduce in number, mainly due to the reduction in the available food, which has been largely stabilised. Other factors still interact in the decrease of bacteria, such as light, flocculation, adsorption and sedimentation.
Decomposition by-products	In the event of anaerobic reactions taking place, by-products are, besides carbon dioxide and water, methane, hydrogen sulphide, mercaptans and others, many of them responsible for the generation of bad odours.
Nitrogen	Nitrogen is still present in the organic form, although the larger part is already in the form of ammonia. At the end of the zone, in the presence of dissolved oxygen, oxidation of ammonia to nitrite may start.
Aquatic community	The number of enteric bacteria, pathogenic or not, decreases rapidly. This is due to the fact that these bacteria, well adapted to the environmental conditions in the human intestinal tract, do not resist to the new environmental conditions, which are adverse to their survival. The number of protozoans increases, leading to the rising to a new level in the food pyramid, in the ecological succession process. The presence of some macroorganisms occurs along with insect larvae, adapted to survive under the prevailing conditions. However, the macro fauna is still restricted in species. Hydras, sponges, crustaceans, molluscs and fish have not yet returned.

RECOVERY ZONE

Characteristics	Description
General characteristics	After the intense phase of organic matter consumption and degradation of the aquatic environment, the recovery stage commences.
Aesthetic aspects	Water is clearer and its general appearance is improved. Sludge deposits at the bottom present a less fine and more granulated texture. There is no release of gases or bad smells.
Organic matter and dissolved oxygen	Organic matter, intensely consumed in the previous zones, is largely stabilised and transformed into inert compounds. This implies a lower rate of oxygen consumption through bacterial respiration. In parallel with this, atmospheric oxygen is introduced into the liquid mass, increasing the level of dissolved oxygen (oxygen production by atmospheric reaeration is now larger than its consumption for the stabilisation of the organic matter). The anaerobic conditions that eventually occurred in the previous zone are not present anymore, resulting in another change in the aquatic fauna and flora.

Characteristics	Description
Nitrogen	Ammonia is converted into nitrite and the nitrite to nitrate. Also the phosphorus compounds are transformed into phosphates. A fertilisation of the medium takes place because of the presence of nitrates and phosphates, which are nutrients for algae.
Algae	Due to the presence of nutrients and the higher transparency of the water (allowing a larger light penetration), there are conditions for development of algae. With them, there is the production of oxygen by photosynthesis, increasing the levels of dissolved oxygen in the medium. Also as a result of the presence of algae, the food web becomes more diversified.
Aquatic community	The number of bacteria is now small and, as a result, so is the number of protozoan bacteriophages. Algae are under full development: the first ones to appear are the blue algae (cyanobacteria) on the surface and banks, followed by flagellates, green algae and finally diatoms. Microcrustaceans occur in their maximum number. Molluscs, various worms, dinoflagellates, sponges and insect larvae are present at high numbers. The food chain is more diversified, generating food for the first more tolerant fishes.

CLEAN WATER ZONE

Characteristics	Description
General characteristics	Water is clean again. Conditions are similar to those upstream of the discharge, at least in respect to dissolved oxygen, organic matter and bacteria levels, and probably pathogenic organisms.
Aesthetic aspects	The appearance of the water is similar to that before the pollution occurred.
Organic matter and dissolved oxygen	In the liquid there is a predominance of the completely oxidised and stable forms of inorganic matter, although sludge at the bottom may not be necessarily stabilised. The concentration of dissolved oxygen is close to the saturation level, owing to the low consumption by the microbial population and possibly high production by the algae.
Aquatic community	Because of the mineralisation that occurred in the previous zone, water is now richer in nutrients than before the pollution. Therefore, the production of algae is higher. There is the re-establishment of the normal food web. Various organisms, including large freshwater crustacea, molluscs and fish are present. Species diversity is high. The ecosystem is now stable and the community reaches its climax again.

3.2.3 Dissolved oxygen balance

3.2.3.1 Interacting factors in the DO balance

In ecological terms, the most negative impact of the pollution in a water body caused by organic matter is the decrease in the level of dissolved oxygen, caused by the respiration of microorganisms involved in the purification of the sewage. The impact is extended to all the aquatic community, and each reduction in the level of the dissolved oxygen is selective for certain species.

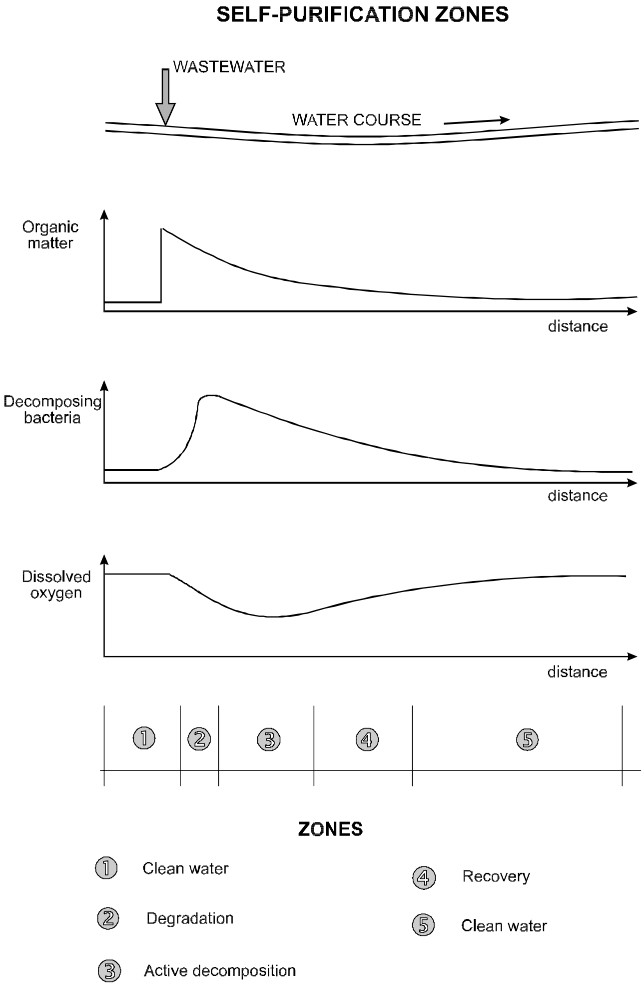


Figure 3.1. Schematic profile of the concentration of the organic matter, bacteria and dissolved oxygen along the length of the water course, with the indication of the self-purification zones

Dissolved oxygen has been traditionally used for the determination of the degree of pollution and self purification in water bodies. Its measurement is simple and its level can be expressed in quantifiable concentrations, allowing mathematical modelling.

Water is an environment poor of oxygen, by virtue of its low solubility. While in the air its concentration is in the order of 270 mg/L, in water, at normal conditions of temperature and pressure, its concentration is reduced approximately to only 9 mg/L. In this way, any large consumption brings substantial impacts in the DO level in the liquid mass.

Table 3.1. Main interacting mechanisms in the DO balance

Oxygen consumption	Oxygen production
– oxidation of the organic matter (respiration)	– atmospheric reaeration
– benthic demand (sludge at the bottom)	– photosynthesis
– nitrification (ammonia oxidation)	

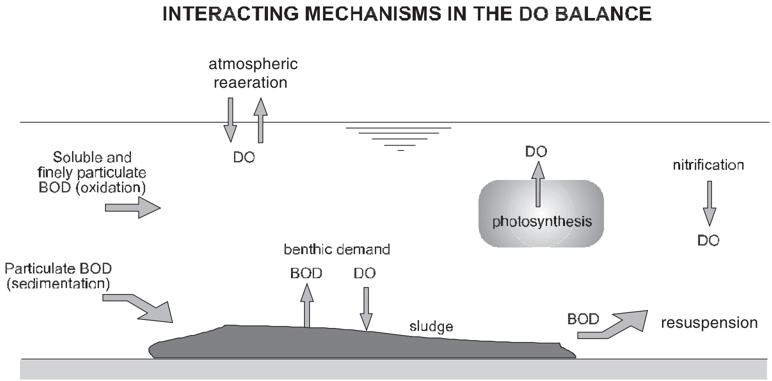


Figure 3.2. Interacting mechanisms in the dissolved oxygen balance

In the self-purification process there is a *balance* between the sources of consumption and the sources of production of oxygen. When the consumption rate is higher than the production rate, the oxygen concentration tends to decrease, the opposite occurring when the consumption rate is lower than the production rate. The main interacting mechanisms in the DO balance in a water body can be found in Figure 3.2 and Table 3.1. In general, the concentrations of the constituents (such as DO) in a water body change as a result of physical processes of *advection* (transportation by the water as it flows in the river channel) and *dispersion* (transportation due to turbulence and molecular diffusion) and biochemical and physical processes of *conversion* (reaction) (Fig. 3.3). The processes take place in the three dimensions of the water body, although in rivers the longitudinal axis (X) is the prevailing one. The mechanisms listed in Table 3.1 are associated with conversion processes.

Changes of concentration with time = **Advection**: *Transport of the constituent in the velocity field of the fluid medium* + **Dispersion**: *Turbulence and diffusion spread particles of the constituent* + **Conversion**: *Biological, chemical and physical reactions*

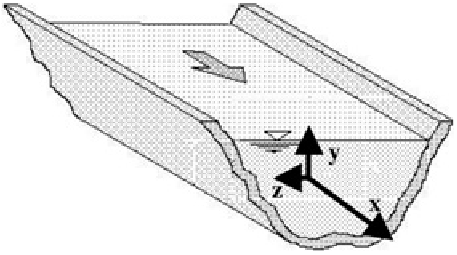


Figure 3.3. Axes along which spatial and temporal variations of the concentrations of the water constituents take place (in rivers, the X axis is predominant)

Oxygen consumption

a) Oxidation of the organic matter

The organic matter in sewage is present in two forms: *suspended* (*particulate*) and *dissolved* (*soluble*). Settleable suspended solids tend to settle in the water body, forming a sludge layer at the bottom. The dissolved matter, together with the suspended solids of small dimensions (hardly settleable) remains in the liquid mass.

The oxidation of the latter fraction of organic matter corresponds to the main factor in the oxygen consumption. The consumption of DO is due to the respiration of the microorganisms responsible for the oxidation, principally aerobic heterotrophic bacteria. The simplified equation for the stabilisation (oxidation) of organic material is:

$$\text{Organic matter} + O_2 + \text{bacteria} \rightarrow CO_2 + H_2O + \text{bacteria} + \text{energy} \qquad (3.1)$$

Bacteria, in the presence of oxygen, convert the organic matter to simple and inert compounds, such as water and carbon dioxide. As a result, bacteria tend to grow and reproduce, generating more bacteria, while there is availability of food (organic matter) and oxygen in the medium.

b) Benthic (sediment) demand

The settled organic matter in suspension, which formed the bottom sludge layer, also needs to be stabilised. A large part of the conversion is completed under anaerobic conditions, because of the difficulty of oxygen to penetrate the sludge layer. This form of conversion, being anaerobic, implies the non- consumption of oxygen.

However, the upper part of the sludge layer, in the order of some millimetres of thickness, still has access to oxygen from the supernatant water. The sludge stabilisation is completed under aerobic conditions in this fine layer, resulting in the consumption of oxygen. Besides, some partial by-products of the anaerobic decomposition may dissolve, cross the aerobic sludge layer and diffuse itself in the bulk of the liquid, exerting an oxygen demand. The oxygen demand originating from these combined factors associated with the sludge is called *benthic* (or *sediment*) demand.

Another factor that can cause oxygen demand is the reintroduction of previously settled organic matter into the bulk of the liquid, caused by the *resuspension* of the sludge layer. This resuspension occurs in occasions of high flows and velocities in the water course. The sludge, not yet completely stabilised, represents a new source of oxygen demand.

The importance of the benthic demand and the resuspension of the sludge in the dissolved oxygen balance depends on a series of simultaneously interacting factors, many of them difficult to quantify.

c) Nitrification

Another oxidation process is the one associated with the conversion of ammonia into nitrite and this nitrite into nitrate, in the process of *nitrification*.

The microorganisms involved in this process are chemoautotrophs, which have carbon dioxide as the main carbon source and which draw their energy from the oxidation of an inorganic substrate, such as ammonia.

The transformation of ammonia into nitrite is completed according to the following simplified reaction:

$$\text{ammonia} + O_2 \rightarrow \text{nitrite} + H^+ + H_2O + \text{energy} \qquad (3.2)$$

The transformation of nitrite into nitrate occurs in sequence, in accordance with the following simplified reaction:

$$\text{nitrite} + O_2 \rightarrow \text{nitrate} + \text{energy} \qquad (3.3)$$

It is seen that in both reactions there is oxygen consumption. This consumption is referred to as nitrogenous demand or second-stage demand, because it takes place after the oxidation of most of the carbonaceous matter. This is due to the fact that the nitrifying bacteria have a slower growth rate compared with the heterotrophic bacteria, implying that nitrification also occurs at a slower rate.

Oxygen production

a) Atmospheric reaeration

Atmospheric reaeration is frequently the main factor responsible for the introduction of oxygen into the liquid medium.

Gas transfer is a physical phenomenon, through which gas molecules are exchanged between the liquid and the gas at their *interface*. This exchange results in an increase in the concentration in the liquid gas phase, if this phase is not saturated with gas.

This is what happens in a water body, in which the DO concentration is reduced due to the processes of the stabilisation of the organic matter. As a consequence, DO levels are lower than the saturation concentration, which is given by the solubility of the gas at a given temperature and pressure. In this situation there is an *oxygen deficit*. If there is a deficit, there is the search for a new equilibrium, thus allowing a larger absorption by the liquid mass.

The transfer of oxygen from the gas phase to the liquid phase occurs basically through the following two mechanisms:

- Molecular diffusion
- Turbulent diffusion

In a quiescent water body, *molecular diffusion* prevails. This diffusion may be described as a tendency of any substance to uniformly spread itself about all of the available space. However, this mechanism is very slow and requires a long time for the gas to reach the deepest layers of the water body.

The mechanism of *turbulent diffusion* is much more efficient, because it involves the main factors of an effective aeration: creation of interfaces and renewal of interfaces. The first one is important, because it is through these interfaces that gas exchange occurs. The second one is also significant, because the fast renewal of the interfaces permits that localised saturation points are not formed, besides conducting the dissolved gas to the various depths of the liquid mass, as a result of the mixing.

The diffusion condition to prevail is a function of the hydrodynamic characteristics of the water body. A shallow river with rapids presents excellent conditions for an efficient turbulence. In these conditions, molecular diffusion is negligible. On the other hand, in lakes, molecular diffusion tends to predominate, unless wind promotes good mixing and interface renewal.

b) Photosynthesis

Photosynthesis is the main process used by autotrophic organisms to synthesise organic matter, being a characteristic of organisms containing chlorophyll. The process takes place only in the presence of light energy, according to the following simplified equation (there are many intermediate steps):

$$CO_2 + H_2O + \text{light energy} \rightarrow \text{organic matter} + O_2 \tag{3.4}$$

Photosynthesis reaction is exactly opposite to the respiration reaction. While photosynthesis is a process of fixing light energy and forming glucose molecules of high energy potential, respiration is essentially the opposite, that is, release of this energy for subsequent use in metabolic processes (Branco, 1976).

Light dependence controls the distribution of photosynthetic organisms to locations to where light is present. In waters with a certain turbidity, such as from soil particles or suspended solids from waste discharges, the possibility of the presence of algae is smaller and, as a result, so is the photosynthetic activity. This is seen in the first self-purification zones, where the predominance is almost exclusively of heterotrophic organisms. In these zones, respiration surpasses production.

Table 3.2. River water quality parameters modelled by some computer software currently available

	Program									
Quality parameter	1	2	3	4	5	6	7	8	9	10
Temperature	√		√	√	√	√	√	o.s.	o.s.	
Bacteria			√	√	√	√	√	o.s.	o.s.	
DO-BOD	√	√	√	√	√	√	√	o.s.	o.s.	√
Nitrogen	√	√	√	√	√	√	√	o.s.	o.s.	√
Phosphorus	√	√	√	√	√	√	√	o.s.	o.s.	√
Silica			√		√	√	√	o.s.	o.s.	
Phytoplankton	√	√	√	√	√	√	√	o.s.	o.s.	√
Zooplankton			√		√	√		o.s.	o.s.	
Benthic algae					√	√	√	o.s.	o.s.	

1 = QUAL2E (USEPA, 1987); 2 = WASP5 (USEPA, 1988); 3 = CE-QUAL-ICM (US Army Engineer Waterways Experiment Station, 1995); 4 = HEC5Q (US Army Engineer Hydrologic Engineering Centre, 1986); 5 = MIKE11 (Danish Hydraulic Institute, 1992); 6 = ATV Model (ATV, Germany, 1996); 7 = Salmon-Q (HR Wallingford, England, 1994); 8 = DUFLOW (Wageningen Univ., Holland, 1995); 9 = AQUASIM (EAWAG, Switzerland, 1994); 10 = DESERT (IIASA, Austria, 1996); o.s. = open structure (can be modified by the user).
Source: Rauch et al, 1998; Shanaham et al, 1998; Somlyódy, 1998

In general, the autotrophs carry out much more synthesis than oxidation, generating a positive balance of organic compounds that constitute an energy reserve for the heterotrophs, besides an excess of oxygen that sustains the respiration of other organisms.

3.2.3.2 Water quality models

a) More complete water quality models

River quality models have been used since the development of the classic model of DO and BOD by Streeter and Phelps, in 1925. This model represented a milestone in water and environmental engineering. Subsequently, various other models were developed, including the model of Camp (1954), increasing the level of complexity and the number of state and input variables, but at the same time maintaining the same conceptual structure of the classic Streeter–Phelps model. A widely known model, within the relatively recent generation of models, is the QUAL2E model, developed by the United States Environmental Protection Agency (USEPA), which represents in greater depth the cycles of O, N and P in water. There still is another class of models, understood as ecosystem models that represent suspended solids, various algae groups, zooplankton, invertebrates, plants and fish.

The Task Group on River Water Quality Modelling (2001) from IWA (International Water Association) developed a new model (IWA, Scientific and Technical Report 12), with a large number of components and processes, presented in matrix-format. This Task Group also presented an interesting comparison between various models currently available, synthesised in Table 3.2 (Rauch et al, 1998; Shanaham et al, 1998; Somlyódy, 1998).

Naturally that, the larger the number of variables (quality parameters) represented by a model, the larger is the number of kinetic parameters and stoichiometric coefficients to be obtained or adopted and, therefore, the larger is the difficulty in calibrating the model.

For developing countries, with all the large regional diversity of problems and solutions concerning water quality, it is difficult to establish generalisations about the use of models. However it is always important to have in mind that all the water quality models mentioned have been developed in countries that have already practically solved their basic pollution problems, such as the discharge of raw wastewater containing organic matter (domestic and industrial). In these conditions, it is natural that attention is now given to transient events and diffused pollution, for example. Nevertheless, in most of the developing counties the basic problems have not yet been solved, and simpler models still have a large contribution to give for the adequate management of water resources.

b) Simplified models

In the present text, for the sake of simplicity, only the two main components in the DO balance are covered, namely:

- *oxygen consumption*: oxidation of organic matter (respiration)
- *oxygen production*: atmospheric reaeration.

Naturally there are cases that justify the inclusion of other components, when it is felt that these are important in the DO balance. However, field and laboratory work for a reliable evaluation of these parameters needs to be undertaken intensively and rigorously, substantially increasing the complexity level of the study. The adoption of sophisticated mathematical models demands the availability of time and financial resources compatible with the proposed formulation, what is frequently not the case in developing countries. Therefore, in the present text, the more simplified version of the model (Streeter–Phelps version) is adopted, allowing an easier identification of occasional problems in its structure and parameter values.

Another important point is that *any user of a sophisticated model should understand well the basic principles of the Streeter–Phelps model*, in order to avoid a blind use of the computer software, without knowing the basic processes that are being represented.

It should be explained that the model described is *restricted to aerobic conditions* in the water body. Under anaerobic conditions, the conversion rate of organic matter is slower, being carried out by a biomass with completely different characteristics. Anaerobic conditions may occur frequently in simulations of the discharge of untreated wastewater to water courses with small dilution capacity.

c) Hydraulic representation

In the model structure, the hydraulic regime of the water body must be taken into consideration. There are basically three types of hydraulic models for a water

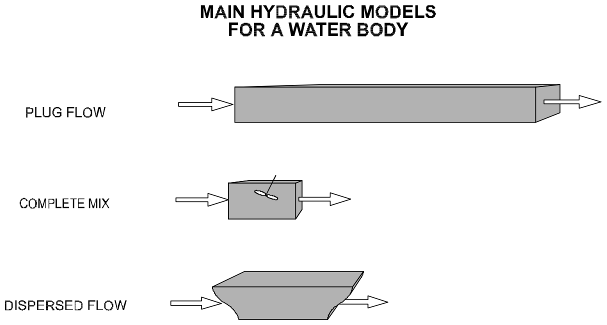


Figure 3.4. Different hydraulic regimes for a water body: (a) plug flow, (b) complete mix, (c) dispersed flow

body or a reactor (see Figure 3.4):

- *Plug flow*
- *Complete mix*
- *Dispersed flow*

A water body in the ideal *complete-mix* regime is characterised by having the same concentration at all points in the liquid mass. Thus, the effluent concentration is equal at whatever point in the water body. This representation is usually applied to well-mixed lakes and reservoirs. This regime is also called *CSTR* (completely-stirred tank reactor).

A predominantly linear water body, such as a river, can be characterised through the *plug-flow* regime. In the ideal plug flow there are no exchanges between the upstream and downstream sections. Each section functions as a plug, in which the water quality is the same in all points and the community is adapted to the ecological conditions prevailing in each moment. Along with the downstream movement of the plug, the various self-purification reactions take place. Hydraulically, this model is similar to the case in which a tank with water, equal to the plug, remains the same period of time subjected to the same reactions and processes, therefore having the same water quality as that of the plug in the water body (see Figure 3.5).

The two characteristics represented above are for idealised situations. In reality, water bodies present a characteristic of dispersion of the pollutants, which is intermediate between the two extreme situations: total dispersion (completely mixing) and no dispersion (plug flow). Therefore the water bodies or their reaches can be characterised by a dispersion coefficient. High dispersion coefficients are associated with water bodies approaching a completely mixing regime, whereas reduced coefficients are associated with water bodies approaching plug-flow conditions. The *dispersed-flow* regime is particularly relevant with rivers under estuarine influence or with very low flow velocities.

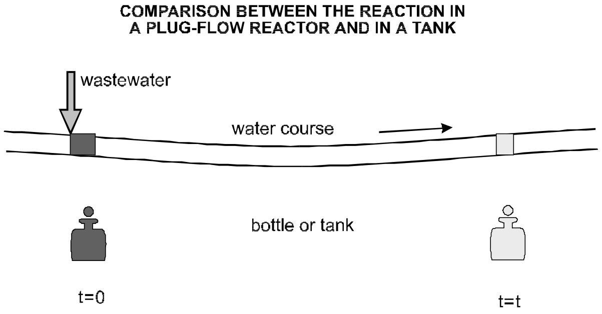


Figure 3.5. Comparison between a bottle, tank or vessel and a plug in a plug-flow reactor

Further detailing of the various flow regimes can be obtained in Chapter 8.

In the present chapter, the simplified approach of considering a river to be represented by the plug-flow regime is adopted, which is acceptable for most situations.

3.2.3.3 The dissolved oxygen profile

It is interesting to analyse the variations (decrease and increase) in the DO concentration along the water course in a graph, which plots the so-called *DO profile* or *DO sag curve*. In this graph, the vertical axis is the DO concentration and the horizontal axis is the distance or travelling time, along which the biochemical transformations take place. From the graph, the following elements can be obtained:

- identification of the consequences of the discharge
- connection of the pollution with the self-purification zones
- relative importance of the consumption and production of oxygen
- critical point of lowest DO concentration
- comparison between the critical DO concentration and the minimum allowable concentration, according with the legislation
- location where the water course returns to the desired conditions

The modelling of these items depends essentially on the understanding of the two main interacting mechanisms in the DO balance: *deoxygenation* and *atmospheric reaeration*. These topics are covered in the following subsections.

3.2.4 Kinetics of deoxygenation

3.2.4.1 Mathematical formulation

As already seen, the main ecological effect of organic pollution in a water body is the decrease in the levels of dissolved oxygen. This decrease is associated with

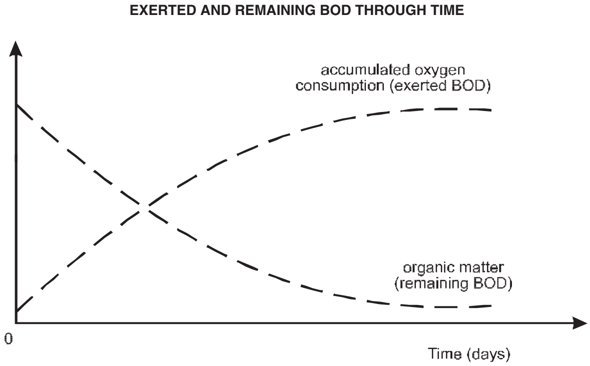


Figure 3.6. Exerted BOD (oxygen consumed) and remaining BOD (remaining organic matter) along time

the Biochemical Oxygen Demand (BOD), described in Chapter 2. To standardise the results, the concept of the standard BOD is frequently used, being expressed by BOD_5^{20}. However, the oxygen consumption in the sample varies with time, that is, the BOD value is different each day. The objective of the present section is to mathematically analyse the progress of oxygen consumption with time.

The concept of BOD, representing both the organic matter concentration and the oxygen consumption, can be understood by the following two distinct angles, both having as units mass of oxygen per unit volume (e.g. mgO_2/L):

- *remaining BOD*: concentration of the organic matter remaining in the liquid mass at a given time
- *exerted BOD*: cumulative oxygen consumption for the stabilisation of the organic matter

The progress of BOD with time, according with both concepts, can be seen in Figure 3.6.

The two curves are symmetrical, like mirror images. At time zero, the organic matter is present in its total concentration, while the oxygen consumed is zero. With the passing of time, the remaining organic matter reduces, implying an increase in the accumulated oxygen consumption. After a period of several days, the organic matter has been practically all stabilised (remaining BOD close to zero), while the oxygen consumption has been practically all exerted (BOD almost completely exerted). The understanding of this phenomenon is important, because both curves are an integral part of the DO model.

The kinetics of the reaction of the remaining organic matter (remaining BOD) follows a *first-order reaction*. A first-order reaction is that in which the rate of change of the concentration of a substance is proportional to the first power of the concentration. The first-order reactions are of fundamental importance in environmental engineering, since many reactions are modelled according with this kinetics. Reaction kinetics is discussed in depth in Chapter 8.

The equation of the progress of the remaining BOD with time can be expressed by the following differential equation:

$$\frac{dL}{dt} = -K_1.L \tag{3.5}$$

where:

L = remaining BOD concentration (mg/L)
t = time (day)
K_1 = deoxygenation coefficient (day^{-1})

The interpretation of Equation 3.5 is that the oxidation rate of the organic matter (dL/dt) is proportional to the organic matter concentration still remaining (L), at any given time t. Therefore, the larger the BOD concentration, the faster is the deoxygenation. After a certain time, in which BOD has been reduced by stabilisation, the reaction rate will be lower, as a result of the lower concentration of organic matter.

The *deoxygenation coefficient* K_1 is a parameter of great importance in the modelling of dissolved oxygen, being discussed in the next section.

The integration of Equation 3.5, between the limits of $L = L_0$ and $t = 0$ and $t = t$, leads to:

$$\boxed{L = L_0.e^{-K_1.t}} \tag{3.6}$$

where:

L = remaining BOD at any given time t (mg/L)
L_0 = remaining BOD in $t = 0$ (mg/L)
t = time (d)

Attention should be given to the fact that, in many references, this equation is written in a decimal form (base 10), and not in base e. Both forms are equivalent, provided the coefficient is expressed in the compatible base (K_1 *base e* = 2.3 × K_1 *base 10*). In the present text, the values of the coefficients are expressed in **base e.**

In terms of *oxygen consumption*, the quantification of the exerted BOD is important. This is obtained through Equation 3.6, leading to:

$$\boxed{y = L_0.(1 - e^{-K_1.t})} \tag{3.7}$$

where:

y = exerted BOD at a time t (mg/L). Note that $y = L_0 - L$.
L_0 = remaining BOD, at $t = 0$ (as defined above), or exerted BOD (when $t = \infty$). Also called *ultimate BOD demand*, by the fact that it represents the total BOD at the end of the stabilisation process (mg/L).

Example 3.1

The interpretation of a laboratory analysis of a river water sample taken downstream from a sewage discharge leads to the following values: (a) coefficient of deoxygenation: $K_1 = 0.25\ d^{-1}$; (b) ultimate demand $L_0 = 100$ mg/L. Calculate the exerted BOD at days 1, 5 and 20.

Solution:

Using Equation 3.7, where $y = L_0 . (1 - e^{-K1.t})$:

- For t = 1 day:

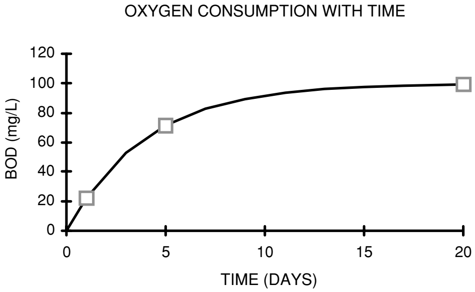
$$y_1 = 100\,(1 - e^{-0.25\times 1}) = 22\ \text{mg/L}$$

- For t = 5 days:

$$y_5 = 100\,(1 - e^{-0.25\times 5}) = 71\ \text{mg/L}\ (= \text{BOD}_5)$$

- For t = 20 days:

$$y_{20} = 100\,(1 - e^{-0.25\times 20}) = 99\ \text{mg/L}$$



It is seen that at day 20 the BOD has been practically all exerted (y_{20} practically equal to L_0).

The ratio between BOD_5 and the ultimate demand L_0 is: 71/100 = 0.71. Therefore, at day 5, approximately 71% of the oxygen consumption have been exerted or, in other words, 71% of the total organic matter (expressed in terms of BOD) has been stabilised. Inversely, the L_0/BOD_5 ratio is equal to 100/71 = 1.41.

3.2.4.2 The deoxygenation coefficient K_1

The coefficient K_1 depends on the characteristics of the organic matter, besides temperature and presence of inhibitory substances. Treated effluents, for example, have a lower degradation rate due to the fact that the larger part of the easily biodegradable organic matter has already been removed in the treatment plant, leaving only the slowly biodegradable fraction in the effluent. Average values of K_1 can be found present in Table 3.3.

There are mathematical and statistical processes that can be utilised for the determination of the deoxygenation coefficient, in case there are samples from the water under investigation. The input data for these methods are the values of the exerted BOD at various days, typically days 1, 2, 3, 4 and 5 or 1, 3, 5, 7 and 9. The laboratory tests must include, not only the BOD at 5 days, but also the BOD for all the other days, so that the rate of deoxygenation can be estimated. Nitrification must be inhibited in the BOD test, especially for the sequence that goes up to nine days.

The determination is not trivial, because there are two parameters to be simultaneously determined: K_1 and L_0. *Non-linear regression analysis* can be used, fitting Equation 3.7 to the various pairs of t and BOD to obtain the values of the parameters K_1 and L_0. In the present book, it is sufficient to use the values of K_1 obtained from the table of typical values (Table 3.3). It should be noted that, especially in the case of shallow rivers receiving untreated sewage, the deoxygenation may be higher than that determined in the laboratory, due to biofilm respiration at the river bottom.

Figure 3.7 illustrates the influence of the value of K_1, through the trajectories of the cumulative oxygen consumption of two samples with different values of K_1 but with the same ultimate demand value ($L_0 = 100$ mg/L). The sample with the higher K_1 ($0.25\ d^{-1}$) presents a faster oxygen consumption rate compared with the sample with the lower K_1 ($0.10\ d^{-1}$). Values of BOD close to the ultimate demand are reached in less time with the sample with the greater K_1.

The importance of the coefficient K_1 and the relativity of the BOD_5 concept can be analysed through the following example (see Figure 3.8). Two distinct samples present the same value of BOD_5 (100 mg/L). Apparently, one could conclude that the impact in terms of dissolved oxygen consumption is the same in the two

Table 3.3. Typical values of K_1 (base e, 20 °C)

Origin	K_1 (day^{-1})
Water course receiving concentrated raw sewage	0.35–0.45
Water course receiving raw sewage of a low concentration	0.30–0.40
Water course receiving primary effluent	0.30–0.40
Water course receiving secondary effluent	0.12–0.24
Water course with clean water	0.09–0.21

Source: Adapted from Fair et al (1973) and Arceivala (1981)

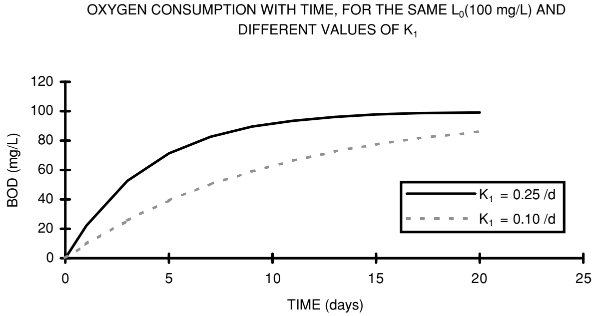


Figure 3.7. Trajectory of the oxygen consumption for different values of K_1 and same values of ultimate BOD

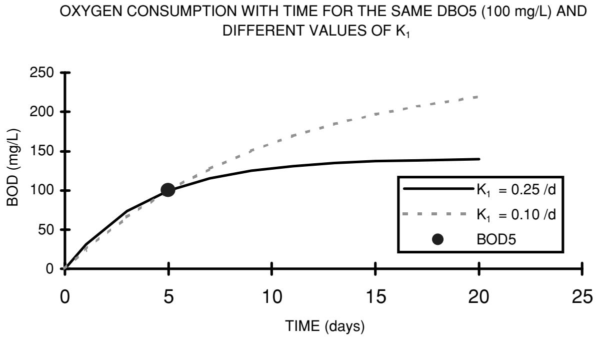


Figure 3.8. Influence of the coefficient K_1 on the progression of BOD. Two samples with the same BOD value at 5 days (100 mg/L), but with different values of K_1 and, hence, different values of the ultimate BOD.

situations. However, if the progression of BOD is measured through various days, different BOD values can be observed for all the days, with the exception of the fifth day. This is due to the fact that the coefficients of deoxygenation are distinct in the two samples. The first presents a slower stabilisation rate ($K_1 = 0.10\ \text{day}^{-1}$), implying a high ultimate BOD, still not reached on day 20. The second sample presents a higher K_1 ($K_1 = 0.25\ \text{day}^{-1}$), and the demand is practically satisfied by day 20.

These considerations emphasise the aspect that the interpretation of the BOD data must always be associated with the concept of the coefficient of deoxygenation and, consequently, the rate of oxidation of the organic matter. This comment is of greater importance with industrial wastewaters, which are capable of presenting a large variability with regards to biodegradability or to the stabilisation rate of the organic matter.

3.2.4.3 Influence of temperature

Temperature has a great influence on the microbial metabolism, affecting, as a result, the stabilisation rates of organic matter. The empirical relation between temperature and the deoxygenation coefficient can be expressed in the following form:

$$\boxed{K_{1_T} = K_{1_{20}}.\theta^{(T-20)}} \tag{3.8}$$

where:

$K_{1_T} = K_1$ at a temperature $T(d^{-1})$

$K_{1_{20}} = K_1$ at a temperature $T = 20\,°C\ (d^{-1})$

$T =$ liquid temperature (°C)

$\theta =$ temperature coefficient (−)

A value usually employed for θ in this reaction is **1.047**. The interpretation of this value with relation to Equation 3.8 is that the value of K_1 increases 4.7% for every 1 °C increment in the temperature of the water.

Also to be commented is that changes in the temperature affect K_1, but do not alter the value of the ultimate demand L_0.

3.2.5 Kinetics of reaeration

3.2.5.1 Mathematical formulation

The theory of gas transfer is covered in detail in Chapter 11. In the present chapter only the essential concepts necessary for the understanding of the atmospheric reaeration phenomenon are presented. When water is exposed to a gas, a continuos exchange of molecules occurs between the liquid and gas phases. As soon as the solubility concentration of the gas in the liquid phase is reached, both flows start to be equal in magnitude, such that there is no overall change of the gas concentration in both phases. This dynamic equilibrium defines the **saturation concentration** (C_s) of the gas in the liquid phase.

However, in case that there is the consumption of dissolved gas in the liquid phase, the main transfer flux is in the gas-liquid direction, in order to re-establish the equilibrium. The atmospheric reaeration process takes place according to this concept. The oxygen consumption in the stabilisation of the organic matter makes the DO concentration to be below the saturation level. As a result, there is a greater flux of atmospheric oxygen to the liquid mass (Figure 3.9).

The kinetics of reaeration can also be characterised by a first-order reaction (similarly to the deoxygenation), according to the following equation:

$$\frac{dD}{dt} = -K_2.D \tag{3.9}$$

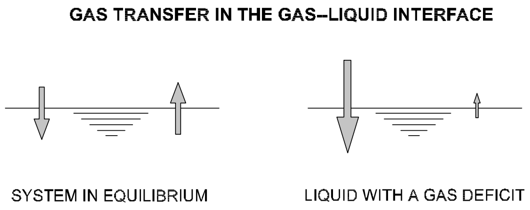


Figure 3.9. Gas exchanges in a system in equilibrium and in a liquid with a dissolved gas deficit

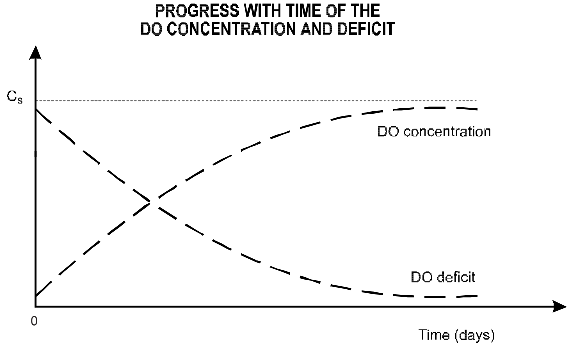


Figure 3.10. Temporal progress of the concentration and deficit of dissolved oxygen

where:

D = dissolved oxygen deficit, that is, the difference between the saturation concentration (C_s) and the existing concentration at a time t (C) (mg/L)

t = time (d)

K_2 = reaeration coefficient (base e) (d^{-1})

Through Equation 3.9 it is seen that the absorption rate of oxygen is directly proportional to the existing deficit. The larger the deficit, the greater the gas transfer rate.

Integration of Equation 3.9, with D_0 when t = 0, leads to:

$$\boxed{D = D_0.e^{-K_2.t}} \qquad (3.10)$$

where:

D_0 = initial oxygen deficit (mg/L)

The temporal progress of the deficit ($D = C_s - C$) and the DO concentration (C) can be seen in Figure 3.10. The deficit and concentration curves are symmetrical and like mirror images. With the increase of the DO concentration with time due to the reaeration, the deficit decreases at the same rate.

Table 3.4. Typical values for K_2 (base e, 20 °C)

	K_2 (day^{-1})	
Water body	Deep	Shallow
Small ponds	0.12	0.23
Slow rivers, large lakes	0.23	0.37
Large rivers with low velocity	0.37	0.46
Large rivers with normal velocity	0.46	0.69
Fast rivers	0.69	1.15
Rapids and waterfalls	>1.15	>1.61

Source: Fair et al (1973), Arceivala (1981)

3.2.5.2 The reaeration coefficient K_2

In a sample of deoxygenated water, the value of the coefficient K_2 can be determined through statistical methods. These methods are based on regression analysis, using either the original Equation 3.10, or some logarithmic transformation of it. The input data are the DO values at various times t. The output data are the saturation concentration C_s and the coefficient K_2. In a water body, however, the experimental determination of K_2 is very complex, being outside the scope of the present text.

The value of the coefficient K_2 has a larger influence on the results of the DO balance than the coefficient K_1, because of the fact that the ranges of variation of K_1 are narrower. There are three methods for estimating the value of the coefficient K_2 in the river under study:

- average tabulated values
- values as a function of the hydraulic characteristics of the water body
- values correlated with the flow of the water body

a) Average tabulated values of K_2

Some researchers, studying water bodies with different characteristics, proposed average values for K_2 based on a qualitative description of the water body (Table 3.4).

Shallower and *faster* water bodies tend to have a *larger reaeration coefficient,* due, respectively, to the greater ease in mixing along the depth and the creation of more turbulence on the surface (see Figure 3.11). The values in Table 3.4 can be used in the absence of specific data from the water body. It must be taken into consideration that *the values from this table are usually lower than those obtained by the other methods* discussed below. However, there are indications that, in some situations, the tabulated values result in better fitting to measured DO data than those obtained from hydraulic formula.

b) K_2 values as a function of the hydraulic characteristics of the water body

Other researchers correlated the reaeration coefficient K_2 with the hydraulic variables of the water body. Various field techniques were employed in their studies, such as tracers, equilibrium disturbance, mass balance and others.

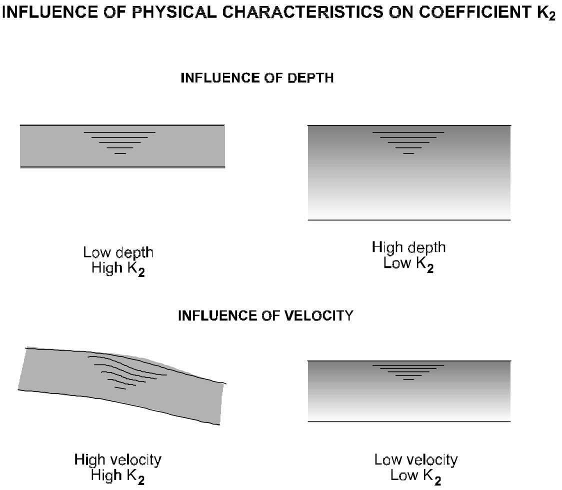


Figure 3.11. Influence of the physical characteristics of the water body on the coefficient K_2

The literature presents various formulas, conceptual and empirical, relating K_2 with the depth and the velocity of the water body. Table 3.5 and Figure 3.12 present three of the main formulas, with application ranges that are complementary.

If there are natural cascades with free water falls, other formulations for the estimation of the atmospheric reaeration may be used. Von Sperling (1987) obtained the following empirical formula, based on the study of some waterfalls in Brazil:

$$C_e = C_0 + K.(C_s - C_0) \tag{3.11}$$

$$K = 1 - 1.343.H^{-0.128}.(C_s - C_0)^{-0.093} \tag{3.12}$$

where:

C_e = effluent (downstream) DO concentration (mg/L)
C_0 = influent (upstream) DO concentration (mg/L)
K = efficiency coefficient (−)
C_s = DO saturation concentration (mg/L)
H = height of each free fall (m)

c) K_2 values correlated with the river flow

Another approach for estimating K_2 is through the correlation with the river flow. This can be justified by the fact that the depth and the velocity are intimately associated with flow.

Table 3.5. Values of the coefficient K_2, according with models based on hydraulic data (base e, 20 °C)

Researcher	Formula	Application range
O'Connor & Dobbins (1958)	$3.73.v^{0.5}H^{-1,5}$	$0.6\ m \leq H < 4.0\ m$ $0.05\ m/s \leq v < 0.8\ m/s$
Churchill et al (1962)	$5.0.v^{0.97}H^{-1,67}$	$0.6\ m \leq H < 4.0\ m$ $0.8\ m/s \leq v < 1.5\ m/s$
Owens et al (cited by Branco, 1976)	$5.3.v^{0.67}H^{-1,85}$	$0.1\ m \leq H < 0.6\ m$ $0.05\ m/s \leq v < 1.5\ m/s$

Notes:

- v: velocity of the water body (m/s)
- H: height of the water column (m)
- Ranges of applicability adapted and slightly modified from Covar (EPA, 1985), for simplicity

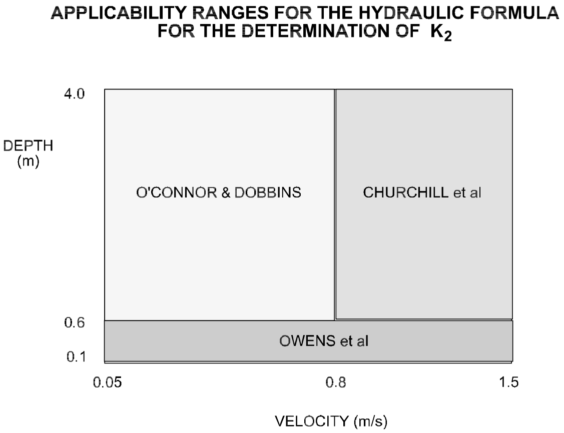


Figure 3.12. Approximate applicability ranges of three hydraulic formulas for estimating K_2 (modified from Covar, cited in EPA, 1985)

The procedure is based on the determination of K_2 using the hydraulic formula (section b above), for each pair of values of v and H from historical records in the river. Subsequently, a regression analysis is performed between the resulting values of K_2 and the corresponding flow values Q. The relation between K_2 and Q may be expressed as $K_2 = m.Q^n$, where m and n are coefficients.

The advantage of this form of expression is that the reaeration coefficient may be calculated for any flow conditions (by interpolation or even some extrapolation), especially minimum flows, independently from depth and velocity values.

3.2.5.3 Influence of temperature

The influence of temperature is felt in two different ways:

- *an increase in temperature reduces the solubility of oxygen in the liquid medium (decrease of the saturation concentration C_s)*
- *an increase in temperature accelerates the oxygen absorption processes (increase of K_2)*

These factors act in opposite directions. The increase in K_2 implies an increase in the reaeration rate. However, a reduction in the saturation concentration corresponds to a decrease in the oxygen deficit D, resulting in a reduction in the reaeration rate. The overall influence on the reaeration rate depends on the magnitude of each variation but is frequently not substantial.

The influence of the temperature on the saturation concentration is discussed in Section 3.2.7k.

The influence of temperature on the reaeration coefficient can be expressed in the traditional form (Equation 3.13):

$$\boxed{K_{2_T} = K_{2_{20}}.\theta^{(T-20)}} \tag{3.13}$$

where:

$K_{2_T} = K_2$ at a temperature $T(d^{-1})$

$K_{2_{20}} = K_2$ at a temperature $T = 20\,°C\ (d^{-1})$

$T =$ liquid temperature (°C)

$\theta =$ temperature coefficient $(-)$

A value frequently used for the temperature coefficient θ is **1.024**.

3.2.6 The DO sag curve

3.2.6.1 Mathematical formulation of the model

In 1925 the researchers Streeter and Phelps established the mathematical bases for the calculation of the dissolved oxygen profile in a water course. The structure of the model proposed by them (known as the **Streeter–Phelps model**) is classical within environmental engineering, setting the basis for all the other more sophisticated models that succeeded it. For the relatively simple situation in which only the deoxygenation and the atmospheric reaeration are taken into account in the DO balance, the rate of change of the oxygen deficit with time can be expressed by the following differential equation, originated from the interaction of the deoxygenation and reaeration equations previously seen:

$$\text{Rate of change of the DO deficit} = \text{DO consumption} - \text{DO production} \tag{3.14}$$

$$\frac{dD}{dt} = K_1.L - K_2.D \tag{3.15}$$

Integration of this equation leads to:

$$D_t = \frac{K_1.L_0}{K_2 - K_1}.(e^{-K_1.t} - e^{-K_2.t}) + D_0.e^{-K_2.t} \tag{3.16}$$

This is the general equation that expresses the variation of the oxygen deficit as a function of time. The DO concentration curve (DO_t or C_t) can be obtained directly from this equation, knowing this:

$$DO_t = C_s - D_t \tag{3.17}$$

Thus:

$$\boxed{C_t = C_s - \left\{\frac{K_1.L_0}{K_2 - K_1}.(e^{-K_1.t} - e^{-K_2.t}) + (C_s - C_0).e^{-K_2.t}\right\}} \tag{3.18}$$

In the DO sag curve, one point is of fundamental importance: the point in which the DO concentration reaches its lowest value. This is called *critical time*, and the DO concentration, the *critical concentration*. The knowledge of the critical concentration is very important, because it is based on it that the need and efficiency of the wastewater treatment will be established. The treatment must be implemented with a BOD removal efficiency which is sufficient to guarantee that the critical DO concentration is higher than the minimum value required by legislation (standard for the water body).

The DO sag curve as a function of time (or of the distance) is S-shaped, as shown in Figure 3.13. In the curve, the main points are identified: the DO concentration in the river and the critical DO concentration.

3.2.6.2 Model equations

a) DO concentration and deficit in the river immediately after mixing with the wastewater

$$\boxed{C_0 = \frac{Q_r.DO_r + Q_w.DO_w}{Q_r + Q_w}} \tag{3.19}$$

$$\boxed{D_0 = C_s - C_0} \tag{3.20}$$

where:

C_0 = initial oxygen concentration, immediately after mixing (mg/L)
D_0 = initial oxygen deficit, immediately after mixing (mg/L)

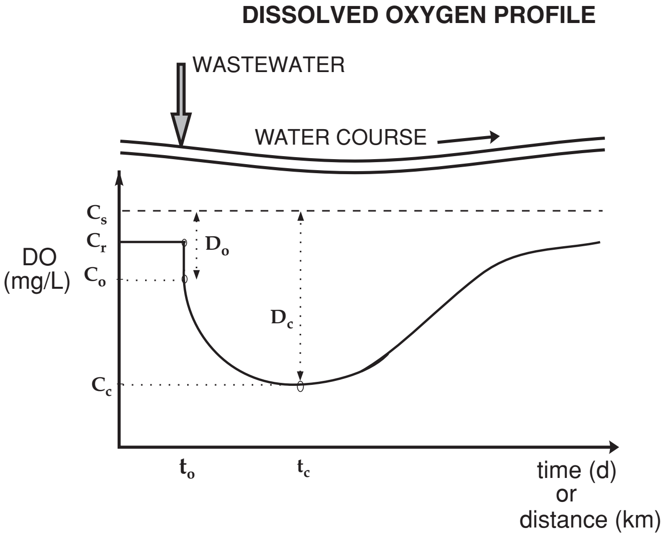


Figure 3.13. Characteristic points in the DO sag curve

C_s = oxygen saturation concentration (mg/L)
Q_r = river flow upstream of the wastewater discharge (m^3/s)
Q_w = wastewater flow (m^3/s)
DO_r = dissolved oxygen concentration in the river, upstream of discharge (mg/L)
DO_w = dissolved oxygen concentration in the wastewater (mg/L)

It can be observed that the value of C_0 is obtained through the weighted average between the flows and the DO levels in the river and the wastewater.

b) BOD₅ and ultimate BOD concentrations in the river immediately after mixing with the wastewater

$$\boxed{BOD_{5_0} = \frac{(Q_r.BOD_r + Q_w.BOD_w)}{Q_r + Q_w}} \tag{3.21}$$

$$\boxed{L_0 = BOD_{5_0}.K_T = \frac{(Q_r.BOD_r + Q_w.BOD_w)}{Q_r + Q_w}.K_T} \tag{3.22}$$

where:

BOD_{5_0} = BOD_5 concentration, immediately after mixing (mg/L)
L_0 = ultimate oxygen demand (BOD_u), immediately after mixing (mg/L)
BOD_r = BOD_5 concentration in the river (mg/L)
BOD_w = BOD_5 concentration in the wastewater (mg/L)
K_T = coefficient for transforming BOD_5 to the ultimate BOD_u (−)

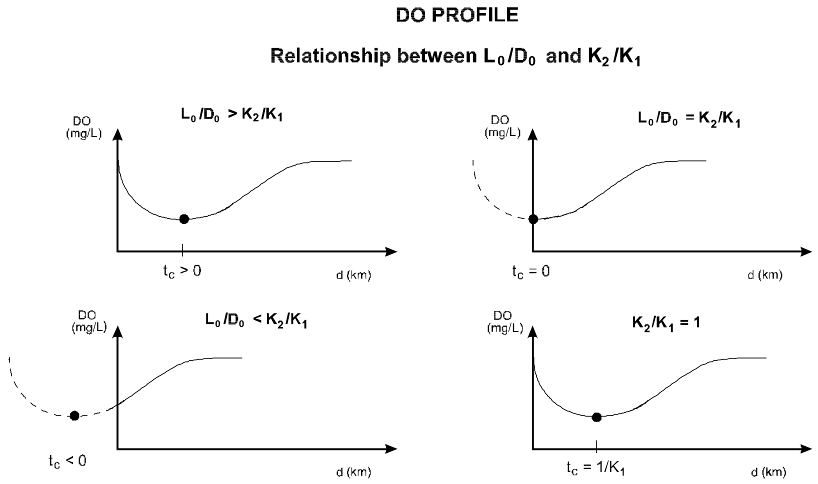


Figure 3.14. Relation between the critical time and the terms (L_0/D_0) and (K_2/K_1)

$$\boxed{K_T = \frac{BOD_u}{BOD_5} = \frac{1}{1 - e^{-5.K_1}}} \tag{3.23}$$

The value of L_0 is also obtained through the weighted average between the flows and the biochemical oxygen demands of the river and of the wastewater.

c) *DO profile as a function of time*

$$\boxed{C_t = C_s - \left\{\frac{K_1.L_0}{K_2 - K_1}.(e^{-K_1.t} - e^{-K_2.t}) + (C_s - C_0).e^{-K_2.t}\right\}} \tag{3.24}$$

In the event that a negative DO concentration ($C_t < 0$) is calculated, even though mathematically possible, there is no physical meaning. In this case, anaerobic conditions (DO = 0 mg/L) have been reached and the Streeter–Phelps model is no longer valid.

d) *Critical time (time when the minimum DO concentration occurs)*

$$\boxed{t_c = \frac{1}{K_2 - K_1}.\ln\left\{\frac{K_2}{K_1}.\left[1 - \frac{D_0.(K_2 - K_1)}{L_0.K_1}\right]\right\}} \tag{3.25}$$

The following situations can occur when using the critical time formula, depending on the relation between (L_0/D_0) and (K_2/K_1) (see Figure 3.14):

- **$L_0/D_0 > K_2/K_1$**
  Critical time is positive. From the mixing point there will be a decrease in the DO concentration, leading to a critical deficit that is higher than the initial deficit.

- **$L_0/D_0 = K_2/K_1$**
  Critical time is equal to zero, that is, it occurs exactly in the mixing point. The initial deficit is equal to the critical deficit. The water course has a good regenerating capacity for the discharge received, and will not suffer a drop in DO level.
- **$L_0/D_0 < K_2/K_1$**
  Critical time is negative. This indicates that, from the mixing point, the dissolved oxygen concentration tends to increase. The initial deficit is the largest observed deficit. In terms of DO, the water course presents a self-purification capacity that is higher than the degeneration capacity of the wastewater. In practical terms, the critical time can be considered equal to zero, with the occurrence of the lowest DO values at the mixing point.
- **$K_2/K_1 = 1$**
  The application of the critical time formula leads to a *mathematical indetermination*. The limit when K_2/K_1 tends to 1 leads to a critical time equal to $1/K_1$.

e) Critical deficit and concentration of dissolved oxygen

$$\boxed{D_c = \frac{K_1}{K_2}.L_0.e^{K_1.t_c}} \tag{3.26}$$

$$\boxed{C_c = C_s - D_c} \tag{3.27}$$

f) BOD removal efficiency required in the wastewater treatment

The Streeter–Phelps model still permits the calculation of the maximum allowable BOD load of the sewage, which will lead to the critical DO concentration being equal to the minimum permitted by the legislation. Such procedure involves some iterations because, for each alteration of the maximum permissible load, there is a modification of the critical time. However, in a real situation, with more than one discharge point, this approach becomes not very practical. What is usually done its to consider BOD removal efficiencies which are compatible with the existing or available wastewater treatment processes, and to recalculate the DO profile for each new condition. *The most economic situation is usually that in which the minimum DO concentration is only marginally higher than the minimum permitted by legislation*.

3.2.7 Input data for the DO model

The following input data are necessary for the utilisation of the Streeter–Phelps model (see Figure 3.15):

- river flow, upstream of the discharge (Q_r)
- wastewater flow (Q_w)

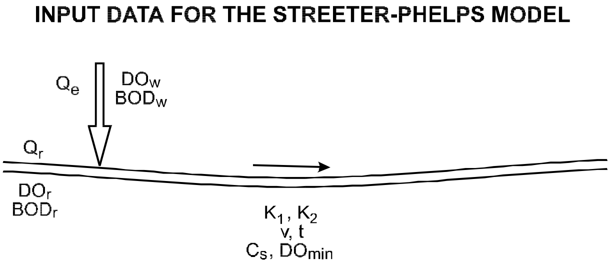


Figure 3.15. Input data required for the Streeter–Phelps model

- dissolved oxygen in the river, upstream of the discharge (DO_r)
- dissolved oxygen in the wastewater (DO_w)
- BOD_5 in the river, upstream of the discharge (BOD_r)
- BOD_5 of the wastewater (BOD_w)
- deoxygenation coefficient (K_1)
- reaeration coefficient (K_2)
- velocity of the river (v)
- travelling time (t)
- saturation concentration of DO (C_s)
- minimum dissolved oxygen permitted by legislation (DO_{min})

a) River flow (Q_r)

The flow of the receiving body is a variable of extreme importance in the model, having a large influence on the simulation results. Therefore it is necessary to obtain the most precise flow value, whenever possible.

The use of the DO model can be with any of the following flows, depending on the objectives:

- flow observed in a certain period
- mean flow (annual average, average in the rainy season, average in the dry season)
- minimum flow

The *observed flow* in a certain period is used for model calibration (adjusting the model coefficients), so that the simulated data are as close as possible to the observed (measured) data in the water body during the period under analysis.

The *mean flow* is adopted when the simulation of the average prevailing conditions is desired, such as during the year, rainy months or dry months.

The *minimum flow* is utilised for the planning of catchment areas, the evaluation of the compliance with environmental standards of the water body and for the allocation of pollutant loads. Therefore, the determination of the required efficiencies for the treatment of various discharges must be determined in the **critical conditions**. These critical conditions in the receiving body occur exactly in the minimum flow period, when the dilution capacity is lower.

The *critical flow* must be calculated from the historical flow measurement data from the water course. The analysis of methods to estimate minimum flows is outside the scope of the present text, being well covered in hydrology books. Usually a minimum flow with a return period of 10 years and a duration of the minimum of 7 days ($Q_{7,10}$), is adopted. This can be understood as a value that may repeat itself in a probability of every 10 years, consisting of the lowest average obtained in 7 consecutive days. Therefore, in each year of the historical data series the 365 average daily flows are analysed. In each year a period of 7 days is selected, which resulted in the lowest average flow (average of 7 values). With the values of the lowest 7-day average for every year, an statistical analysis is undertaken, allowing interpolation or extrapolation of the value for a return period of 10 years.

Adoption of the 10-year return period in the $Q_{7,10}$ concept leads to small flows and frequently to the requirement of high BOD removal efficiencies, the cost of which should always be borne in mind, especially in developing countries. For these countries, probably a shorter return period would be more realistic, especially considering that the current condition is probably already of a polluted river.

Another approach is the utilisation of percentiles, such as a 90%ile value ($\mathbf{Q_{90}}$). In this concept, 90% of the flow values are greater than the critical flow, and only 10% are lower than it. This approach usually leads to critical flows that are greater than those based on $Q_{7,10}$.

Under any flow conditions, the utilisation of the concept of the *specific discharge* (L/s.km^2) is helpful. Knowing the drainage area at the discharge point and adopting a value for the specific discharge, the product of both leads to the flow of the water course. The specific discharge values can vary greatly from region to region, as a function of climate, topography, soil, etc., For $Q_{7,10}$ conditions, the following ranges are typical: (a) arid regions: probably less than 1,0 L/s.km^2; (b) regions with good water resources availability: maybe higher than 3,0 L/s.km^2; and (c) in intermediate regions: values close to 2.0 L/s.km^2.

b) Wastewater flow (Q_w)

Wastewater flows considered in DO modelling are usually *average flows*, without coefficients for the hour and day of highest consumption. The sewage flow is obtained through conventional procedures, using data from population, per capita water consumption, infiltration, specific contribution (in the case of industrial wastes), etc. The calculation is detailed in Chapter 2.

c) Dissolved oxygen in the river, upstream of the discharge point (DO_r)

The dissolved oxygen level in a water body, upstream of a waste discharge, is a result of the upstream activities in the catchment area.

Ideally, historical data should be used in this analysis. When doing so, coherence is required: if the simulation is for a dry period, only samples pertaining to the dry period should be considered.

In case that it is not possible to collect water samples at this point, the DO concentration can be estimated as a function of the approximate pollution level of the water body. If there are *few indications of pollution*, a DO_r value of 80% to 90% of the oxygen saturation value (see item k below) can be adopted.

In the event that the water body is already well polluted upstream of the discharge, a sampling campaign is justified, or even an upstream extension of the boundaries of the studies should be considered, in order to include the main polluting points. In such a situation, the value of DO_r will be well below the saturation level.

d) Dissolved oxygen in the wastewater (DO_w)

In sewage, the dissolved oxygen levels are normally nihil or close to zero. This is due to the large quantity of organic matter present, implying a high consumption of oxygen by the microorganisms. Therefore the DO of **raw sewage** is usually adopted as **zero** in the calculations.

In case that the wastewater is **treated**, the following considerations could be made:

- *Primary treatment*. Primary effluents can be assumed as having DO equal to zero.
- *Anaerobic treatment*. Anaerobic effluents also have DO equal to zero.
- *Activated sludge* and *biofilm reactors*. Effluents from these systems undergo a certain aeration at the effluent weir on the secondary sedimentation tanks, enabling DO to increase to 2 mg/L or slightly more. If the discharge outfall is long, this oxygen could be consumed as a result of the remaining BOD from the treatment.
- *Facultative or maturation ponds*. Effluents from facultative or maturation ponds can have during day time DO levels close to saturation, or even higher, due to the production of pure oxygen by the algae. At night the DO levels decrease. For the purpose of the calculations, DO_W values around 4 to 6 mg/L can be adopted.
- *Effluents subjected to final reaeration*. Treatment plant effluents may be subject to a final reaeration stage, in order to increase the level of dissolved oxygen. A simple system is composed by cascade aeration, made up of a sequence of steps in which there is a free fall of the liquid. DO values may raise a few milligrams per litre, depending on the number and height of the steps. Sufficient head must be available for the free falls. Gravity aeration should not be used directly for anaerobic effluents, due to the release of H_2S in the gas-transfer operation. Section 11.10 presents the methodology for calculating the increase in DO.

Table 3.6. BOD_5 as a function of the water body characteristics

River condition	BOD_5 of the river (mg/L)
Very clean	1
Clean	2
Reasonably clean	3
Doubtful	5
Bad	>10

Source: Klein (1962)

e) BOD_5 in the river, upstream of discharge (BOD_r)

BOD_5 in the river, upstream of the discharge, is a function of the wastewater discharges (point or diffuse sources) along the river down to the mixing point. The same considerations made for DO_r about sampling campaigns and the inclusion of upstream polluting points are also valid here.

Klein (1962) proposed the classification presented in Table 3.6, in the absence of specific data.

f) BOD_5 in the wastewater (BOD_w)

The BOD_5 concentration in **raw** domestic sewage has an average value in the order of 300 mg/L. The BOD can also be estimated through the quotient between the BOD load (calculated from the population and the per capita BOD contribution) and the wastewater flow (domestic sewage + infiltration). For more details, see Section 2.2.5.

In case there are industrial discharges of importance, particularly from agro-industries and others with high content of organic matter in the effluent, they must be included in the calculation. These values can be obtained by sampling or through literature data. See Section 2.2.6.

For a treated wastewater, of course the BOD removal efficiency must be taken into account, since treatment is the main environmental control measure to be adopted. In this case, the BOD_5 in the wastewater is:

$$\boxed{BOD_{tw} = \left(1 - \frac{E}{100}\right) \cdot BOD_{rw}} \qquad (3.28)$$

where:

$BOD_{tw} = BOD_5$ of the treated wastewater (mg/L)
$BOD_{rw} = BOD_5$ of the raw wastewater (mg/L)
$E = BOD_5$ removal efficiency of the treatment (%)

Table 4.9 presents typical ranges of BOD removal efficiency from various wastewater treatment systems. An overview of these systems can be found in

Chapter 4. Various other chapters of this book are dedicated to the detailed description of these systems.

g) Deoxygenation coefficient (K_1)

The deoxygenation coefficient can be obtained following the criteria presented in Section 3.2.4.2. It must be noted that water bodies that receive biologically treated wastewater have a lower value of K_1 (see Table 3.3). For liquid temperatures different from 20 °C, the value of K_1 must be corrected (seer Section 3.2.4.3).

h) Reaeration coefficient (K_2)

The reaeration coefficient can be obtained following the procedures outlined in Section 3.2.5.2. For liquid temperatures different from 20 °C, the value of K_2 must be corrected (see Section 3.2.5.3).

i) Velocity of the water body (v)

The velocity of the liquid mass in the water course may be estimated using the following methods:

- direct measurement in the water course
- data obtained from flow-measuring points
- use of hydraulic formulas for open channels
- correlation with flow

In DO simulations that can be done under any flow conditions, obtaining the velocity through the last two methods is more convenient. In other words, it is important that the velocity is coherent with the flow under consideration, since in dry periods the velocities are usually lower, with the opposite occurring in wet periods.

The hydraulic formulas are presented in pertinent literature. The most adequate friction factor should be chosen as a function of the river bed characteristics (see Chow, 1959).

The flow-correlation method should follow a methodology similar to the one described in Item 3.2.5.2c for the reaeration coefficient. The model to be obtained can have the form $v = cQ^d$, where c and d are coefficients obtained from regression analysis.

j) Travel time (t)

In the Streeter–Phelps model, the theoretical travel time that a particle takes to complete a certain river reach is a function of the velocity and the distance. This is because the model assumes a plug-flow regime and does not consider the effects of dispersion. Therefore, knowing the distances and having determined the velocities in each reach, the residence time is obtained directly from the relation:

$$\boxed{t = \frac{d}{v \cdot 86,400}} \tag{3.29}$$

where:

t = travel (residence) time (d)
d = distance (m)
v = velocity in the water body (m/s)
86,400 = number of seconds per day (s/d)

k) DO saturation concentration (C_s)

The saturation concentration of the oxygen can be calculated based on theoretical considerations, or through the use of empirical formulas. The value of C_s is a function of water temperature and altitude:

- *The increase in temperature reduces the saturation concentration* (the greater agitation of molecules in the water tends to make the dissolved gases pass to the gas phase)
- *The increase in altitude reduces the saturation concentration* (the atmospheric pressure is lower, thus exerting a lower pressure for the gas to be dissolved in the water).

There are some empirical formulas in the literature (the majority based on regression analysis) that directly supply the value of C_s (mg/L) as a function of, for example, the temperature T (°C). A formula frequently employed is (Pöpel, 1979):

$$C_s = 14.652 - 4.1022 \times 10^{-1}.T + 7.9910 \times 10^{-3}.T^2 - 7.7774 \times 10^{-5}.T^3 \tag{3.30}$$

The influence of the altitude can be computed by the following relation (Qasim, 1985):

$$f_H = \frac{C_s{}'}{C_s} = \left(1 - \frac{\text{Altitude}}{9450}\right) \tag{3.31}$$

where:

f_H = correction factor for altitude, for the DO saturation concentration (−)
C_s' = DO saturation concentration at the altitude H (mg/L)
Altitude = altitude (m above sea level)

Salinity also affects the solubility of oxygen. The influence of dissolved salts can be computed by the following empirical formula (Pöpel, 1979):

$$\boxed{\gamma = 1 - 9 \times 10^{-6} \cdot C_{sal}} \tag{3.32}$$

where:

γ = solubility reduction factor ($\gamma = 1$ for pure water)
C_{sal} = dissolved salts concentration (mg Cl^-/L)

Table 3.7. Saturation concentration for oxygen in clean water (mg/L)

	Altitude (m)			
Temperature (°C)	0	500	1000	1500
10	11.3	10.7	10.1	9.5
11	11.1	10.5	9.9	9.3
12	10.8	10.2	9.7	9.1
13	10.6	10.0	9.5	8.9
14	10.4	9.8	9.3	8.7
15	10.2	9.7	9.1	8.6
16	10.0	9.5	8.9	8.4
17	9.7	9.2	8.7	8.2
18	9.5	9.0	8.5	8.0
19	9.4	8.9	8.4	7.9
20	9.2	8.7	8.2	7.7
21	9.0	8.5	8.0	7.6
22	8.8	8.3	7.9	7.4
23	8.7	8.2	7.8	7.3
24	8.5	8.1	7.6	7.2
25	8.4	8.0	7.5	7.1
26	8.2	7.8	7.3	6.9
27	8.1	7.7	7.2	6.8
28	7.9	7.5	7.1	6.6
29	7.8	7.4	7.0	6.6
30	7.6	7.2	6.8	6.4

Table 3.7 presents the saturation concentrations for oxygen in clean water at different temperatures and heights.

l) Minimum allowable dissolved oxygen concentration in the water body (DO_{min})

The minimum levels of dissolved oxygen that need to be maintained in the water body are stipulated by the legislation applicable in the country or region. In the absence of specific legislation, it is usual to try to maintain DO concentrations in the water body equal to or above 5.0 mg/L.

3.2.8 Measures to control water pollution by organic matter

When analysing the possible pollution control strategies for a water body, it is fundamental to have a regionalised view of the catchment area as a whole, aiming at reaching the desired water quality, instead of treating the problems as isolated points. When a regional focus is employed, a great variety of alternative strategies becomes available, normally leading to lower costs and greater safety. An adequate organisational and institutional structure is essential.

Among the main control measures, there are:

- *wastewater treatment*

- *flow regularisation in the water body*
- *aeration of the water body*
- *aeration of the treated wastewater*
- *allocation of other uses for the water body*

a) Wastewater treatment

Individual or collective sewage treatment before discharge is usually the main and often the only control strategy. However, its possible combination with some of the other presented strategies should be analysed, aiming at obtaining a technically favourable solution at the lowest cost. Wastewater treatment is the main alternative analysed in the present book.

b) Flow regularisation of the water body

This alternative generally consists of building an upstream dam, in order to augment the low flow under critical conditions. The most attractive option is to include multiple uses for the dam, such as irrigation, hydroelectric power generation, recreation, water supply and others.

Another positive aspect is that the effluent from the dams can contain higher levels of dissolved oxygen because of the aeration at the effluent weir.

It must be remembered that the implementation of dams is a delicate topic from an environmental point of view. Also, if the upstream catchment area is not properly protected, the reservoir can become a point of localised pollution and risks of eutrophication.

c) Aeration of the water body

Another possibility is to provide aeration in the water body at a point downstream of the discharge, maintaining the DO concentrations above the minimum allowable.

An advantage of this alternative resides in the fact that the assimilative capacity of the water course can be totally used in periods of high flow and the aeration can be limited to dry periods only. This is a form of collective treatment and involves the distribution of the costs between the various beneficiaries.

The following aeration methods can be employed:

- diffused-air aeration
- surface (mechanical) aeration
- aeration at weirs
- turbine aeration
- injection by pressure

Besides this, natural waterfalls can contribute significantly to the DO increase (see Section 3.2.5.2, equations 3.11 and 3.12).

d) Aeration of the treated wastewater

At the effluent weir of the WWTP, after satisfaction of the oxygen demand, the effluent can suffer a simple aeration, usually by means of weirs. These devices can increase the DO concentration in the order of some milligrams per litre (1 to 3 mg/L) contributing to the fact that, already at the mixing point, a slightly higher DO concentration is achieved. In anaerobic effluents, however, aeration must be avoided because it causes the release of hydrogen sulphide, which causes problems of bad odours.

e) Allocation of other uses for the water body

In case it is not possible (mainly for economic reasons) to control the polluting discharges in order to preserve the water quality as a function of the intended uses of the water body, these uses could be re-evaluated in the river or in selected reaches.

Therefore, it could be necessary to attribute less noble uses for a certain reach of the river, due to the unfeasibility of implementing the control measure at the desired level. The allocation of uses for the water body should be carried out in such a way as to optimise regional water resources, aiming at their various uses (Arceivala, 1981).

Example 3.2

The city and the industry from the general example in Chapter 2 (Section 2.2.7) discharge together their effluents into a water course. The catchment area upstream does not present any other significant discharges. Downstream of the discharge point the stream travels a distance of 50 km until it reaches the main river. In this downstream reach there are no other significant discharges.

Main data:

- Wastewater characteristics (values obtained from the mentioned example, for year 20 of operation):
  - Average wastewater flow: 0.114 m^3/s
  - BOD concentration: 341 mg/L
- Catchment area characteristics:
  - Drainage area upstream of the discharge point: 355 km^2
  - Specific discharge of the water body (minimum flow per unit area of the basin) 2.0 $L/s.km^2$
- Water body characteristics:
  - Altitude: 1,000 m
  - Water temperature: 25 °C
  - Average depth: 1.0 m
  - Average velocity: 0.35 m/s

Assume all other necessary data.

Required items:

- Calculate and plot the DO sag curve until the stream joins the main river
- Present wastewater treatment alternatives for the pollution control of the water body
- Calculate and plot the DO sag curves for the alternatives analysed

***Example 3.2* (*Continued*)**

Solution:

Determination of the input data (raw wastewater)

a) River flow (Q_r)

Minimum specific discharge: $Q_{r_{spec}} = 2.0$ L/s.km^2
Drainage basin area: $A = 355$ km^2

$$Q_r = Q_{r_{spec}}.A = 2.0\,\text{L/s.km}^2 \times 355\,\text{km}^2 = 710\,\text{L/s} = 0.710\,\text{m}^3/\text{s}$$

b) Wastewater flow (Q_w)

$$Q_w = 0.114\ \text{m}^3/\text{s (stated in the problem)}$$

c) Dissolved oxygen in the river (DO_r)

Considering that the water body does not present significant discharges, adopt the DO concentration upstream of the discharge as 90% of the saturation value. Saturation concentration: $C_s = 7.5$ mg/L (25 °C, 1,000 m of altitude) (see item j below)

$$DO_r = 0.9 \times C_s = 0.9 \times 7.5\ \text{mg/L} = 6.8\ \text{mg/L}$$

d) Dissolved oxygen in the sewage (DO_w)

$$DO_w = 0.0\ \text{mg/L (adopted)}$$

e) Biochemical oxygen demand in the river (BOD_r)

From Table 3.6, for a clean river:

$$BOD_r = 2.0\ \text{mg/L}$$

f) Biochemical oxygen demand of the wastewater (BOD_w)

$$BOD_w = 341\ \text{mg/L (stated in the problem)}$$

g) Deoxygenation coefficient (K_1)

As laboratory tests were not possible, K_1 is adopted as an average value from the literature (raw sewage – see Table 3.3):

$$K_1 = 0.38\ \text{d}^{-1}(20\,^\circ\text{C, base e})$$

Correction of K_1 for a temperature of 25 °C (Equation 3.8):

$$K_{1_T} = K_{1_{20C}} \cdot \theta^{(T-20)} = 0.38 \times 1.047^{(25-20)} = 0.48\text{d}^{-1}$$

Example 3.2 (_Continued_)

h) Reaeration coefficient (K_2)

Depth of the water body: H = 1.0 m
Velocity of the water body: v = 0.35 m/s

Formula to be used, according with the applicability range (see Table 3.5 and Figure 3.11): O'Connor and Dobbins formula:

$$K_2 = 3.73 \cdot \frac{v^{0.5}}{H^{1.5}} = 3.73 \cdot \frac{(0.35\ \mathrm{m/s})^{0.5}}{(1.0\ \mathrm{m})^{1.5}} = 2.21 \mathrm{d}^{-1} (20\,^{\circ}\mathrm{C}, \text{base e})$$

Correction for the temperature of 25 °C (Equation 3.13):

$$K_{2_T} = K_{2_{20C}} \theta^{(T-20)} = 2.21 \times 1.024^{(25-20)} = 2.49\ \mathrm{d}^{-1}$$

i) Travel time

Velocity of the water body: v = 0.35 m/s
Travel distance: d = 50,000 m

The travel time to arrive at the confluence with the main river is (Equation 3.29):

$$t = \frac{d}{v.86,400} = \frac{50,000\,\mathrm{m}}{0.35\,\mathrm{m/s}.86{,}400\,\mathrm{s/d}} = 1.65\,\mathrm{d}$$

j) Saturation concentration of dissolved oxygen (C_s)

Water temperature: T = 25 °C
Altitude: 1,000 m

From Table 3.6:

$$C_s = 7.5\ \mathrm{mg/L}$$

l) Minimum allowable dissolved oxygen (DO_{min})

$$DO_{min} = 5.0\ \mathrm{mg/L}\ \text{(adopted)}$$

Example 3.2 (*Continued*)

Summary:

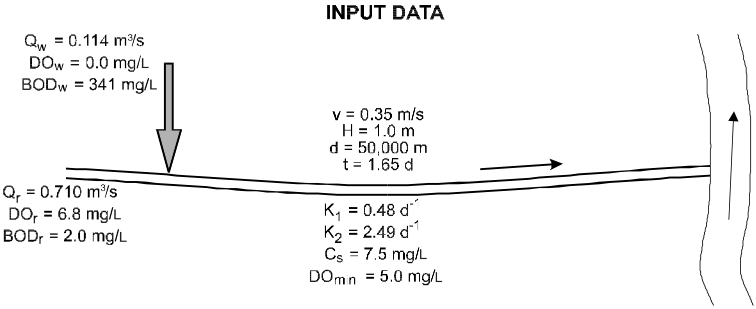


Fig Ex 3.2d Input data for the example. Raw wastewater.

Calculation of the output data – raw wastewater

a) Oxygen concentration at the mixing point (C_0)

From Equation 3.19:

$$C_0 = \frac{Q_r.DO_r + Q_w.DO_w}{Q_r + Q_w} = \frac{0.710 \times 6.8 + 0.114 \times 0.0}{0.710 + 0.144} = 5.9\ \text{mg/L}$$

The dissolved oxygen deficit is (see Equation 3.20):

$$D_0 = C_s - C_0 = 7.5 - 5.9 = 1.6\,\text{mg/L}$$

b) Ultimate BOD concentration at the mixing point (L_0)

The transformation factor BOD_5 to BOD ultimate is given by Equation 3.23:

$$K_T = \frac{BOD_u}{BOD_5} = \frac{1}{1 - \exp(-5.K_1)} = \frac{1}{1 - \exp(-5\ \times\ 0.48)} = 1.10$$

The BOD_5 at the mixing point is obtained from Equation 3.21:

$$\begin{aligned} BOD5_0 &= \frac{(Q_r.BOD_r + Q_w.BOD_w)}{Q_r + Q_w} \\ &= \frac{(0.710 \times 2.0 + 0.114 \times 341)}{0.710 + 0.114} = 49\ \text{mg/L} \end{aligned}$$

The ultimate BOD at the mixing point is obtained from Equation 3.22:

$$L_0 = BOD5_0.K_T = 49 \times 1.10 = 54\ \text{mg/L}$$

Example 3.2 **(*Continued*)**

c) Critical time (t_c)

From Equation 3.25:

$$t_c = \frac{1}{K_2 - K_1}.\ln\left\{\frac{K_2}{K_1}\cdot\left[1 - \frac{D_0.(K_2 - K_1)}{L_0.K_1}\right]\right\}$$

$$= \frac{1}{2.49 - 0.48}.\ln\left\{\frac{2.49}{0.48}\cdot\left[1 - \frac{1.6.(2.49 - 0.48)}{54 \times 0.48}\right]\right\} = 0.75\mathrm{d}$$

The critical distance is obtained from the critical time and the velocity:

$$d_c = t.v.86{,}400 = 0.75 \times 0.35 \times 86{,}400 = 22{,}680\ \mathrm{m} = 22.7\ \mathrm{km}$$

d) Critical concentration of the dissolved oxygen (DO_c)

The critical deficit is given by Equation 3.26:

$$D_c = \frac{K_1}{K_2}.L_0.e^{-K_1.t_c} = \frac{0.48}{2.49}.54.e^{-0.48 \times 0.75} = 7.2\mathrm{mg/L}$$

The critical concentration is given by Equation 3.27:

$$DO_c = C_s - D_c = 7.5 - 7.2 = 0.3\ \mathrm{mg/L}$$

Environmental control measures need to be adopted, since there are DO concentrations lower than the minimum allowable ($DO_{min} = 5.0$ mg/L).

In case a negative value of DO concentration had been calculated, one should always keep in mind that negative concentrations have no physical meaning. The Streeter–Phelps model would be no longer valid under these conditions (from the moment when DO = 0 mg/L), and the calculation and the graph must be interrupted at this point. However, even in this case the model played an important role, since the requirement for control measures was identified.

e) DO sag curve

Along the water course, in the absence of specific data, it is assumed that the dilution by natural contributions (direct drainage) is counterbalanced by the BOD load occasionally distributed along the reach (diffuse pollution).

In case there were significant tributaries or sewage discharges downstream, the water body should be subdivided into new reaches. It is an essential condition of the Streeter–Phelps model that each reach is homogeneous.

From Equation 3.24:

$$C_t = C_s - \left[\frac{K_1.L_0}{K_2 - K_1}.(e^{-K_1.t} - e^{-K_2.t}) + (C_s - C_0).e^{-K_2.t}\right]$$

$$= 7.5 - \left[\frac{0.48 \times 54}{2.49 - 0.48}.(e^{-0.48 \times t} - e^{-2.49 \times t}) + (7.5 - 5.9).e^{-2.49 \times t}\right]$$

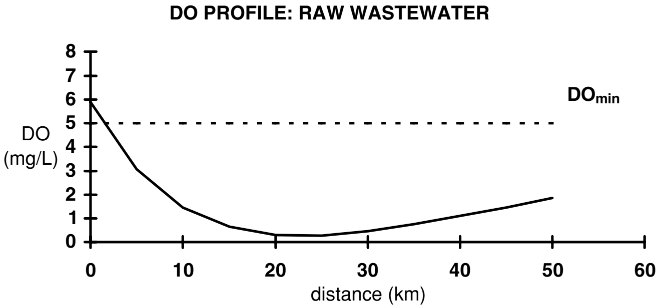
Example 3.2 (*Continued*)

For various values of t:

d (km)	t (d)	C_t (mg/L)
0.0	0.00	5.9
5.0	0.17	3.1
10.0	0.33	1.5
15.0	0.50	0.6
20.0	0.66	0.3
25.0	0.83	0.3
30.0	0.99	0.5
35.0	1.16	0.8
40.0	1.32	1.1
45.0	1.49	1.5
50.0	1.65	1.9

It can be observed that in practically all the distance, the DO is below the minimum allowable level of 5.0 mg/L. The DO profile can be visualised in Figure 3.16.

If a negative DO concentration had occurred, the model would stop being used at the point when DO became less than zero, and the negative values should not be reported or plotted.



DO profile in the river. Raw sewage.

Calculation of the output data – treated wastewater

After the confirmation of the need for wastewater treatment, the different alternatives of BOD removal efficiencies must be investigated. The concept of treatment level (primary, secondary) used in this example is covered in Chapter 4.

For the sake of simplicity, in this example it is assumed that the domestic and industrial wastewaters are mixed and treated together, at the same plant and, therefore, with the same removal efficiency. Other approaches are possible, involving different plants and treatment efficiencies if the domestic and industrial effluents are separated.

***Example 3.2* (*Continued*)**

a) *Alternative 1*: Primary treatment – Efficiency 35%

From Equation 3.28, the BOD of the treated wastewater is:

$$\mathrm{BOD_{tw}} = \mathrm{BOD_{rw}}.\left(1 - \frac{\mathrm{E}}{100}\right) = 341.\left(1 - \frac{35}{100}\right) = 222 \text{ mg/L}$$

The new coefficient $\mathrm{K_1}$ (treated wastewater at primary level) can be obtained from Table 3.3, and is adopted in this example as:

$$\mathrm{K_1} = 0.35\ \mathrm{d^{-1}}(\mathrm{T} = 20\,^\circ\mathrm{C})$$

$$\mathrm{K_1} = 0.44\ \mathrm{d^{-1}}(\text{after correction for T} = 25\,^\circ\text{C using Equation 3.8})$$

The remaining input data are the same. The calculation sequence is also the same.

The calculated and plotted DO values are shown in item d.

The critical DO concentration (2.8 mg/L) occurs at a distance of 22.1 km. The minimum allowable value (5.0 mg/L) continues not to be complied with in a large part of the river reach. The efficiency of the proposed treatment is insufficient. Therefore a higher efficiency must be adopted, associated with secondary treatment level.

b) *Alternative 2*: Secondary treatment – Efficiency 65%

All secondary-level sewage treatment processes reach a BOD removal efficiency of at least 65%, even the simplest ones. In this item, sewage treatment by UASB (Upflow Anaerobic Sludge Blanket) reactors is analysed.

The effluent BOD from the treatment plant is:

$$\mathrm{BOD_{tw}} = 341.\left(1 - \frac{65}{100}\right) = 119 \text{ mg/L}$$

The new coefficient $\mathrm{K_1}$ (treated wastewater at secondary level) can be obtained from Table 3.3, and is adopted in this example as:

$$\mathrm{K_1} = 0.18\ \mathrm{d^{-1}}(\mathrm{T} = 20\,^\circ\mathrm{C})$$

$$\mathrm{K_1} = 0.23\ \mathrm{d^{-1}}(\mathrm{T} = 25\,^\circ\mathrm{C})$$

It was assumed that the effluent DO from the treatment plant is zero (0.0 mg/L), since the effluent is anaerobic. If a different treatment process is adopted and the effluent has higher levels of DO in the effluent, this must be taken into consideration. Naturally, if only anaerobic reactors are adopted, aeration of the effluent must not be practised since hydrogen sulphide may be released into the atmosphere.

The calculated DO values and the graph of the DO profile are presented in item d.

Example 3.2 (*Continued*)

Along the whole length of the water course the DO values are above the minimum allowable concentration (the critical DO is 5.4 mg/L, greater than the minimum allowable of 5.0 mg/L). In this way, from the viewpoint of the receiving body, this alternative is *satisfactory.* Existing BOD discharge standards are not analysed here. These standards vary from country to country or from region to region and they can be taken into consideration when applicable. In the present case, the BOD of the discharge is 119 mg/L. In the case that the environmental agency establishes discharge standards for BOD of, say, 25 mg/L, these standards will not be satisfied in this alternative. Under certain conditions, environmental agencies relax the discharge standard, provided that the standard for the receiving body standard is satisfied.

Assuming that the environmental agency has accepted this alternative of 65% BOD removal efficiency, which has been shown to be sufficient in terms of DO, there is no need to investigate other alternatives of greater removal efficiencies, which probably would have higher costs. The most economic situation is usually that in which the critical DO is only marginally greater than the minimum allowable DO. This aspect is of great importance for developing countries. Similarly, there is no need to analyse efficiencies lower than 65%, since this is already on the lower boundary of typical efficiencies for secondary treatment level.

If the efficiency of 65% had been unsatisfactory, new efficiencies should be tried in a sequentially increasing way, until the receiving body standard is reached.

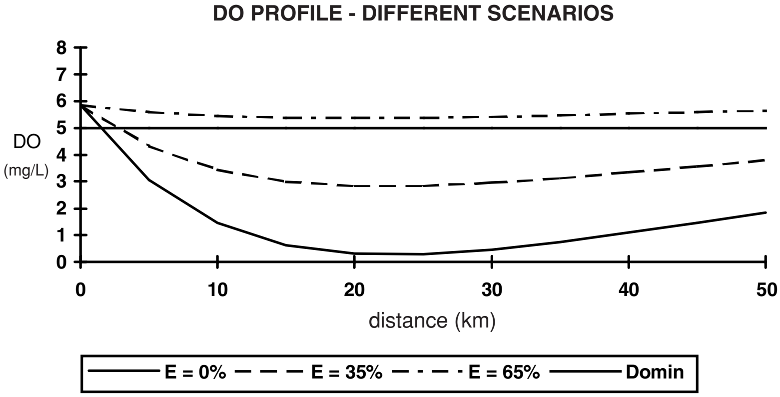
c) Summary

The alternative to be adopted is alternative 2 – sewage treatment at a secondary level, with a BOD removal efficiency of 65%.

The DO concentrations in the water body for the three scenarios are presented below.

		DO concentration (mg/L)		
d (km)	t (d)	E = 0%	E = 35%	E = 65%
0.0	0.00	5.9	5.9	5.9
5.0	0.17	3.1	4.3	5.6
10.0	0.33	1.5	3.5	5.5
15.0	0.50	0.6	3.0	5.4
20.0	0.66	0.3	2.8	5.4
25.0	0.83	0.3	2.8	5.4
30.0	0.99	0.5	3.0	5.4
35.0	1.16	0.8	3.1	5.5
40.0	1.32	1.1	3.4	5.5
45.0	1.49	1.5	3.6	5.6
50.0	1.65	1.9	3.8	5.7

Example 3.2 (*Continued*)



DO profiles for three different BOD removal efficiencies in the wastewater treatment.

The values above were obtained using a spreadsheet. Small differences in decimals may occur, depending on the criteria employed for rounding the values of the intermediate calculations, especially if they are performed using calculators.

3.3 CONTAMINATION BY PATHOGENIC MICROORGANISMS

3.3.1 Introduction

One of the most important aspects of water pollution is that related with public health, associated with water-borne diseases. This topic, including the main pathogens of interest and the concept of indicator organisms of faecal contamination, is discussed in Chapter 2.

A water body receiving the discharge of sewage may incorporate into itself a wide range of pathogenic organisms. This fact may not generate a direct impact on the aquatic organisms themselves, but may affect some of prevailing uses of the water, such as potable water supply, irrigation and bathing.

Therefore, it is very important to know the behaviour of the pathogenic organisms in the water body, starting from the discharge point until places where water is likely to be used. It is known that most of these agents have optimal conditions for their growth and reproduction in the human intestinal tract. Once submitted to the adverse conditions that prevail in the water body, they tend to decrease in number, characterising the so-called **decay**.

In Chapter 2 it was seen that the bacteria of the coliform group are used as **indicators of faecal contamination**; that is, they indicate if the water has been contaminated by faeces and, as a result, if it presents a potential for having pathogens

Table 3.8. Important factors that contribute to bacterial decay in water bodies

Physical factors	Physical-chemical factors	Biological and biochemical factors
• solar light (ultraviolet radiation) • temperature (values in water are usually lower than those in human bodies • adsorption • flocculation • sedimentation	• osmotic effects (salinity) • pH • chemical toxicity • redox potential	• lack of nutrients • predation • competition

and therefore transmitting diseases. The present item covers the qualitative and quantitative relations associated with the decay of the coliforms in water bodies. It is assumed that this decay represents, with a certain safety, an indication of the behaviour of the pathogens (especially bacteria) discharged into the water body.

3.3.2 Bacterial decay kinetics

3.3.2.1 Intervening factors

Coliforms and other microorganisms of intestinal origin present a natural mortality when exposed to environmental conditions that are different from the previously preponderant conditions inside the human system, which were ideal for their development and reproduction. Table 3.8 lists important factors that contribute to the bacterial decay in water bodies (Arceivala, 1981; EPA, 1985; Thomann and Mueller, 1987). These factors may act simultaneously and with different degrees of importance.

3.3.2.2 Kinetics of bacterial decay

The bacterial mortality rate is generally estimated by Chick's law, according to which, the higher the concentration of bacteria, the higher is the decay rate (first-order reaction):

$$\frac{dN}{dt} = -K_b.N \tag{3.33}$$

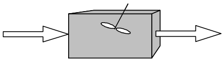
where:

N = number of coliforms (organisms /100 ml)

K_b = coefficient of bacterial decay (d^{-1})

t = time (d)

Table 3.9. Formulas for the calculation of the coliform concentrations in water bodies

Hydraulic regime	Scheme	Coliform concentration N (organisms/100ml)
Plug flow *(e.g.: rivers)*		$N = N_0.e^{-K_b.t}$
Completely mixed *(e.g.: lakes)*		$N = \frac{N_0}{1+K_b.t}$

N_0 = number of coliforms in the influent (organisms/100 ml). In plug-flow reactors, coliforms at time t = 0

N = number of coliforms after time t (organisms/100 ml)

K_b = coefficient of bacterial decay (d^{-1})

t = time (d)

The formula to calculate the coliform concentration after a time t depends on the hydraulic regime of the water body. Rivers are usually represented as plug-flow reactors, while reservoirs are frequently represented as completely-mixed reactors. These concepts, including the analysis of the other more realistic hydraulic models, are described in greater detail in Chapter 8. Depending on the characteristics of the water body, the formulas shown in Table 3 can be used.

For completely-mixed reactors, the time t corresponds to the detention time, given by: t = V/Q. The concentration of the coliforms at any point in the reactor is the same, coinciding with the effluent concentration.

3.3.2.3 Bacterial decay coefficient

Values of K_b obtained in various studies in fresh water vary within a wide range. Typical values, however, are close to (Arceivala, 1981; EPA, 1985; Thomann and Mueller, 1987):

$$\boxed{K_b = 0.5 \text{ to } 1.5\ d^{-1}\ (\text{base e}, 20\,^\circ C) \quad \text{Typical value} \approx 1.0\ d^{-1}}$$

The effect of temperature on the decay coefficient can be formulated as:

$$K_{b_T} = K_{b_{20}}.\theta^{(T-20)} \tag{3.34}$$

where:

θ = temperature coefficient (−)

A typical value for θ can be **1.07** (Castagnino, 1977; Thomann and Mueller, 1987), though there is a great variation in the data presented in the literature.

Table 3.10. Main processes for the removal of pathogenic organisms in wastewater treatment

Type	Process
Natural	Maturation ponds Land infiltration
Artificial	Chlorination Ozonisation Ultraviolet radiation Membranes

Note: for a description of the process – see Chapter 4

3.3.3 Control of the contamination by pathogenic organisms

The best measure to control contamination of a water body by pathogenic organisms from sewage is through their removal at the wastewater treatment stage. However, this approach is not practised throughout the world. In various countries there is systematic disinfection of the sewage treatment effluent, while in others disinfection is only carried out in the potable water treatment. However, in any case, approaches that preserve the defined uses of the water body should be adopted.

The wastewater treatment processes usually applied are very efficient in the removal of suspended solids and organic matter, but generally insufficient for the removal of pathogenic microorganisms. In spite of the great importance of this item in developing countries, it has not yet received due consideration. Table 4.9 in Chapter 4 lists the coliform removal efficiencies obtained in the main wastewater treatment systems. It should be always remembered that the coliforms are not a direct indication of the presence of pathogens, and they may represent only those organisms that have similar decay (or removal) mechanisms and similar (or higher) mortality rates. Protozoan cysts and helminth eggs are removed by different mechanisms (e.g. sedimentation) and are not well represented by coliforms.

Even though removal efficiencies of 90% shown in Table 4.9 may seem high, it should be borne in mind that, when dealing with coliforms, much higher efficiencies are generally necessary in order to have low concentrations in the water body, as a result of the very high concentrations in the raw sewage. High coliform removal efficiencies can be obtained by the processes listed in Table 3.10, which are further detailed in Chapter 4.

The processes listed above are capable of reaching coliform removal efficiencies of 99.99% or more. Frequently the coliform removal efficiency is expressed in a logarithmic scale, according to:

Removal efficiencies	
Log units	Percentage (%)
1	90
2	99
3	99.9
4	99.99

For instance, a coliform concentration, which is reduced from 10^7 organisms/100 ml to 10^4 organisms/100 ml, is reduced in 3 orders of magnitude, or 99.9%. If the logarithms of the concentrations are calculated, the reduction is from 7 to 4 units, in other words, 3 log units. Coliform concentrations are frequently represented in terms of the order of magnitude (powers of 10) or in their logarithms, considering their great variability and the uncertainty in more precise numerical values, and because coliform data usually tend to follow a log-normal distribution. The following formulas relate the efficiency expressed as percentage removal with log units removed.

$$\boxed{\text{Efficiency (\%)} = (N_0 - N)/N_0 = 100 \times (1 - 10^{-\text{log.units removed}})} \tag{3.35}$$

$$\boxed{\text{Log units removed} = -\log_{10}[1 - (\text{Efficiency (\%)}/100)]} \tag{3.36}$$

Not all countries or regions have coliform standards for the water body. When existent, they vary as a function of the water use and a number of local aspects. Values are usually situated around 10^2 to 10^3 faecal (thermotolerant) coliforms per 100 ml.

Example 3.3

Calculate the concentration profile of faecal (thermotolerant) coliforms in the river of Example 3.2. Calculate the coliform removal efficiency necessary in the wastewater treatment, so that the river presents a coliform concentration lower than 10^3 CF/100 ml.

Data:

- river flow: $Q_r = 0.710\ m^3/s$
- wastewater flow: $Q_w = 0.114\ m^3/s$
- water temperature: $T = 25\,°C$
- travel distance: $d = 50$ km
- velocity of the water: $v = 0.35$ m/s

Solution:

a) Faecal coliform concentration in the raw sewage

Adopt a faecal coliform concentration of $N_{rw} = 1 \times 10^7$ org/100 mL in the raw wastewater (see Chapter 2).

b) Faecal coliform concentration in the wastewater–river mixture, after the discharge

Assume that the river is clean upstream of the discharge, with a negligible concentration of coliforms ($N_r = 0$ organisms/100 mL).

***Example 3.3* (*Continued*)**

The concentration in the mixing point is calculated by a weighted average with the flows:

$$N_0 = \frac{Q_r.N_r + Q_w.N_{rw}}{Q_r + Q_w} = \frac{0.710 \times 0 + 0.114 \times 1 \times 10^7}{0.710 + 0.114}$$

$$= 1.38 \times 10^6 \,\text{org}/100\,\text{mL}$$

c) Concentration profile along the distance

The faecal coliform concentration is calculated by the equation for plug flow (rivers), presented in Table 3.9. Adopting $K_b = 1.0\ d^{-1}$ (20 °C), the value for the temperature of 25 °C is obtained:

$$K_{b_T} = K_{b_{20}}.^{(T-20)} = 1.0 \times 1.07^{(25-20)} = 1.40\,d^{-1}$$

The concentrations as a function of time are calculated from:

$$N = N_0.e^{-K_b.t} = 1.38 \times 10^6.e^{-1.4.t}$$

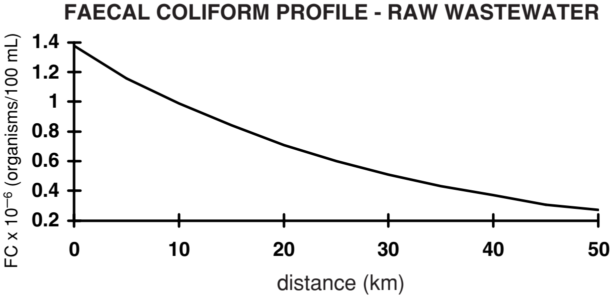
Varying t, the values of N_t are obtained. The correspondence between distance and time is given by:

$$d = v.t = (0.35\,m/s \times 86{,}400\,s/d).t/(1000\,m/l_2m$$

The table and graph below present N_t for various values of t and d:

d (km)	t (d)	N_t (organisms/100 mL)
0.0	0.00	1.38×10^6
5.0	0.17	1.09×10^6
10.0	0.33	8.69×10^5
15.0	0.50	6.89×10^5
20.0	0.66	5.47×10^5
25.0	0.83	4.34×10^5
30.0	0.99	3.44×10^5
35.0	1.16	2.73×10^5
40.0	1.32	2.17×10^5
45.0	1.49	1.72×10^5
50.0	1.65	1.36×10^5

***Example 3.3* (*Continued*)**



In spite of the considerable decrease along the travel distance, the concentrations are still very high and far greater than the desired value of 10^3 organisms/100 mL.

d) Maximum allowable concentration in the wastewater

At the discharge point, the faecal coliform concentration needs to be less than 1,000 organisms/100 mL. Using the equation for the concentration in the mixing point, the maximum desirable concentration in the treated wastewater is obtained.

$$N_0 = \frac{Q_r.N_r + Q_w.N_{tw}}{Q_r + Q_w} = 1,000 = \frac{0.710 \times 0 + 0.114 \times N_{tw}}{0.710 + 0.114}$$

$$N_{tw} = 7,228 \text{ organisms/100 mL}$$

e) Required removal efficiency of faecal coliforms in the wastewater treatment

The required efficiency is:

$$E = \frac{1.0 \times 10^7 - 7,228}{1.0 \times 10^7} = 0.9993 = 99.93\%$$

In log units, the removal efficiency is:

$$\text{Log units removed} = -\log_{10}[1 - (E(\%)/100)] = -\log_{10}[1 - 0.9993] = 3.15 \text{ log units}$$

Therefore, the high efficiency of 99.93% (3.15 log units) for the removal of faecal coliforms in the wastewater treatment will be required. Such a high efficiency is not usually reached in the conventional treatment processes, requiring a specific stage for coliform removal (see Table 3.10).

Example 3.4

Calculate the concentration of the faecal coliforms in a *reservoir* with a volume of 5,000,000 m^3. The reservoir receives, together, a river and a sewage discharge, both with the same characteristics as in Example 3.3. Calculate the necessary coliform removal efficiency in the wastewater treatment, so that the reservoir has faecal (thermotolerant) coliform concentrations less than or equal to 1000 FC/100 ml.

Data:

- river flow: $Q_r = 0.710\ m^3/s$
- wastewater flow: $Q_w = 0.114\ m^3/s$
- water temperature: $T = 25\,^\circ C$

Solution:

a) Faecal coliform concentration in the raw sewage

$N_{rw} = 1 \times 10^7$ organisms/100 mL (same as in Example 3.3).

b) Faecal coliform concentration in the wastewater–river mixture

$N_0 = 1.38 \times 10^6$ organisms/100 mL (same as in Example 3.3)

c) Detention time in the reservoir

Total influent flow to the reservoir:

$$Q = Q_r + Q_w = 0.710 + 0.114 = 0.824\,m^3/s$$

$$t = \frac{V}{Q} = \frac{5{,}000{,}000\,m^3}{(0.824\,m^3/s) \times (86.400\,s/d)} = 70.2\,d$$

d) Coliform concentration in the reservoir

Assuming a complete-mix model and a K_b value of 1.4 d^{-1} (equal to Example 3.3, for $T = 25\ ^\circ C$), the concentrations of coliforms in the reservoir and in the reservoir effluent are given by (see equation in Table 3.9):

$$N = \frac{N_0}{1 + K_b.t} = \frac{1.38 \times 10^6}{1 + 1.4 \times 70.2} = 13,900\,\text{organisms}/100\,\text{mL}$$

$$= 1.39 \times 10^4\,\text{organisms}/100\,\text{mL}$$

This value is above the desired standard of 1,000 organisms/100 mL.

Example 3.4 **(*Continued*)**

e) Maximum allowable concentration in the wastewater

Using the same equation for completely-mixed reactors:

$$N = \frac{N_0}{1 + K_b.t} = 1,000 = \frac{N_0}{1 + 1.4 \times 70.2}$$

$$N_0 = 99{,}280\ \text{organisms}/100\,\text{mL} = 9.93 \times 10^4\ \text{organisms}/100\,\text{mL}$$

At the sewage–river mixing point, the concentration must be 99.280 organisms/100 mL. Using the equation for the concentration in the mixture (weighted averages), the maximum desirable concentration in the sewage is obtained.

$$N_0 = \frac{Q_r.N_r + Q_w.N_w}{Q_r + Q_w} = 99.280 = \frac{0.710 \times 0 + 0.114 \times N_w}{0.710 + 0.114}$$

$$N_w = 717.603\ \text{organisms}/100\,\text{ml} = 7.18 \times 10^5 \text{organisms}/100\,\text{ml}$$

f) Required efficiency for coliform removal in the wastewater treatment

$$E = \frac{1.0 \times 10^7 - 7.18 \times 10^5}{1.0 \times 10^7} = 0.928 = 92.8\%$$

This efficiency is lower than the efficiency required in Example 3.3 but this is due to the high detention time in the reservoir (70.2 days) compared with the reduced time in the river (1.65 days). If both systems had the same detention time, the plug-flow reactor (river) would have been more efficient compared with the completely-mixed reactor (reservoir).

3.4 EUTROPHICATION OF LAKES AND RESERVOIRS

3.4.1 The eutrophication process

Eutrophication is the excessive growth of aquatic plants, either planktonic, attached or rooted, at such levels as to cause interference with the desired uses of the water body (Thomann and Mueller, 1987). As discussed in this chapter, the main stimulating factor is an excessive level of **nutrients** in the water body, principally nitrogen and phosphorus.

In this chapter, the water bodies under consideration are lakes and reservoirs. The process of eutrophication can also occur in rivers, though this is less frequent, owing to the environmental conditions being less favourable for the growth of algae and other plants, because of factors such as turbidity and high velocities.

The following description illustrates the possible sequence of the eutrophication process in a water body, such as a lake or reservoir (see Figure 3.16). The level of eutrophication is usually associated with the predominant land use and occupation in the catchment area.

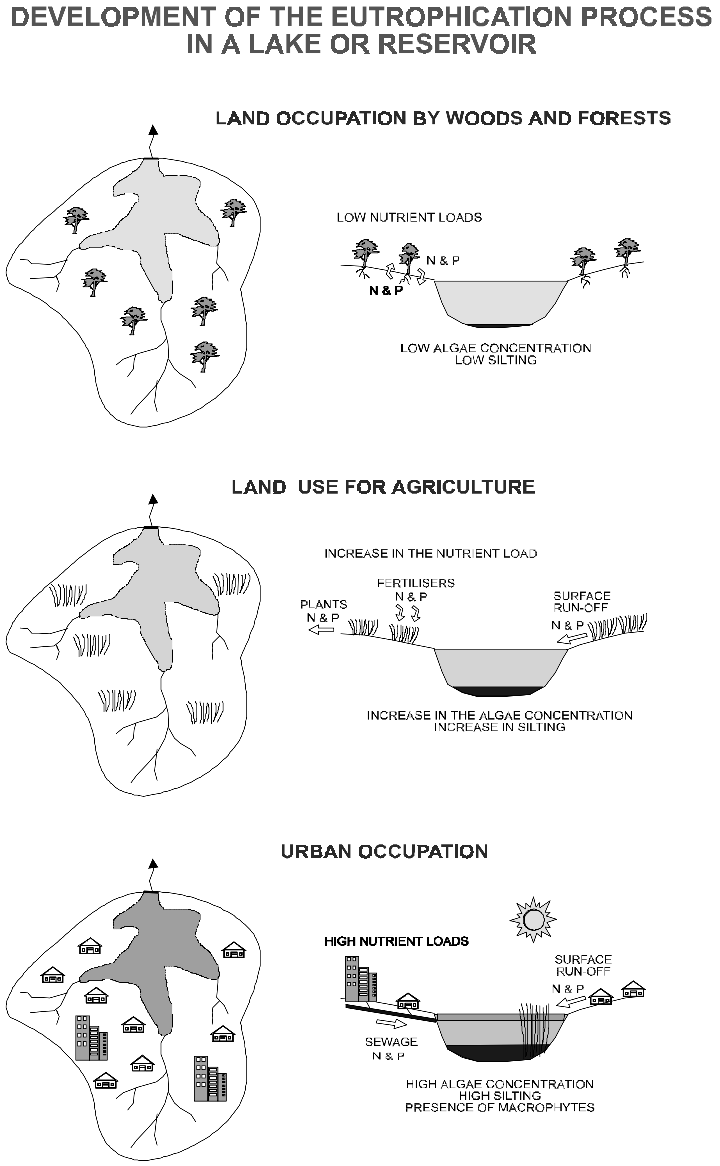


Figure 3.16. Sequence of the eutrophication process in a lake or reservoir. Association between land use and eutrophication.

a) Occupation by woods and forests

A lake situated in a catchment area occupied by woods and forests usually presents a low productivity; that is to say, there is little biological activity of production (synthesis) in it. Even in these natural conditions and in the absence of human interference, the lake tends to accumulate solids that settle, which form a layer of sludge at the bottom. With the decomposition of the settled material, there is a certain increase, still incipient, of the level of nutrients in the liquid mass. As a result, there is a progressive increase in the population of aquatic plants and, in consequence, of the other organisms situated at a higher level in the food chain.

In the catchment area, the larger part of the nutrients is retained within a nearly closed cycle. The plants die and, in the soil, are decomposed, releasing nutrients. In a region of woods and forests, the infiltration capacity of the rainwater into the soil is high. The nutrients then percolate into the soil, where they are absorbed by the roots of the plants, making part again of their composition and closing the cycle. The input of nutrients to the water body is small.

The water body still presents a low trophic level.

b) Agricultural occupation

The removal of natural vegetation from the catchment area for agricultural use generally leads to an intermediate stage in the deterioration process of the water body. The crops planted in the basin are harvested and transported for human consumption, probably outside the catchment area. With this, there is a removal of nutrients that is not naturally compensated, causing a break in the internal cycle. To compensate this removal and to make the agriculture more intensive, fertilisers containing high levels of nitrogen and phosphorus are added artificially. The farmers aim at guaranteeing a high production and thus add high quantities of N and P, frequently greater than the assimilative capacity of the plants.

The substitution of woods by plants for agricultural purposes can also cause a reduction in the infiltration capacity of the soil. Therefore the nutrients, already added in excess, are less retained and run off the soil until they eventually reach a lake or reservoir.

The increase in the nutrient level in the water body causes a certain increase in the number of algae and, in consequence, of the other organisms located at higher levels in the food chain, culminating with the fish. This relative increase in the productivity of the water body can even be welcome, depending on its intended uses, as would be the case, for instance, in aquaculture. The balance between the positive and negative aspects will depend, to a large extent, on the nutrient assimilative capacity of the water body.

c) Urban occupation

If an agricultural or forest area in a catchment area is substituted by urban occupation, a series of consequences should take place, this time at a faster rate.

- *Silting*. The implementation of housing developments implies land movement for the works. Urbanisation also reduces the water infiltration capacity into the soil. The soil particles tend to be transported to the lower parts of the catchment area until they reach the lake or reservoir. In these water bodies, they tend to settle, owing to the low horizontal velocities and turbulence. The sedimentation of the soil particles causes silting and reduces the net volume of the water body. The settled material also serves as a support medium for the growth of rooted plants of larger dimensions (macrophytes) near the shores. In spite of some ecological advantages (e.g. physical retention of pollutants, reduction of sediment resuspension, refuge for fishes and macroinvertebrates), these plants cause an evident deterioration in the visual aspect of the water body.

- *Urban stormwater drainage*. Urban drainage transports a far greater load of nutrients in comparison with the other types of occupation of the catchment area. This nutrient input contributes to a rise in the level of algae in the reservoir.
- *Sewage*. The greatest deterioration factor, however, is associated with wastewater originating from urban activities. The wastewater contains nitrogen and phosphorus present in faeces and urine, food remains, detergents and other by-products of human activity. The N and P contribution from sewage is much higher than the contribution originating from urban drainage.

Therefore, there is a great increase in the input of N and P onto the lake or reservoir, bringing as a result an elevation in the population of algae and other plants. Depending on the assimilative capacity of the water body, the algal population can reach very high values, bringing about a series of problems, which are described in the subsequent item. In a period with high sunshine (light energy for photosynthesis), the algae can reach superpopulations and be present at massive concentrations at the surface layer. This surface layer hinders the penetration of light energy for the lower layers in the water body, causing the death of algae situated in these regions. The death of these algae brings in itself a series of other problems. These events of superpopulation of algae are called **algal blooms**.

3.4.2 Problems of eutrophication

The following are the main undesired effects of eutrophication (Arceivala, 1981; Thomann and Mueller, 1987; von Sperling, 1994):

- *Recreational and aesthetic problems*. Reduction of the use of water for recreation, bathing and as a general tourist attraction because of the:
  - frequent algal blooms
  - excessive vegetation growth
  - disturbances with mosquitoes and insects
  - occasional bad odours
  - occasional fish mortality
- *Anaerobic conditions in the bottom of the water body*. The increase in productivity of the water body causes a rise in the concentration of heterotrophic bacteria, which feed on the organic matter from algae and other dead microorganisms, consuming dissolved oxygen from the liquid medium. In the bottom of the water body there are predominantly anaerobic conditions, owing to the sedimentation of organic matter and the small penetration of oxygen, together with the absence of photosynthesis (absence

of light). With the anaerobiosis, reducing conditions prevail, leading to compounds and elements being present in a reduced state:
  - iron and manganese are found in a soluble form, which may bring problems with the water supply;
  - phosphate is also found in a soluble form, and may represent an internal source of phosphorus for algae;
  - hydrogen sulphide may also causes problems of toxicity and bad odours.
- *Occasional anaerobic conditions in the water body as a whole.* Depending on the degree of bacterial growth, during periods of total mixing of the liquid mass (thermal inversion) or in the absence of photosynthesis (night time), fish mortality and the reintroduction of reduced compounds from the bottom onto the whole liquid mass could occur, leading to a large deterioration in the water quality.
- *Occasional fish mortality*. Fish mortality could occur as a result of:
  - anaerobiosis (mentioned above)
  - ammonia toxicity. Under conditions of high pH (frequent during periods of high photosynthetic activity), ammonia may be present in significant amounts in its free form (NH_3), toxic for the fish, instead of the ionised form (NH_4^+), which is non-toxic.
- *Greater difficulty and increase in the costs of water treatment*. The excessive presence of algae substantially affects the treatment of water abstracted from a lake or reservoir, due to the necessity of:
  - removal of the algae themselves
  - colour removal
  - taste and odour removal
  - higher consumption of chemical products
  - more frequent filter backwashings
- *Problems with industrial water supply*. Elevation in the costs of industrial water supply due to reasons similar to those already mentioned, and also to the presence of algae deposits in cooling waters.
- *Water toxicity*. Impairment of water for human and animal supply because of the presence of toxic secretions from cyanobacteria (cyanotoxins).
- *Alteration in the quality and quantity of commercial fish*.
- *Reduction in navigation and transport capacity*. The excessive growth of rooted macrophytes interferes with navigation, aeration and transport capacity of the water body.
- *Negative interference in the equipment for energy generation* (macrophytes in turbines).
- *Gradual disappearance of the lake.* As a result of eutrophication and silting, there is an increase in the accumulation of material and vegetation, and the lake becomes progressively shallower until it disappears. This tendency of disappearing (conversion to swamps or marshes) is irreversible, although usually extremely slow. With human interference, the process can accelerate abruptly. In case there is no control in the source or the dredging of the sediments, the water body could disappear relatively quickly.

Table 3.11. Trophic characterisation of lakes and reservoirs

	Trophic level				
Item	Ultraoligotrophic	Oligotrophic	Mesotrophic	Eutrophic	Hypereutrophic
Biomass	Very low	Low	Intermediate	High	Very high
Fraction of green algae and/or cyanobacteria	Low	Low	Variable	High	Very high
Macrophytes	Low or absent	Low	Variable	High or low	Low
Production dynamics	Very low	Low	Intermediate	High	High, unstable
Oxygen dynamics at the upper layer	Normally saturated	Normally saturated	Variable around supersaturation	Frequently supersaturated	Very unstable, from supersaturation to absence
Oxygen dynamics of the lower layer	Normally saturated	Normally saturated	Variable below saturation	Below saturation to complete absence	Very unstable, from supersaturation to absence
Impairment of multiple uses	Low	Low	Variable	High	Very high

Adapted from Vollenweider (cited by Salas and Martino, 1991)

3.4.3 Trophic levels

With the objective of characterising the stage of eutrophication of the water body, allowing the undertaking of preventative and/or corrective measures, it is interesting to adopt a classification system. In a simplified way, there are the following trophic levels:

- *oligotrophic* (clear lakes with a low productivity)
- *mesotrophic* (lakes with an intermediate productivity)
- *eutrophic* (lakes with a high productivity, compared with the natural basic level)

This classification can be further detailed, with the inclusion of other trophic levels, such as: ultraoligotrophic, oligotrophic, oligomesotrophic, mesotrophic, mesoeutrophic, eutrophic, eupolitrophic and hypereutrophic (from lowest to highest productivity).

A qualitative classification between the main trophic levels may be as presented in Table 3.11.

The quantification of the trophic level is, however, more difficult, especially for tropical lakes. Von Sperling (1994) presents a collection of various references, in terms of total phosphorus concentration, chlorophyll *a* and transparency, which shows the large amplitude in the ranges proposed by various authors. Besides this, the cited reference presents other possible indices to be used, always with the safeguard of the difficulty in generalising the data from one water body to another. It should be kept in mind that tropical water bodies present a larger capacity of phosphorus assimilation in comparison with water bodies in temperate climates. An interpretation of the synthesis reported by von Sperling may be as presented in Table 3.12 in terms of the total phosphorus concentration.

The establishment of the trophic levels based only on phosphorus is due to a mathematical modelling convenience. In the same way that in the other water pollution topics covered in the book, representative variables, such as dissolved oxygen (pollution by organic matter) and coliforms (contamination by pathogens),

Table 3.12. Approximate range of values of total phosphorus for the main trophic levels

Trophic level	Total phosphorus concentration in the reservoir (mgP/m^3)
Ultraoligotrophic	<5
Oligotrophic	<10–20
Mesotrophic	10–50
Eutrophic	25–100
Hypereutrophic	>100

Source: table constructed using the data presented by von Sperling (1994)

Note: the overlapping of values between two ranges indicates the difficulty in establishing rigid ranges

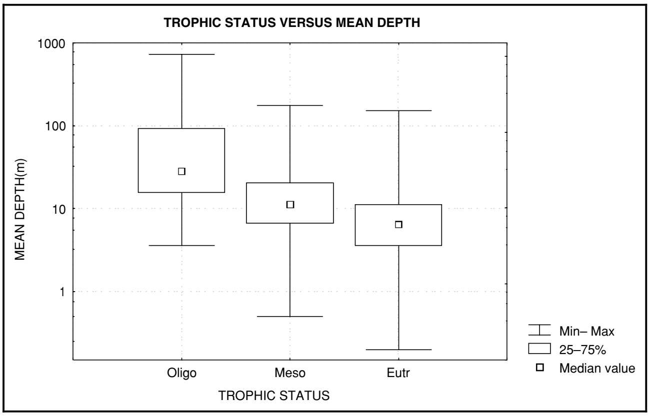


Figure 3.17. Box-and-whisker plot of mean depth for three groups of trophic levels. *Source:* von Sperling et al (2002) – 269 lakes and reservoirs. Note: mean depth: lake volume / area of the lake

phosphorus is adopted as a representative of the trophic level in this chapter. However, as seen in Section 3.4.5, there are some situations in which nitrogen controls the eutrophication process.

The researcher should always be open to include other variables in the analysis, in order to get a picture as close as possible to the actual behaviour of the water body under study. Von Sperling et al (2002), analysing data from more than 1,500 lakes and reservoirs around the world, investigated the correlation between morphometric variables and trophic level (using a subset of 269 water bodies, which had information on trophic level). From the statistical analysis it was clear that depth (mean depth, maximum depth and relative depth) is the morphometric variable most closely related with the trophic status: the shallower the water body, the greater the tendency for having a higher trophic level, mainly because of a higher light penetration over the full water body. Figure 3.17 shows a resulting box-and-whisker plot for the mean depth, where this association is clearly seen.

The association between trophic levels and water uses is shown in Table 3.13.

3.4.4 Dynamics of lakes and reservoirs

The vertical temperature profile in lakes and reservoirs varies with the seasons of the year. This temperature variation affects the density of the water and as a result the mixing and stratification capacity of the water body.

During warm periods, the temperature at the surface layer is much higher than the temperature at the bottom, because of solar radiation. Owing to this fact, the density of the water at the surface is lower than the density of the bottom layer,

Table 3.13. Association between trophic levels and water uses in a water body

	Trophic level					
Use	Ultraoligotrophic	Oligotrophic	Mesotrophic	Mesoeutrophic	Eutrophic	Hypereutrophic
Drinking water supply		Desirable	Acceptable			
Process water supply			Desirable	Acceptable		
Cooling water supply					Acceptable	
Primary contact recreation			Desirable	Acceptable		
Secondary contact recreation			Desirable		Acceptable	
Landscaping				Acceptable		
Fish culture (sensitive species)			Desirable	Acceptable		
Fish culture (tolerant species)					Acceptable	
Irrigation						Acceptable
Energy production						Acceptable

Source: adapted from Thornton e Rast (1994) which causes the existence of distinct layers in the water body:

- *epilimnion*: upper, warmer, less dense, with higher circulation
- *thermocline*: transition layer
- *hypolimnion*: bottom layer, colder, denser, with greater stagnation

The difference in densities can be such as to cause a complete **stratification** in the water body, with the three layers not mixing with each other. This stratification has a great influence on the water quality. Depending on the trophic level of the water body, a complete absence of dissolved oxygen in the hypolimnion can occur. As a result, in this layer there will be a predominance of iron, manganese and other compounds in their reduced forms.

With the arrival of a cold period, the upper layer in the lake cools, causing a certain homogenisation in the temperature along the depth. With the homogenisation of the temperature, there is a greater similarity in the water densities. The upper layer, suddenly cooled, tends to go to the bottom of the lake, dislodging the bottom layer and causing a complete turn over in the lake. This phenomenon is called thermal inversion or **turn over**. In lakes that present a high concentration of reduced compounds in the hypolimnion, the reintroduction of these compounds into the whole liquid mass of the lake can cause a large deterioration in the water quality. The decrease in dissolved oxygen due to the reintroduced demand by the reduced organic and inorganic compounds, together with the resuspension of the anaerobic bottom layer, can cause fish mortality.

Figure 3.18 presents typical temperature and DO profiles under conditions of stratification and turn over.

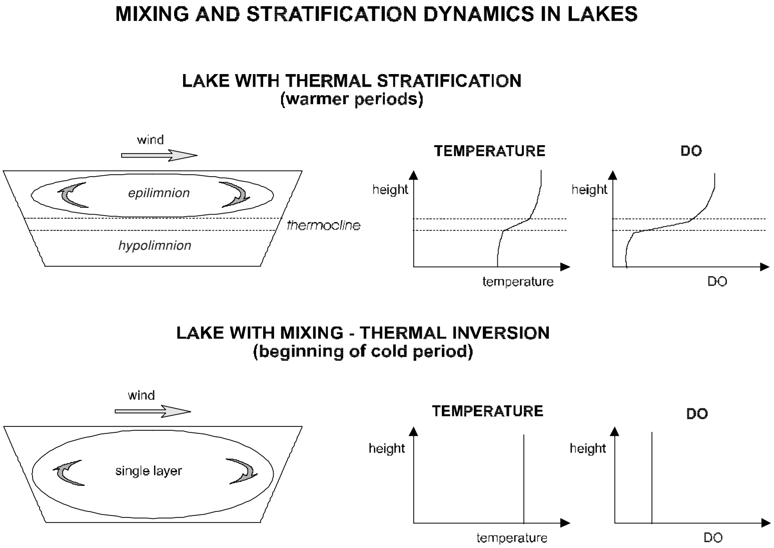


Figure 3.18. Typical temperature and DO profiles in a lake under stratification and turn-over conditions.

Lakes and reservoirs may present different mixing patterns and frequencies (von Sperling, 1999; Dantas, 2000):

a) **Holomictic lakes**. *Complete circulation over the whole water column*. Depending on the number of circulations per year, they can be classified as:
    - **Monomictic**. *One circulation per year*. Usually located where there are clear seasonal variations. Two types:
        - **Warm monomictic**. *One circulation in winter*. Located in temperate regions, high-altitude subtropical regions and in tropical regions.
        - **Cold monomictic**. *One circulation in summer*. Located in subpolar regions and high-altitude regions in temperate climates.
    - **Dimictic**. *Two circulations per year, one in spring, one in autumn*. Located in temperate climates. Warm months: stratification; autumn: cooling of upper layer and mixing. Cold months: ice cover; spring: ice melting, wind-induced mixing.
    - **Oligomictic**. *Few circulations per year*. Usually deep lakes in wet tropics, where there is little seasonal variation. Warm water along the water column.
    - **Polimictic**. *Many circulations per year*. Usually shallow lakes with daily circulations, unprotected from wind action, and located in warmer regions. Influence from daily temperature variations. Day hours: stratification. Night hours: cooling of upper layer and mixing.

b) **Meromictic lakes**. *Circulation does not occur at the whole water column*. Bottom layer (monimolimnion): stagnated due to high concentration of dissolved substances. Little influence from temperature.

c) **Amictic lakes**. *No circulation*. Usually ice-covered lakes at very high altitudes in equatorial regions or high latitudes.

For warm regions (the main focus of this book), the prevailing mixing patterns are either *warm monomictic* or *polimictic*. The variables that most significantly affect the mixing pattern are those related with depth (mean depth, maximum depth, relative depth) (von Sperling et al, 2002).

3.4.5 Limiting nutrient

The **limiting nutrient** is the one that, being essential for a certain population, limits its growth. According to Liebig's law, a limiting nutrient is the one whose concentration is closest to the minimum related to the organism's demand. With low concentrations of the limiting nutrient, the population growth is low. With an increase in the limiting nutrient concentration, the population growth also increases. This situation persists until the point in which the concentration of this nutrient starts to be so high in the system, that another nutrient starts to be the new limiting factor, since it is not present at concentrations sufficiently high for the requirements of the large population. This nutrient is now the new limiting nutrient, because there is no impact in increasing the concentration of the first nutrient, since the population will not rise, because it will be limited by the insufficiency of the new limiting nutrient.

Thomann and Mueller (1987) suggested the following criterion, based on the ratio between the nitrogen and phosphorus concentrations (N/P), in order to make a preliminary estimate of whether the algal growth is being controlled by phosphorus or nitrogen.

- large lakes, with a predominance of diffuse sources: **N/P** >> **10**: *limited by phosphorus*
- small lakes, with a predominance of point sources: **N/P** << **10**: *limited by nitrogen*

According to Salas and Martino (1991), most of the tropical lakes in Latin America are limited by phosphorus. Another aspect is that, even if the external input of nitrogen is controlled, there are organisms (cyanobacteria) which are capable of fixing atmospheric nitrogen. These organisms would be not reduced in numbers with the decrease in the influent load of nitrogen. Because of this, *usually a larger priority is given to the control of the phosphorus sources when the eutrophication of a lake or reservoir needs to be controlled*. The present text follows this approach.

3.4.6 Estimation of the phosphorus load into a lake or reservoir

The main sources of phosphorus to a lake or reservoir are, in increasing order of importance:

- *Stormwater drainage*
    - Areas with woods and forests
    - Agricultural areas
    - Urban areas
- *Wastewater*

Stormwater drainage from areas with ample vegetation coverage, such as woods and forests, transports a lower quantity of phosphorus. In these areas, phosphorus is not in excess in the environment, since the ecosystem is close to an equilibrium, without having large excesses or scarcities of the main elements.

Drainage from agricultural areas leads to higher and more variable P loads, depending on the soil retention capacity, irrigation, type of fertilisers and climatic conditions.

Urban drainage is associated with the highest loads. Domestic sewage transported by waterborne sewerage systems is usually the greatest source of phosphorus. Phosphorus can be found in human wastes, household detergents and other by-products of human activities. Regarding industrial wastewater, the generalisation of its contribution is difficult because of the variability of the various industrial wastewaters, even within the same industrial processing activity.

Table 3.14. Typical values of unit phosphorus contributions

Source	Type	Typical values	Unit
Drainage	Areas of woods and forests	10	$kgP/km^2.year$
	Agricultural areas	50	$kgP/km^2.year$
	Urban areas	100	$kgP/km^2.year$
Domestic sewage	Domestic	0.5	kgP/inhab.year

Note: values may vary widely from place to place; data presented are only references of orders of magnitude

Table 3.14 presents typical values of the unit phosphorus contribution, compiled from various references (von Sperling, 1985). The unit of time adopted is "year", convenient for modelling of P in lakes. Naturally the values can vary widely, from place to place. However, the values presented aim only to show an order of magnitude of the typical values.

3.4.7 Estimation of the phosphorus concentration in the water body

Literature presents a series of simplified empirical models to estimate the phosphorus concentration in a water body, as a function of influent load, detention time and geometric characteristics. The empirical models can be applied in any of the following applications:

- *Estimation of the trophic level*. Once the phosphorus concentration in the water body has been estimated, the trophic level of the lake can be evaluated, based on the considerations of Section 3.4.3.
- *Estimation of the maximum allowable load*. The maximum allowable P load into the lake can be estimated, such that the resulting P concentration is lower than a maximum desired value (e.g. a concentration that characterises eutrophic conditions).

The empirical approach has been more applied for planning than conceptual and more sophisticated models, because of the difficulty in structuring and obtaining the coefficients and input data necessary for these models.

The most widely known empirical model is that proposed by Vollenweider (1976), which has been developed, however, for temperate climatic conditions. The model, presented in a convenient form for the present text, is:

$$\boxed{P = L.10^3 \big/ \left[V.\left(\tfrac{1}{t} + K_s\right)\right]} \qquad (3.37)$$

where:

P = phosphorus concentration in the water body (gP/m^3)

L = influent phosphorus load (kgP/year)

V = volume of the reservoir (m^3)
t = hydraulic detention time (year)
K_s = loss coefficient of P by sedimentation (1/year)

Vollenweider obtained the value for K_s by regression analysis with the detention time in some reservoirs. The obtained value was:

$$K_s = 1/\sqrt{t} \qquad (3.38)$$

Castagnino (1982), theoretically analysing the P loss by sedimentation in **tropical lakes,** found a value of K_s equal to 2.5 times the value of Vollenweider. This magnifying factor of 2.5 is composed by a factor of 1.3 for the faster sedimentation at higher temperatures and a factor of 1.9 for the faster phytoplankton growth rate ($1.3 \times 1.9 = 2.5$). According to Castagnino, the corrected K_s value for tropical conditions is:

$$K_s = 2.5/\sqrt{t} \qquad (3.39)$$

Salas and Martino (1991), analysing experimental data from 40 lakes and reservoirs in Latin America and the Caribbean obtained, by regression analysis, the following relation for K_s:

$$\boxed{K_s = 2/\sqrt{t}} \qquad (3.40)$$

With the values obtained by Salas and Martino (1991), the *P concentration* in the reservoir becomes:

P concentration in the reservoir:

$$\boxed{P = L.10^3 \Big/ \left[V.\left(\frac{1}{t} + \frac{2}{\sqrt{t}}\right)\right]} \qquad (3.41)$$

Equation 3.41 can be arranged to lead to the maximum allowable *P load* into a lake or reservoir, so as not to surpass a maximum desired phosphorus concentration in the water body.

Maximum allowable P load:

$$\boxed{L = P.V.\left(\frac{1}{t} + \frac{2}{\sqrt{t}}\right) \Big/ 10^3} \qquad (3.42)$$

To use Equation 3.42, L must be estimated so that P is below the limit for eutrophic conditions. From Table 3.12, the P concentration range in an eutrophic water body is 25 to 100 mgP/m^3 or 0.025 to 0.100 gP/m^3. The establishment of a more relaxed or restrictive value for P must be done for each case, analysing the multiple uses of the reservoir and its level of importance.

Because of the fact that it was developed based on tropical water bodies, the empirical model proposed by Salas and Martino (1991) is probably the most adequate to be used for planning and management of lakes and reservoirs in warm-climate regions. Naturally, the critical view and experience of the researcher must always be present, to avoid distortions, given the specificity of each reservoir and lake under study.

3.4.8 Control of eutrophication

The control strategies usually adopted can be classified into two broad categories (Thomann and Mueller, 1987; von Sperling, 1995a):

- *Preventive measures* (action in the catchment area)
  - Reduction of external sources
- *Corrective methods* (action in the lake or reservoir)
  - Mechanical processes
  - Chemical processes
  - Biological processes

a) Preventive methods

Preventive methods, which comprise the reduction of the phosphorus input to the lake by acting on external sources, can include strategies related to the wastewater or to the stormwater drainage. The control strategies for wastewater are illustrated in Figure 3.19.

Control of wastewater

- Wastewater treatment with nutrient removal (tertiary treatment)
- Sewage diversion to downstream of the reservoir, associated with conventional (secondary) wastewater treatment
- Exportation of the wastewater to another catchment area without lakes or reservoirs, followed by conventional wastewater treatment
- Land infiltration of the wastewater

Control of stormwater drainage

- Control of the land use and occupation in the catchment area
- Protective green belts along the reservoir and its tributaries
- Construction of P retention reservoirs upstream of the main reservoir

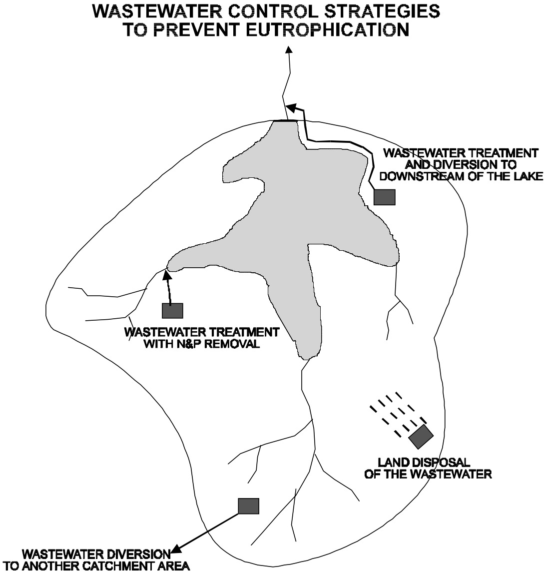


Figure 3.19. Wastewater control strategies aiming at the prevention of nutrient inputs into the reservoir.

Regarding wastewater treatment with phosphorus removal, this can be undertaken by biological and/or physical–chemical processes.

Biological phosphorus removal from the wastewater is currently consolidated and undertaken in many countries. The process is based on alternating between aerobic and anaerobic conditions, a situation which makes a certain group of microorganisms (phosphate accumulating organisms) assimilate a higher quantity of phosphorus than would be required in their usual metabolic processes. When removing these bacteria from the system in the biological excess sludge, the phosphorus absorbed by them is also removed. With biological P removal, effluents with concentrations of 0.5 mgP/L can be reached, although it is more appropriate to consider a more conservative value of 1.0 mgP/L. Chapters 35 and 36 cover in detail the process of biological P removal.

Phosphorus removal by *physical–chemical processes* is based on the precipitation of phosphorus after the addition of aluminium sulphate, ferric chloride or lime. The consumption of chemical products and the sludge generation are high. Physical–chemical polishing after biological P removal can generate effluents with concentrations in the order of 0.1 mgP/L.

b) Corrective methods

Corrective methods that can be adopted can include one or more of the strategies listed in Table 3.15 (von Sperling, 1995a).

Whenever possible, greater emphasis should be given to preventive measures, usually cheaper and more effective.

Table 3.15. Corrective methods for the recovery of lakes and reservoirs

Processes	Technique	Characteristics
Mechanical	Hypolimnetic aeration	• Injection of compressed air or oxygen into the bottom layers of the lake, promoting the stabilisation of the organic matter accumulated at the bottom and avoiding the release of nutrients from the sediments. • Presents high operational costs and requires the acquisition of specialised equipment, but is a widely applied and highly efficient technique.
	Destratification	• Consists of the injection of compressed air or oxygen into the bottom layers of the lake, leading to the circulation of the whole water body. • Use of simpler equipment. • Presents as inconvenient the transportation of reducing compounds to the upper layer, leading to the fertilisation of the epilimnion.
	Removal of deep waters	• Aims at the removal and substitution of deep waters by upper-layer waters, richer in oxygen, reducing the accumulation of nutrients in the hypolimnion. • The volume of liquid removed through hydrostatic pressure or pumping can be used for irrigation or directed to a WWTP.
	Addition of water with a higher quality	• Dilution technique that reduces the nutrient concentration in the water body. • Its application limits the formation of hydrogen sulphide in the hypolimnion, thus avoiding fish mortality.
	Sediment removal	• Upper layers of the sediment are removed by dredging, favouring the exposure of layers with a lower polluting potential. • The sludge removed, after treatment, can be used as a soil conditioner.
	Covering of the sediment	• Corrective method to avoid the release of nutrients in the deeper layers. • The sediment is isolated from the rest of the water body by covering it with plastic material or finely particulated substances. • Expensive method which presents difficulties in its installation.
	Removal of the aquatic macrophytes	• Aquatic macrophytes, which, when present in excessive numbers may interfere with various water uses, can be removed by manual or mechanical processes.
	Removal of the planktonic biomass	• The planktonic biomass, which presents a large pollutant storage capacity, may be removed by centrifuging or by the use of microsieves.
	Shading	• Acts against the excessive vegetation growth, by reducing the level of solar radiation received, by means of: – tree planting in the shores of small water bodies – installation of panels in the shores – application of supernatant material or light dyes at the surface layer

(*Continued*)

Table 3.15 (*Continued*)

Processes	Technique	Characteristics
Chemical	Chemical precipitation of phosphorus	• Recommended in the case of diffused sources of phosphorus, in which the removal of nutrients is impractical.
	Oxidation of the sediment with nitrate	• Efficient for the reduction of the internal fertilisation problem. • Avoids the excessive decrease of the oxygen concentration in deep waters.
	Application of herbicides	• Avoids excessive vegetation growth. • Associated with problems of toxicity, taste and odour and bioaccumulation.
	Application of lime	• Used for the sediment disinfection and to eliminate algae and submerged plants in small water bodies, and also for the neutralisation of the water in acidic lakes.
Biological	Use of fish that feed on the plants	• Reduces the plant community because of the activity of herbivorous fish.
	Use of cyanophages	• Reduces the density of cyanobacteria, by the attack of specific viruses. • Little employed.
	Manipulation of the food chain	• Reduces the phytoplanktonic community by the increase of the zooplanktonic population

Source: von Sperling (1995a)

Example 3.5

Estimate the trophic level of a reservoir based on the phosphorus concentration. In case eutrophic conditions are identified, estimate the maximum allowable P load so that eutrophic conditions are avoided.

Data:

- Reservoir volume: 10×10^6 m^3
- Average influent flow (tributaries + wastewater): 50×10^6 m^3/year
- Drainage area: 60 km^2
  - Woods and forests: 40 km^2
  - Agriculture: 10 km^2
  - Urban area: 10 km^2
- Contributing population (connected to the sewerage system): 16,000 inhabitants
- Wastewater characteristics: raw domestic sewage

Solution:

a) Estimation of the influent P load into the reservoir

Adopting the unit load values proposed in Table 3.14, the influent loads are:

- Raw domestic sewage: 16,000 inhab. × 0.5 kgP/inhab.year = 8,000 kgP/year

Example 3.5 (Continued)

- Drainage from the wooded areas: 40 km^2 × 10 $kgP/km^2.year$ = 400 kgP/year
- Drainage from the agricultural areas: 10 km^2 × 50 $kgP/km^2.year$ = 500 kgP/year
- Drainage from the urban areas: 10 km^2 × 100 $kgP/km^2.year$ = 1,000 kgP/year

Total influent load into the reservoir: $8{,}000 + 400 + 500 + 1{,}000 = 9{,}900$ kgP/year

b) Estimation of the hydraulic detention time

The hydraulic detention time is given by:

$$t = \frac{V}{Q} = \frac{10 \times 10^6\,m^3}{50 \times 10^6\,m^3/year} = 0.20\,years$$

c) Estimation of the phosphorus concentration in the reservoir

Adopting the model of Salas and Martino (1991), Equation 3.41:

$$P = \frac{L.10^3}{V.\left(\frac{1}{t} + \frac{2}{\sqrt{t}}\right)} = \frac{9{,}900 \times 10^3}{10 \times 10^6.\left(\frac{1}{0.20} + \frac{2}{\sqrt{0.20}}\right)} = 0.105\,gP/m^3 = 105\,mgP/m^3$$

d) Evaluation of the trophic level of the reservoir

Based on the P concentration of 105 mgP/m^3 and on the interpretation of Table 3.12, the reservoir is in the borderline between eutrophy and hypereutrophy. Therefore, control methods are necessary, so that the lake does not present eutrophic conditions.

e) Reduction of the influent phosphorus load

Through the adoption of preventive methods of wastewater and stormwater control, the influent phosphorus load into the reservoir can be drastically reduced. The influent load must be reduced down to a value that is below the limit for eutrophic conditions. Using Table 3.12, a not very conservative value of 50 mgP/m^3 can be used as the limit between mesotrophy and eutrophy. Under these conditions, the maximum allowable phosphorus load into the reservoir is given by Equation 3.42:

$$L = P.V.\left(\frac{1}{t} + \frac{2}{\sqrt{t}}\right)/10^3 = 0.050 \times 10 \times 10^6.\left(\frac{1}{0.20} + \frac{2}{\sqrt{0.20}}\right)/10^3$$
$$= 4{,}736\ kgP/year$$

The influent load needs to be reduced from 9,900 kgP/year to 4,736 kgP/year. Integrated action between wastewater and stormwater control can reach this reduction without difficulty.

3.5 QUALITY STANDARDS FOR WASTEWATER DISCHARGES AND WATERBODIES

3.5.1 Introduction

This section presents a discussion on the establishment of quality standards. In the perspective of this book, these standards are an important topic in the prevention and control of the impacts of the discharges of wastewater, which are the main issue of this chapter. This section, based on von Sperling and Fattal (2001) and von Sperling & Chernicharo (2002), analyses the practical implementation of standards, with a special focus on developing countries.

The impact of the discharge of urban wastewater into rivers, lakes, estuaries and the sea is a matter of great concern in most countries. An important point in this scenario is the establishment of an adequate legislation for the protection of the quality of water resources, this being a crucial point in the environmental and public health development of all countries. Most *developed nations* have already surpassed the basic stages of water pollution problems, and are currently fine-tuning the control of micro-pollutants, the impacts of pollutants in sensitive areas or the pollution caused by drainage of stormwater. However, *developing nations* are under constant pressure, from one side observing or attempting to follow the international trends of frequently lowering the limit concentrations of the standards, and from the other side being unable to reverse the continuous trend of environmental degradation. The increase in the sanitary infrastructure can barely cope with the net population growth in many countries. The implementation of sanitation and sewage treatment depends largely on political will and, even when this is present, financial constraints are the final barrier to undermine the necessary steps towards environmental restoration and public health maintenance. Time passes, and the distance between desirable and achievable, between laws and reality, continues to enlarge.

Figure 3.20 presents a comparison between the current status of developed and developing countries in terms of actual effluent concentrations of a particular pollutant and its associated discharge standard. In most developed countries, compliance occurs for most of the time, and the main concern relates to occasional episodes of non-compliance, at which most of the current effort is concentrated. However, in most developing nations the concentrations of pollutants discharged into the water bodies are still very high, and efforts are directed towards reducing the distance to the discharge standards, and eventually achieving compliance.

An adequate legislation for the protection of public health and the quality of water resources is an essential tool in the environmental development of all countries. The transfer of written codes from paper into really practicable standards, which are used not merely for enforcement, but mainly as an integral part of the public health and environmental protection policy, has been a challenge for most countries.

Besides the water quality *requirements* (see Section 1.3) that represent in a generalised and conceptual way the desired quality for a water, there is the need to establish quality standards, supported by a legal framework. *Standards* must be

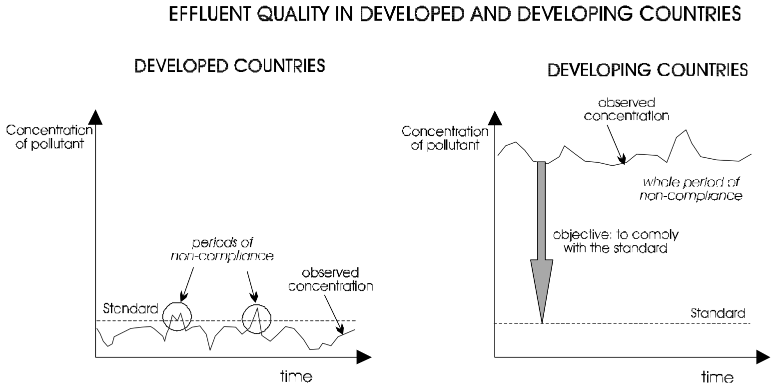


Figure 3.20. Comparison between developed and developing countries in terms of compliance to discharge standards. Source: von Sperling & Fattal (2001), von Sperling & Chernicharo (2002).

complied with, through enforcement of the legislation, by the entities involved the discharges and use of the water.

The following concepts are also important. *National standards* are defined by each country, have legal status and are based on the specific conditions of the country itself. Depending on the political structure of the country, *regional standards* may also be developed, for each state or other form of political division. Usually, regional standards are at least equal to national standards and are often more stringent or complete. *Guidelines* or recommendations are proposed by entities of wide acceptance (e.g. World Health Organization (WHO)), are generic by nature and usually aim at the protection of public health and environment in worldwide terms.

Economic, social and cultural aspects, prevailing diseases, acceptable risks and technological development are all particular to each country or region, and are better taken into account by the country or region itself, when converting guidelines into national/regional standards. This adaptation is crucial, and adequate consideration of the guidelines prior to the adoption of standards may be an invaluable tool in the health and environmental development of a country, whereas inadequate consideration may lead to discredit, frustration, unnecessary monetary expenditure, unsustainable systems and other problems. The setting of standards should be based on sound, logical, scientific grounds and should be aimed at achieving a measured or estimated benefit or minimising a given risk for a known cost (Johnstone and Horan, 1994).

In practical terms, there are the following types of standards or guidelines of direct interest to the topics of this book:

- *Discharge* (emission, effluent) standards
- Quality standards for the *receiving water body*
- Standards or quality guidelines for a certain *use of the treated effluent* (e.g.: irrigation)

3.5.2 Considerations about the development of discharge standards in developing countries

3.5.2.1 Typical problems with setting up and implementing standards in developing countries

Table 3.16 presents a list of common problems associated with setting up and implementing standards, especially in developing countries.

Table 3.16. Common problems associated with setting up and implementing standards, especially in developing countries

Problem	How it should be	How it frequently is
Guidelines are directly taken as national standards	Guidelines are general worldwide values. Each country should adapt the guidelines, based on local conditions, and derive the corresponding national standards.	In many cases the adaptation is not done in developing countries, and the worldwide guidelines are directly taken as national standards, without recognising the country's singularities.
Guideline values are treated as absolute values, and not as target values	Guideline values should be treated as target values, to be attained on a short, medium or local term, depending on the country's technological, institutional or financial conditions.	Guideline values are treated as absolute rigid values, leading to simple "pass" or "fail" interpretations, without recognising the current difficulty of many countries to comply with them.
Protection measures that do not lead to immediate compliance with the standards do not obtain licensing or financing	Environmental agencies should license and banks should fund control measures (e.g. wastewater treatment plants) which allow for a stepwise improvement of water quality, even though standards are not immediately achieved.	The environmental agencies or financial institutions do not support control measures which, based on their design, do not prove to lead to compliance with the standards. Without licensing or financing, intermediate measures are not implemented. The ideal solution, even though approved, is also not implemented, because of lack of funds. As a result, no control measures are implemented.
Standards are frequently copied from developed countries	National standards should be based on the country's specific economical, institutional, technological and climatic conditions.	National standards are frequently directly copied from developed countries' standards, either because of lack of confidence on their own capacity, desirability to achieve developed countries' status, lack of knowledge or poor knowledge transfer from international consulting companies. Cost implications are not taken into account. The standards become purely theoretical and are not implemented or enforced.

Table 3.16 (*Continued*)

Problem	How it should be	How it frequently is
Developed countries sometimes attempt to reach developed countries' status too quickly	If the guidelines and even the standards are treated as target values, time would be necessary to lead to compliance. Each country, based on the economic and technological capacity, should take the time that is reasonably necessary to achieve compliance. Developing countries are naturally likely to take more time than developed countries. Developing countries should understand that current standards in developed countries result from a long period of investment in infrastructure, during which standards progressively improved.	The desire to achieve developed countries' status too quickly can lead to the use of inappropriate technology, thus creating unsustainable systems.
Some standards are excessively stringent or excessively relaxed	Standards should reflect water quality criteria and objectives, based on the intended water uses.	In most cases, standards are excessively stringent, more than would be necessary to guarantee the safe use of water. In this case, they are frequently not achieved. Designers may also want to use additional safety factors in the design, thus increasing the costs. In other cases, standards are too relaxed, and do not guarantee the safe intended uses of the water.
There is no affordable technology to lead to compliance of standards	Control technologies should be within the countries financial conditions. The use of appropriate technology should be always pursued.	Existing technologies are in many cases too expensive for developing countries. Either because the technology is inappropriate, or because there is no political will or the countries' priorities are different, control measures are not implemented.
Compliance with standards are at a lower level of priority compared to other basic environmental sanitation needs	Each country, based on the knowledge of its basic conditions and needs, should set priorities to be achieved. If standards are well set up, they will naturally be integrated with the environmental control measures.	Basic water supply and sanitation needs are so acute in some countries, that standards are seen as an unnecessary sophistication.

(*Continued*)

Table 3.16 (*Continued*)

Problem	How it should be	How it frequently is
Standards are not actually enforced	Standards should be enforceable and actually enforced. Standard values should be achievable and allow for enforcement, based on existing and affordable control measures. Environmental agencies should be institutionally well developed in order to enforce standards.	Standards are not enforced, leading to a discredit in their usefulness and application, and creating the culture that standards are to remain on paper only.
Discharge standards are not compatible with water quality standards	In terms of pollution control, the true objective is the preservation of the quality of the water bodies. Discharge standards exist only by practical (and justifiable) reasons. However, discharge standards should be compatible with water quality standards, assuming a certain dilution or assimilation capacity of the water bodies.	Even if water quality standards are well set up, based on water quality objectives, discharge standards may not be compatible with them. Some parameters in the discharge standards may be too stringent and others too relaxed. In this case, different assimilation capacities of the water bodies are implicit. The aim of protecting the water bodies is thus not guaranteed.
Number of parameters are frequently inadequate (too many or too few)	The list of parameters covered by the national standards should reflect the desired protection of the intended water uses, without excesses or limitations.	In some countries, the standards include an excessively large list of parameters, many of which have no actual regional importance, are very costly to monitor or are not supported by satisfactory laboratory capabilities. In other situations, standards cover only a limited list of parameters, which are not sufficient to safeguard the intended water uses.
Monitoring requirements are undefined or inadequate	Monitoring requirements and frequency of sampling should be defined, in order to allow proper statistical interpretation of results. The cost implications for monitoring need to be taken into account in the overall regulatory framework.	In many cases, monitoring requirements are not specified, leading to difficulty in the interpretation of the results. In other cases, monitoring requirements are excessive and thus unnecessarily costly. Still in other cases, monitoring requirements are very relaxed, not allowing interpretation of results with confidence.

Table 3.16 (*Continued*)

Problem	How it should be	How it frequently is
Required percentage of compliance is not defined	It should be clear how to interpret the monitoring results and the related compliance with the standards (e.g. mean values, maximum values, absolute values, percentiles or other criteria).	The non specification of how to treat the monitoring results may lead to different interpretations, which may result in diverging positions as to whether compliance has been achieved or not.
Low standard values are sometimes below laboratory detection limits	If standards are treated as target values and are well linked with the water quality objectives, they should not be limited by current laboratory detection limits. In due time, laboratory techniques will improve and be consistent with the standard values.	Standards which are below the detection limit are sometimes seen as unjustifiable, which may be true in some cases, but not in many other cases.
There is no institutional development which could support and regulate the implementation of standards	The efficient implementation of standards requires an adequate infrastructure and institutional capacity to license, guide and control polluting activities and enforce standards.	In many countries the health and environmental agencies are not adequately structured or sufficiently equipped, leading to a poor control of the various activities associated with the implementation of standards.
Reduction of health or environmental risks due to compliance with the standards is not immediately perceived by decision makers or the population.	Decision makers and the population at large should be well informed about the benefits and costs associated with the maintenance of good water quality, as specified by the standards.	Decision makers are frequently more sensitive to costs than to benefits resulting from the implementation of control measures. The population is not well informed, and does not drive politicians and decision makers in order to invest in health and environmental protection.
Excessive expenditure on unjustifiably high standards may lead to population disagreement with really worthwhile standards	Standards should really reflect the water quality objectives, and these objectives should result from a consensus from the various segments of the society, directly involved in the catchment area.	Representatives of the society frequently do not participate in the decision-making process. High costs, which are not seen as bringing correspondingly high benefits, may lead to discredit and disagreement when aiming at implementing standards that are really important for the involved community.

Source: von Sperling & Fattal (2001), von Sperling & Chernicharo (2002).

3.5.2.2 Stepwise implementation of standards

Usually the stepwise implementation of a wastewater treatment plant is through the *physical expansion* of the size or number of units. A plant can have, for instance,

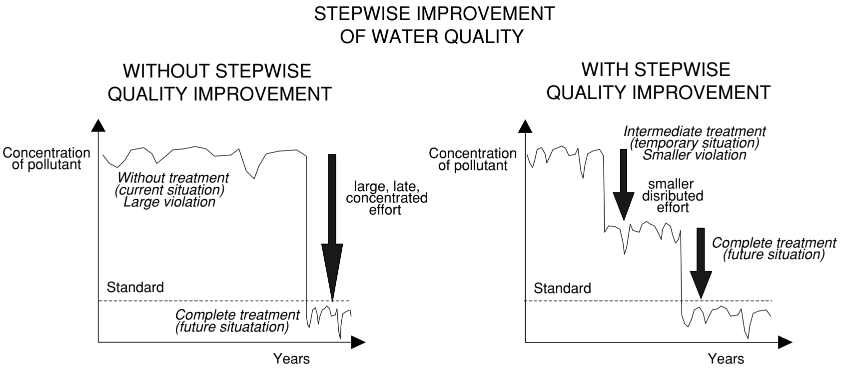


Figure 3.21. Concept of the stepwise improvement of water quality. Source: von Sperling & Fattal (2001), von Sperling & Chernicharo (2002).

two tanks built in the first stage, and another tank built in the second stage, after it has been verified that the influent load has increased, frequently due to the population growth. This stepwise implementation is essential, in order to allow reduction in present value construction costs.

However, another concept of stepwise implementation, which should be put in practice, especially in developing countries, is the *gradual improvement of the water or wastewater quality*. It should be possible, in a large number of situations, to implement in the first stage a less efficient process, or a process that removes less pollutants, transferring to a second stage the improvement towards a system more efficient or more wide-reaching in terms of pollutants. If the planning is well structured, the environmental agency could make allowances in the sense of permitting a temporary small violation in the standards in the first stage. Naturally a great deal of care must be exercised in not allowing that a temporary situation becomes permanent, which is a very common occurrence in developing countries. This alternative of stepwise development of water or wastewater quality is undoubtedly much more desirable than a large violation of the standards, whose solution is often unpredictable over time.

Figure 3.21 presents a typical situation concerning the implementation of wastewater treatment. If a country decides to implement treatment plants that can potentially lead to an immediate compliance with the standards, this will require a large and concentrated effort, since the current water quality is probably very poor, especially in developing countries. This large effort is naturally associated with a large cost. In most instances, the country cannot afford this large cost, and the plant construction is postponed and eventually never put into effect. On the other hand, if the country decides to implement only a partial treatment, financial resources may be available. A certain improvement in the water quality is obtained and health and environmental risks are reduced, even though the standards have not been satisfied. In this case, the standards are treated as target values, to be achieved whenever possible. The environmental agency is a partner in the solution of the problem, and establishes a programme of future improvements. After some time, there are additional funds for expanding the efficiency of the treatment plant, and the standards are finally satisfied. In this case, compliance with the standards is likely to be obtained before the alternative without stepwise implementation.

Not only wastewater systems should expand on a stepwise basis on developing countries, but also the standards for water quality. There should be a knowledge about the targets that are desired to be achieved over time, and these targets could eventually be the same as the general guidelines. However, with the standards the approach should be different, and the numeric values of the limit concentrations should progress stepwisely towards stringency. The standards should be adapted periodically, eventually reaching the same values as those in the guidelines.

The advantages of a stepwise implementation of standards and sanitary infrastructure are listed in Table 3.17.

An important issue in the stepwise approach is how to guarantee that the second and subsequent stages of improvement will be implemented, and not interrupted in the first stage. Because of financial restrictions, there is always the risk that the subsequent stages will be indefinitely postponed, under the argument that the priority has now shifted to systems that have not yet implemented the first stage. Even though this might well be a justifiable argument, it cannot be converted into a commonly used excuse. The environmental agency must set up scenarios of intervention targets with the entity responsible for the sanitary system. The scenarios should include the minimum intervention, associated with the first stage, and subsequent prospective scenarios, including required measures, benefits, costs and timetable. The formalisation of the commitment also helps in ensuring the continuation of the water quality improvement.

3.5.2.3 The principle of equity

The principle of equity means that all peoples, irrespective of race, culture, religion, geographic position or economic status are entitled to the same life expectancy and quality of life. Broadly speaking, the reasons for a lower quality of life are associated with environmental conditions, and if these improve, life quality is expected to rise accordingly. On this basis, there is no justification for accepting different environmental guideline values between developed and developing countries.

If guideline values (e.g. WHO guidelines) are treated as absolute values, than only developed countries are more likely to achieve them, and developing nations possibly will not be able to afford the required investments. However, if guideline values are treated as **targets**, than hopefully all countries will eventually be able to achieve them, some on a short, some on a medium and others only on a long term.

Figure 3.22 illustrates this point, for three different countries. For all of them, the guideline values are the same. The very developed country has been already compliant, and presents a better water quality than actually required. The developed country requires only a small effort and achieves compliance in a short term. The developing country requires a stepwise approach and achieves compliance only on a long term. However, in the end all countries will hopefully be compliant with the guidelines.

Table 3.17. Advantages of a stepwise implementation of standards and sanitary infrastructure

Advantage	Comment
Polluters are more likely to afford gradual investment for control measures	Polluters and/or water authorities will find it much more feasible to divide investments in different steps, than to make a large and in many cases unaffordable investment
The present value of construction costs is reduced	The division of construction costs into different stages leads to a lower present value than a single, large, initial cost. This aspect is more relevant in countries in which, due to inflation problems, interest rates are high.
The cost-benefit of the first stage is likely to be more favourable than in the subsequent stages	In the first stage, when environmental conditions are poor, usually a large benefit is achieved with a comparatively low cost. This means that already in the first stage a significant benefit is likely to be achieved, with only a fraction of the overall costs. In the subsequent stages, the increase of the benefit is not so substantial, but the associated costs are high. The cost-benefit is then less favourable.
There is more time and better conditions to know the wastewater characteristics	The operation of the system will involve monitoring, which, on its course, will allow a good knowledge of the wastewater characteristics. The design of the second or subsequent stages will be based on the actual characteristics, and not on generic values taken from the literature.
There is the opportunity to optimise operation, without necessarily making physical expansion	The experience in the operation of the system will lead to a good knowledge of its behaviour. This will allow, in some cases, the optimisation of the process (improvement of efficiency or capacity), without necessarily requiring the physical expansion of the system. The first stage will be analogous to a pilot plant.
There is time and opportunity to implement, in the second stage, new techniques or better developed processes	The availability of new or more efficient processes for wastewater treatment is always increasing with time. Process development is continuous and fast. The second or subsequent steps can make use of better and/or cheaper technologies, than it would be possible with a single step.
The country has more time to develop its own standards	As time passes, the experience in operating the system and evaluating its positive and negative implications in terms of water quality, health status and environmental conditions will lead to the establishment of standards that are really appropriate for the local conditions.
The country has more time and better conditions to develop a suitable regulatory framework and institutional capacity	Experience obtained in the operation of the system and in setting up the required infrastructure and institutional capacity for regulation and enforcement will also improve progressively, as the system expands on the second and subsequent stages.

Source: von Sperling & Fattal (2001), von Sperling & Chernicharo (2002).

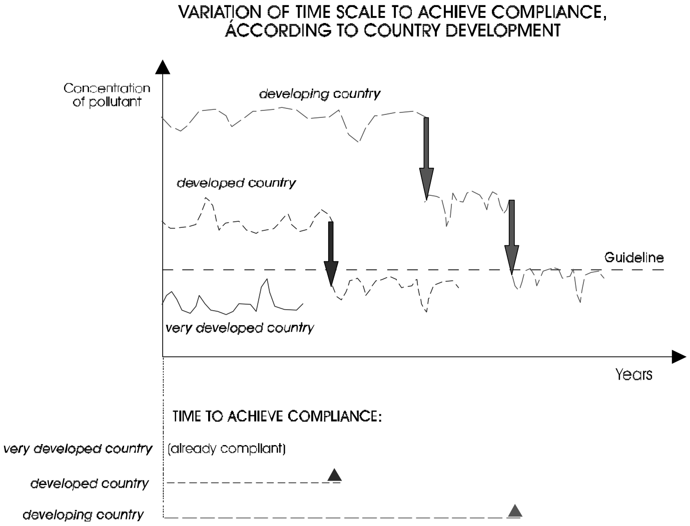


Figure 3.22. Variation of time scale to achieve guideline compliance, for a very developed country, a developed country and a developing country (in all cases, the guideline value is the same). Source: von Sperling & Fattal (2001).

3.5.2.4 Institutional development

An efficient implementation of standards must go in parallel with the development of the institutional framework necessary for monitoring, controlling, regulating and enforcing the standards. This topic is well discussed by Johnstone and Horan (1996) and some of the points are summarised below.

Institutional development takes time and the models cannot be directly copied from developed countries. Even though lessons should be learned from other countries that have already passed the basic steps of institutional development, an adaptation is also required in order to accommodate the countries' specific economic, cultural and social conditions. However, experience from other countries can help in structuring the organisations, especially when they are introduced for the first time. It must be recognised that institutional development is a continuous process, building on the experience of prior organisations.

Another important point is the need to separate the duties and responsibilities of regulating quality with those of achieving standards. This is especially true when private sector operators have to comply with standards.

The main points to be emphasised for developing countries are (Johnstone and Horan, 1996): (a) consider the process of institutional development and technical improvements to be long term; (b) build on past experiences; (c) separate regulatory and operational duties and responsibilities; (d) develop regulatory systems and procedures needed to enforce standards; (e) ensure that sufficient legal powers are in force; (f) recognise the costs of regulation and legal enforcement.

Table 3.18. European Community requirements for discharges from urban wastewater treatment plants

Parameter	Concentration	Minimum percentage of reduction (1)	Notes
BOD_5 (2)(3)	25 mg/L O_2	70–90 %	–
COD (3)	125 mg/L O_2	75 %	–
Total suspended solids	35 mg/L (4)	90 %	P.E. greater than 10,000 inhab.
	60 mg/L	70%	P.E. between 2,000 and 10,000 inhab.
	150 mg/L	–	For ponds effluents
Total nitrogen (5) (6)	10 mg/L ($^-$)	70–80	P.E. greater than 100,000 inhab.
	15 mg/L		P.E. between 10,000 and 100,000 inhab.
Total phosphorus (5)	1 mg/L	80	P.E. greater than 100,000 inhab.
	2 mg/L		P.E. between 10,000 and 100,000 inhab.

Source: Official Journal of the European Communities No. L 135/40 (Council of the European Communities, 1991).

P.E. = Population Equivalent (inhabitants)

BOD, COD and SS: the maximum number of samples that are allowed to fail the requirements is specified in the Directive

(1) Removal in relation to the load of the influent.

(2) The parameter can be replaced by another parameter: Total Organic Carbon (TOC) or Total Oxygen Demand (TOD) if a relationship can be established between BOD_5 and the substitute parameter.

(3) Analyses concerning discharges from ponds (lagoons) shall be carried out in filtered samples. However, the concentration of total suspended solids in unfiltered samples shall not exceed 150 mg/L.

(4) This requirement is optional.

(5) Total N and Total P: requirement for discharge in sensitive water bodies only (one or both parameters may be applied, depending on the local situation). Values are annual means.

(6) Alternatively, the daily average of Total Nitrogen must not exceed 20 mg/L (for water temperature of 12 °C or more).

3.5.3 Examples of standards and guidelines

3.5.3.1 Introduction

As commented, discharge standards vary from country to country and, in many cases, from state to state, reflecting their specificities, development stage, economical level, commitment of environmental protection and various other factors.

In any case, much more important than the discharge standards are the *quality standards for the water body*, because the quality in the water body is the one really associated with its uses. *Discharge standards* exist because of a practical aspect: it is easier for the environmental agencies to control, monitor and enforce point discharges, whose responsible agent is known. In a water body receiving multiple discharges, the occasional detection of non-compliance of the standards in the water body is not a trivial matter in terms of assigning those responsible. Therefore, discharge standards play an important role in most countries in the world.

Table 3.19. WHO recommended microbiological quality guidelines for treated wastewater reuse in agricultural irrigation

Category	Reuse conditions	Exposed group	Intestinal nematodes (eggs/L)$^{(b)}$ (arithmetic mean)	Faecal coliforms (FC/100 mL)$^{(c)}$ (geometric mean)
A	Irrigation of crops likely to be eaten uncooked, sports fields, public parks$^{(d)}$	Workers, consumers, public	≤ 1	$\leq 1000^{(d)}$
B	Irrigation of cereal crops, industrial crops, fodder crops, pasture and trees	Workers	≤ 1	No standard recommended
C	Localised irrigation of crops in category B if exposure to workers and the public does not occur	None	Not applicable	Not applicable

Source: WHO (1989).

(a) In specific cases, local epidemiological, sociocultural and environmental factors should be taken into account and the guidelines modified accordingly.
(b) *Ascaris* and *Trichuris* species and hookworms.
(c) During the irrigation period.
(d) A more stringent guideline limit ($\leq$ 200 faecal coliforms/100 mL) is appropriate for public lawns, such as hotel lawns, with which the public may come into direct contact.
(e) In the case of fruit trees, irrigation should cease two weeks before fruit is picked, and no fruit should be picked off the ground. Sprinkler irrigation should not be used.

This section presents some important standards and guidelines and suggestions to be applied for domestic sewage:

- European Community Directive concerning urban wastewater treatment
- WHO guidelines for treated-wastewater reuse in agricultural irrigation
- Possible discharge standards to be applied for domestic sewage

Various other important standards and guidelines exist, but their compilation is beyond the scope of this book.

3.5.3.2 European Community Directive for urban wastewater treatment

This section summarises the main requirements for urban (domestic + non-domestic) wastewater treatment plants in Europe – Council Directive 91/271/EEC, 21/05/1991 (Council of the European Communities, 1991). This directive specifies the minimum removal efficiencies and limit concentrations of BOD_5, COD, SS, N and P. Note that the values for N and P apply only when the discharge is to sensitive water bodies. The criteria for classifying a sensitive water body are presented in the legislation, but these are typically lakes, reservoirs, estuaries, bays and coastal waters, subject to certain conditions. The values presented in Table 3.18 for concentration or removal efficiency must apply.

Table 3.20. Possible discharge standards, according to different levels of stringency, for the main pollutants in domestic sewage

Parameter	Discharge to	Discharge standard (mg/L)		
		Less stringent	Stringent	Very stringent
BOD	Any water body	60	20–30	10
COD	Any water body	200	100–150	50
SS	Any water body	60	20–30	10
Total N	Sensitive water body	–	10–15	10
Total P	Sensitive water body	–	1–2	1

The directive also specifies (a) the minimum number of annual samples and (b) the maximum permitted number of samples that could fail to conform (for BOD, COD and SS).

3.5.3.3 WHO guidelines for the reuse of treated wastewater in agricultural irrigation

The WHO (1989) established microbiological quality guidelines for treated wastewater reuse in agricultural irrigation (Table 3.19). In these guidelines, two types or indicator organisms are applied: faecal coliforms and helminth (nematode) eggs, depending on the type of irrigation and on the group exposed. The guidelines also present suggestions for treatment processes to be applied and other relevant information.

3.5.3.4 Possible discharge standards for domestic sewage

Table 3.20 presents a simplified synthesis of possible discharge standards, according to different restriction levels, for the main pollutants of interest in domestic sewage. Depending on each country, region or situation, less stringent, stringent or very stringent standards may be adopted (or a combination of them, depending on the relative degree of importance of each parameter.

4

Overview of wastewater treatment systems

4.1 WASTEWATER TREATMENT LEVELS

In planning studies for the implementation of the wastewater treatment, the following points must be clearly addressed:

- *environmental impact studies on the receiving body*
- *treatment objectives*
- *treatment level and removal efficiencies*

The environmental impact studies that are necessary for the evaluation of the compliance with the receiving body standards were detailed in Chapter 3. The requirements to be reached for the effluent are also a function of the specific legislation that defines the quality standards for the effluent and for the receiving body. The legislation was also covered in Chapter 3.

The removal of pollutants during treatment in order to reach a desired quality or required discharge standard is associated with the concepts of *treatment level* and *treatment efficiency*.

Wastewater treatment is usually classified according to the following levels (see Tables 4.1 and 4.2):

- Preliminary
- Primary

Biological Wastewater Treatment in Warm Climate Regions by Marcos von Sperling and Carlos Augusto de Lemos Chernicharo. ISBN: 1 84339 002 7. Published by IWA Publishing, London, UK.

Table 4.1. Wastewater treatment levels

Level	Removal
Preliminary	• Coarse suspended solids (larger material and sand)
Primary	• Settleable suspended solids • Particulate (suspended) BOD (associated to the organic matter component of the settleable suspended solids)
Secondary	• Particulate (suspended) BOD (associated to the particulate organic matter present in the raw sewage, or to the non settleable particulate organic matter, not removed in the possibly existing primary treatment) • Soluble BOD (associated to the organic matter in the form of dissolved solids)
Tertiary	• Nutrients • Pathogenic organisms • Non-biodegradable compounds • Metals • Inorganic dissolved solids • Remaining suspended solids

Note: depending on the treatment process adopted, the removal of nutrients (by biological processes) and pathogens can be considered an integral part of secondary treatment.

- Secondary
- Tertiary

The objective of **preliminary treatment** is only the removal of coarse solids, while **primary treatment** aims at removing settleable solids and part of the organic matter. *Physical* pollutant removal mechanisms are predominant in both levels. In **secondary treatment** the aim is the removal of organic matter and possibly nutrients (nitrogen and phosphorus) by predominantly *biological* mechanisms. The objective of **tertiary treatment** is the removal of specific pollutants (usually toxic or non-biodegradable compounds) or the complementary removal of pollutants that were not sufficiently removed in the secondary treatment. Tertiary treatment is rare in developing countries.

The *removal efficiency* of a pollutant in the treatment or in a treatment stage is given by the formula:

$$\boxed{E = \frac{C_o - C_e}{C_o} . 100} \tag{4.1}$$

where:

E = removal efficiency (%)

C_o = influent concentration of the pollutant (mg/L)

C_e = effluent concentration of the pollutant (mg/L)

Table 4.2. Characteristics of the main wastewater treatment levels

	Treatment level⁽¹⁾		
Item	Preliminary	Primary	Secondary
Pollutants removed	• Coarse solids	• Settleable solids • Particulate BOD	• Non-settleable solids • Fine particulate BOD • Soluble BOD • Nutrients⁽⁴⁾ • Pathogens⁽⁴⁾
Removal efficiencies	–	• SS: 60–70% • BOD: 25–40% • Coliforms: 30–40%	• SS: 65–95% • BOD: 60–99% • Coliforms: 60–99%⁽³⁾
Predominant treatment mechanism	• Physical	• Physical	• Biological
Complies with usual discharge standards?⁽²⁾	• No	• No	• Usually yes
Application	• Upstream of pumping stations • Initial treatment stage	• Partial treatment • Intermediate stage of a more complete treatment	• More complete treatment (for organic matter)

Notes:

(1) A secondary level WWTP usually has preliminary treatment, but may or may not have primary treatment (depends on the process).
(2) Discharge standard as stated in the legislation. The environmental agency may authorise other values, if environmental studies demonstrate that the receiving body is able to assimilate a higher loading.
(3) The coliform removal efficiency can be higher if a specific removal stage is included.
(4) Depending on the treatment process, nutrients and pathogens may be removed in the secondary stage.

4.2 WASTEWATER TREATMENT OPERATIONS, PROCESSES AND SYSTEMS

The treatment methods are composed by unit operations and processes, and their integration makes up the treatment systems.

The concepts of unit operations and unit process are frequently used interchangeably, because they can occur simultaneously in the same treatment unit. In general, the following definitions can be adopted (Metcalf & Eddy, 1991):

- **Physical unit operations:** treatment methods in which *physical forces* are predominant (e.g. screening, mixing, flocculation, sedimentation, flotation, filtration)

- **Chemical unit processes:** treatment methods in which the removal or the conversion of the contaminants occurs by the addition of *chemical products* or due to *chemical reactions* (e.g. precipitation, adsorption, disinfection).
- **Biological unit processes:** treatment methods in which the removal of the contaminants occurs by means of *biological activity* (e.g. carbonaceous organic matter removal, nitrification, denitrification)

Various mechanisms can act separately or simultaneously in the removal of the pollutants, depending on the process being used. The main mechanisms are listed in Table 4.3.

Table 4.4 lists the main processes, operations, and treatment systems frequently used in the treatment of *domestic sewage*, as a function of the pollutant to be removed. These methods are employed in the **liquid phase** (or liquid lines), which corresponds to the main flow of the liquid (sewage) in sewage treatment works. On the other hand, the **solid phase** (covered in Section 5) is associated with the solid by-products generated in the treatment, notably sludge. The present text concentrates on the *biological treatment* of wastewater, which is the reason why physical – chemical treatment systems are not covered (these depend on the addition of chemical products and are used more frequently for the treatment of industrial wastewaters).

Table 4.5 presents a summary of the main secondary level domestic sewage treatment systems. The technology of wastewater treatment has various other processes and variants, but the present book addresses only the most frequently used systems in warm-climate countries. The flowsheets of the systems described in this table are presented in Figures 4.1a–f. The integration between the various operations and processes listed in Table 4.5 can be seen in the flowsheets. In all flowsheets, besides going to the receiving water body, the effluent may be reused (agricultural / industrial / other) if conditions so permit.

In order to allow a better understanding of the main wastewater treatment systems, the remainder of the chapter is devoted to a preliminary description of them. Further details may be found in various chapters throughout this book.

Table 4.3. Main mechanisms for the removal of pollutants in wastewater treatment

Pollutant	Subdivision	Main removal mechanisms	
Solids	Coarse solids (> ∼1 cm)	*Screening*	Retention of the solids with dimensions greater than the spacing between the bars
	Suspended solids (> ∼1 μm)	*Sedimentation*	Separation of the particles with a density greater than the sewage
	Dissolved solids (< ∼1 μm)	*Adsorption*	Retention on the surface of biomass flocs or biofilms
Organic matter	BOD in suspension (particulate BOD) (> ∼1 μm)	*Sedimentation*	Separation of the particles with a density greater than the sewage
		Adsorption	Retention on the surface of biomass flocs or biofilms
		Hydrolysis	Conversion of the BOD in suspension into soluble BOD by means of enzymes, allowing its stabilisation
		Stabilisation	Utilisation by biomass as food, with conversion into gases, water and other inert compounds.
	Soluble BOD (< ∼1 μm)	*Adsorption*	Retention on the surface of biomass flocs or biofilms
		Stabilisation	Utilisation by biomass as food, with conversion into gases, water and other inert compounds.
Pathogens	Larger dimensions and/or with protective layer (protozoan cysts and helminth eggs)	*Sedimentation*	Separation of pathogens with larger dimensions and density greater than the sewage
		Filtration	Retention of pathogens in a filter medium with adequate pore size
	Lower dimensions (bacteria and viruses)	*Adverse environmental conditions*	Temperature, pH, lack of food, competition with other species, predation
		Ultraviolet radiation	Radiation from the sun or artificial
		Disinfection	Addition of a disinfecting agent, such as chlorine

(*Continued*)

Table 4.3 (*Continued*)

Pollutant	Subdivision	Main removal mechanisms	
Nitrogen	Organic nitrogen	*Ammonification*	Conversion of organic nitrogen into ammonia
	Ammonia	*Nitrification*	Conversion of ammonia into nitrite, and the nitrite into nitrate, by means of nitrifying bacteria
		Bacterial assimilation	Incorporation of ammonia into the composition of bacterial cells
		Stripping	Release of free ammonia (NH_3) into the atmosphere, under high pH conditions
		Break-point chlorination	Conversion of ammonia into chloramines, through the addition of chlorine
	Nitrate	*Denitrification*	Conversion of nitrate into molecular nitrogen (N_2), which escapes into the atmosphere, under anoxic conditions
Phosphorus	Phosphate	*Bacterial assimilation*	Assimilation in excess of the phosphate from the liquid by phosphate accumulating organisms, which takes place when aerobic and anaerobic conditions are alternated
		Precipitation	Phosphorus precipitation under conditions of high pH, or through the addition of metallic salts
		Filtration	Retention of phosphorus-rich biomass, after stage of biological excessive P assimilation

Table 4.4. Treatment operations, processes and systems frequently used for the removal of pollutants from domestic sewage

Pollutant	Operation, process or treatment system
Suspended solids	• Screening • Grit removal • Sedimentation • Land disposal
Biodegradable organic matter	• Stabilisation ponds and variants • Land disposal • Anaerobic reactors • Activated sludge and variants • Aerobic biofilm reactors
Pathogenic organisms	• Maturation ponds • Land disposal • Disinfection with chemical products • Disinfection with ultraviolet radiation • Membranes

Table 4.4 (*Continued*)

Pollutant	Operation, process or treatment system
Nitrogen	• Nitrification and biological • denitrification • Maturation and high-rate ponds • Land disposal • Physical–chemical processes
Phosphorus	• Biological removal • Maturation and high-rate ponds • Physical chemical processes

Table 4.5. Summary description of the main biological wastewater treatment systems

STABILISATION PONDS	
Facultative pond	Wastewater flows continuously through a pond especially constructed for wastewater treatment. The wastewater remains in the ponds for many days. The soluble and fine particulate BOD is aerobically stabilised by bacteria which grow dispersed in the liquid medium, while the BOD in suspension tends to settle, being converted anaerobically by bacteria at the bottom of the pond. The oxygen required by the aerobic bacteria is supplied by algae through photosynthesis. The land requirements are high.
Anaerobic pond – facultative pond	Around 50 to 65% of the BOD is converted in the anaerobic pond (deeper and with a smaller volume), while the remaining BOD is removed in the facultative pond. The system occupies an area smaller than that of a single facultative pond.
Facultative aerated lagoon	The BOD removal mechanisms are similar to those of a facultative pond. However, oxygen is supplied by mechanical aerators instead of through photosynthesis. The aeration is not enough to keep the solids in suspension, and a large part of the sewage solids and biomass settles, being decomposed anaerobically at the bottom.
Completely mixed aerated lagoon – sedimentation pond	The energy introduced per unit volume of the pond is high, what makes the solids (principally the biomass) remain dispersed in the liquid medium, in complete mixing. The resulting higher biomass concentration in the liquid medium increases the BOD removal efficiency, which allows this pond to have a volume smaller than a facultative aerated lagoon. However, the effluent contains high levels of solids (bacteria) that need to be removed before being discharged into the receiving body. The sedimentation pond downstream provides conditions for this removal. The sludge of the sedimentation pond must be removed every few years.
High rate ponds	High rate ponds are conceived in order to maximise algal production, in a totally aerobic environment. To accomplish this, lower depths are employed, allowing light penetration throughout the liquid mass. Therefore, photosynthetic activity is high, leading to high dissolved oxygen concentrations and pH levels. These factors contribute to the increase of the pathogens die-off and to the removal of nutrients. High rate ponds usually receive a high organic load per unit surface area. Usually a moderate agitation in the liquid is introduced, caused by a low-power mechanical equipment.

(*Continued*)

Table 4.5 (*Continued*)

Maturation ponds	The main objective of maturation ponds is the removal of pathogenic organisms. In maturation ponds prevail environmental conditions which are adverse to these organisms, such as ultraviolet radiation, high pH, high DO, lower temperature (compared with the human intestinal tract), lack of nutrients and predation by other organisms. Maturation ponds are a post-treatment stage for BOD-removal processes, being usually designed as a series of ponds or a single-baffled pond. The coliform removal efficiency is very high.
	LAND DISPOSAL
Slow rate system	The objectives may be for (a) wastewater treatment or (b) water reuse through crop production or landscape irrigation. In each case, design criteria are different. Wastewater is applied to the soil, supplying water and nutrients necessary for plant growth. Part of the liquid evaporates, part percolates into the soil, and the largest fraction is absorbed by the plants The surface application rates are very low. The liquid can be applied by sprinkling, graded-border, furrow and drip irrigation.
Rapid infiltration	Wastewater is applied in shallow basins. The liquid passes through the porous bottom and percolates into the soil. The evaporation loss is lower in view of the higher application rates. Vegetation may or may not be used. The application is intermittent, which provides a rest period for the soil. The most common types are: application for groundwater recharge, recovery using underdrains and recovery using wells.
Subsurface infiltration	Pre-settled sewage (usually from septic tanks) is applied below the soil surface. The infiltration trenches or chambers are filled with a porous medium, which provides transportation, storage and partial treatment, followed by the infiltration itself.
Overland flow	Wastewater is distributed in the upper part of vegetated slopes, flows over the slopes and is collected by ditches at the lower part. Treatment occurs in the root-soil system. The application is intermittent. Distribution of wastewater may be by high-pressure sprinklers, low-pressure sprays and gated or perforated pipes or channels.
Constructed wetlands	While the former systems are land-based systems, these are aquatic-based systems. The systems are composed by shallow basins or channels in which aquatic plants grow. The system can be of free-water surface (water level above ground level) or subsurface flow (water level below ground level). Biological, chemical and physical mechanisms act on the root–soil system.
	ANAEROBIC SYSTEMS
Upflow anaerobic sludge blanket reactor (UASB)	BOD is converted anaerobically by bacteria dispersed in the reactor. The liquid flow is upwards. The upper part of the reactor is divided into settling and gas collection zones. The settling zone allows the exit of the clarified effluent in the upper part and the return of the solids (biomass) by gravity to the system, increasing its concentration in the reactor. Amongst the gases formed is methane. The system has no primary sedimentation tank. The sludge production is low, and the excess sludge wasted is already thickened and stabilised.

Table 4.5 (*Continued*)

Anaerobic filter	BOD is converted anaerobically by bacteria that grow attached to a support medium (usually stones) in the reactor. The tank works submerged and the flow is upwards. The system requires a primary sedimentation tank (frequently septic tanks). The sludge production is low and the excess sludge is already stabilised.
Anaerobic reactor – post-treatment	UASB reactors produce an effluent that has difficulty in complying with most existing discharge standards. Therefore, some form of post treatment is frequently necessary. The post treatment may be biological (aerobic or anaerobic) or physical-chemical (with the addition of coagulants). Practically all wastewater treatment processes may be used as a post treatment of the anaerobic reactors. The global efficiency of the system is usually similar to the one that would be obtained if the process were being applied for raw wastewater. However, land, volume and energy requirements are lower. Sludge production is also lower.
	ACTIVATED SLUDGE
Conventional activated sludge	The biological stage comprises two units: aeration tank (reactor) and secondary sedimentation tank. The biomass concentration in the reactor is very high, due to the recirculation of the settled solids (bacteria) from the bottom of the secondary sedimentation tank. The biomass remains in the system longer than the liquid, which guarantees a high BOD removal efficiency. It is necessary to remove a quantity of the sludge (biomass) that is equivalent to what is produced. This excess sludge removed needs to be stabilised in the sludge treatment stage. The oxygen supply is done by mechanical aerators or by diffused air. Upstream of the reactor there is a primary sedimentation tank to remove the settleable solids from the raw sewage.
Activated sludge (extended aeration)	Similar to the previous system, but with the difference that the biomass stays longer in the system (the aeration tanks are bigger). With this, there is less substrate (BOD) available for the bacteria, which makes them use their own cellular material as organic matter for their maintenance. Consequently, the removed excess sludge (bacteria) is already stabilised. Primary sedimentation tanks are usually not included.
Intermittently operated activated sludge (sequencing batch reactors)	The operation of the system is intermittent. In this way, the reaction (aerators on) and settling (aerators off) stages occur in different phases in the same tank. When the aerators are turned off, the solids settle, which allows the removal of the clarified effluent (supernatant). When the aerators are turned on again, the settled solids return to the liquid mass, with no need of sludge recirculation pumps. There are no secondary sedimentation tanks. It can be in the conventional or extended aeration modes.
Activated sludge with biological nitrogen removal	The biological reactor incorporates an anoxic zone (absence of oxygen, but presence of nitrates). The anoxic zone can be upstream and/or downstream of the aerated zone. The nitrates formed in the nitrification process that takes place in the aerobic zone are used in the respiration of facultative microorganisms in the anoxic zones, being reduced to gaseous molecular nitrogen, which escapes to the atmosphere.

(*Continued*)

Table 4.5 (*Continued*)

Activated sludge with biological nitrogen and phosphorus removal	Besides the aerobic and anoxic zones, the biological reactor also incorporates an anaerobic zone, situated at the upper end of the tank. Internal recirculations make the biomass to be successively exposed to anaerobic and aerobic conditions. With this alternation, a certain group of microorganisms absorbs phosphorus from the liquid medium, in quantities that are much higher than those which would be normally necessary for their metabolism. The withdrawal of these organisms in the excess sludge results in the removal of phosphorus from the biological reactor.
	AEROBIC BIOFILM REACTORS
Low rate trickling filter	BOD is stabilised aerobically by bacteria that grow attached to a support medium (commonly stones or plastic material). The sewage is applied on the surface of the tank through rotating distributors. The liquid percolates through the tank and leaves from the bottom, while the organic matter is retained and then further removed by the bacteria. The free spaces permit the circulation of air. In the low rate system there is a low availability of substrate (BOD) for the bacteria, which makes them undergo self-digestion and leave the system stabilised. Sludge that is detached from the support medium is removed in the secondary sedimentation tank. The system requires primary sedimentation.
High rate trickling filter	Similar to the previous system but with the difference that a higher BOD load is applied. The bacteria (excess sludge) need to be stabilised within the sludge treatment. The effluent from the secondary sedimentation tank is recirculated to the filter in order to dilute the influent and to guarantee a homogeneous hydraulic load.
Submerged aerated biofilter	The submerged aerated biofilter is composed by a tank filled with a porous material (usually submerged), through which sewage and air flow permanently. The air flow is always upwards, while the liquid flow can be downward or upward. The biofilters with granular material undertake, in the same reactor, the removal of soluble organic compounds and particulate matter. Besides being a support medium for biomass growth, the granular material acts also as a filter medium. Periodic backwashings are necessary to eliminate the excess biomass accumulated, reducing the head loss through the medium.
Rotating biological contactor (biodisc)	The biomass grows adhered to a support medium, which is usually composed by a series of discs. The discs, partially immersed in the liquid, rotate, exposing their surface alternately to liquid and air.

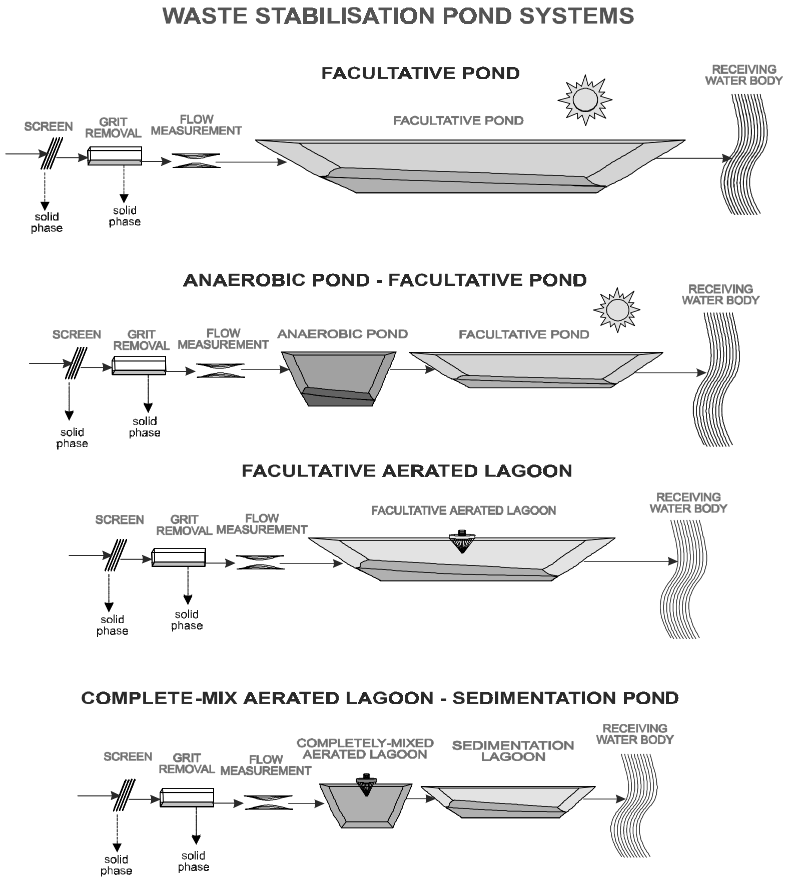


Figure 4.1a. Flowsheet of stabilisation pond systems (liquid phase only).

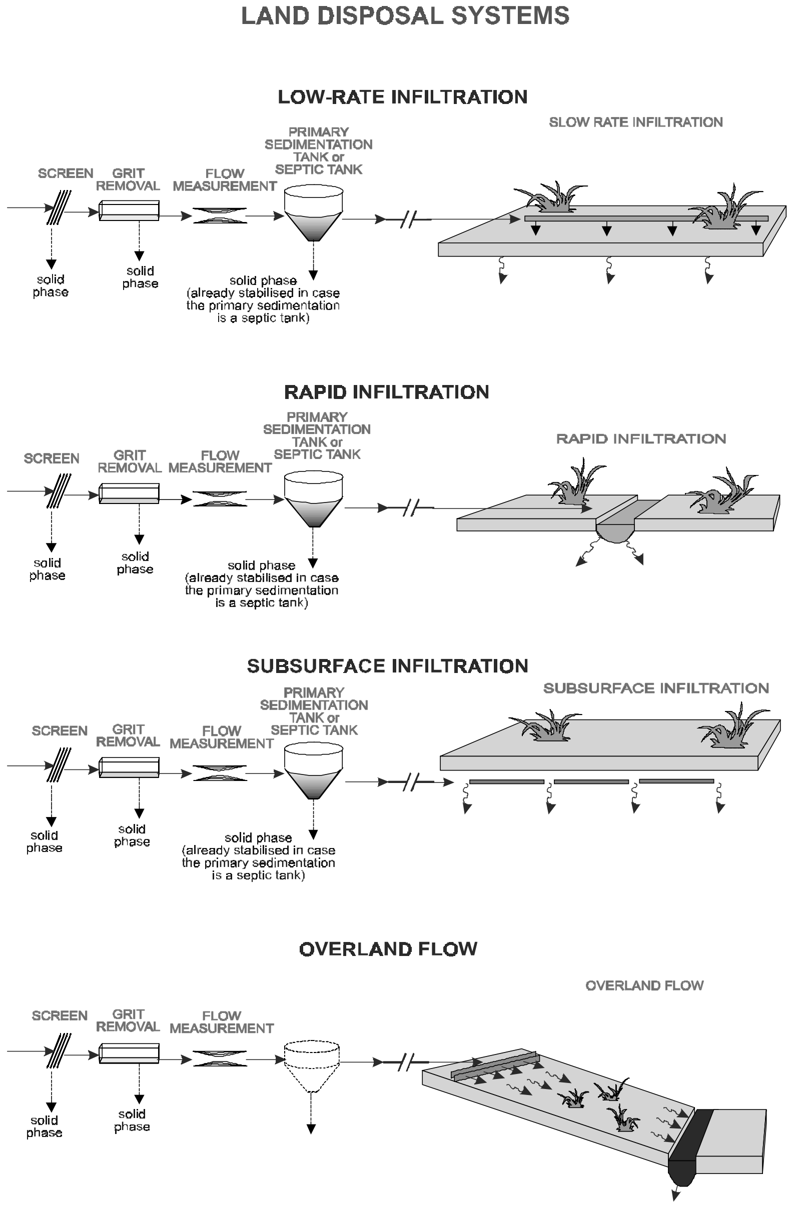


Figure 4.1b. Flowsheet of soil-based land treatment systems (liquid phase only).

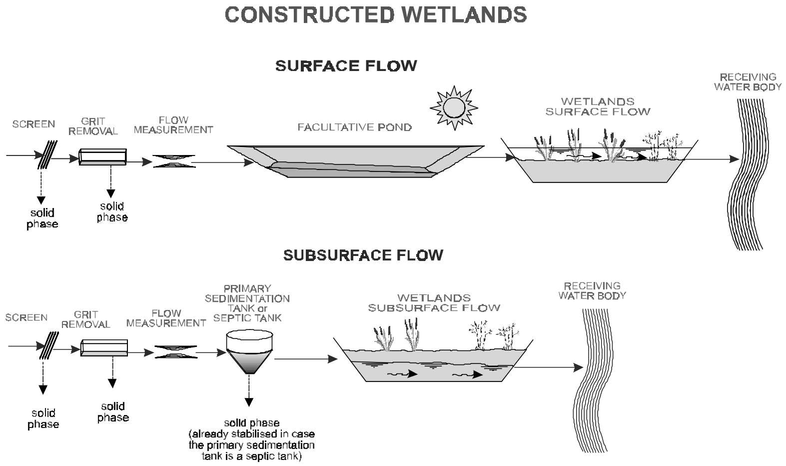


Figure 4.1c. Flowsheet of aquatic-based land treatment systems (liquid phase only).

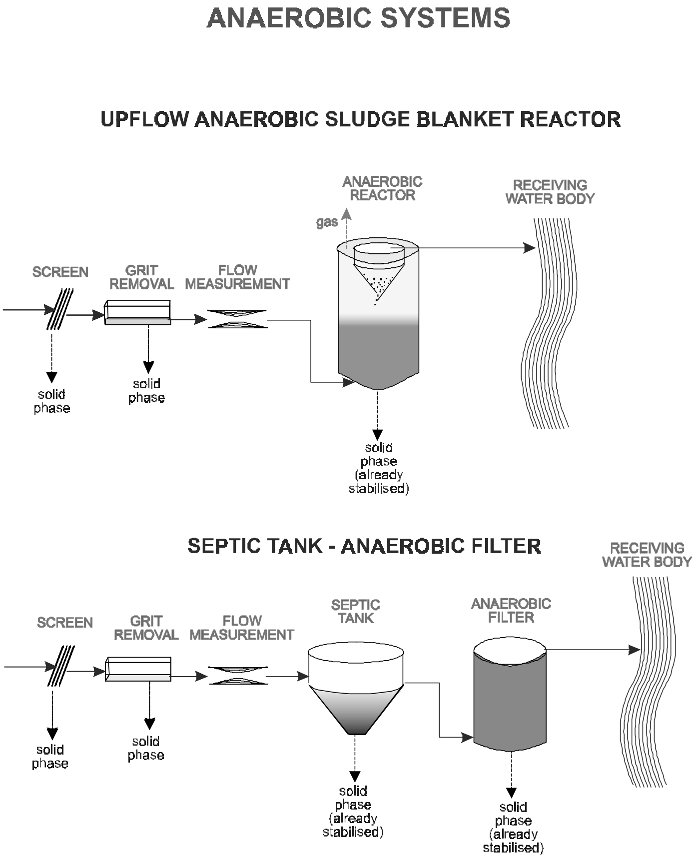


Figure 4.1d. Flowsheet of anaerobic reactors (liquid phase only).

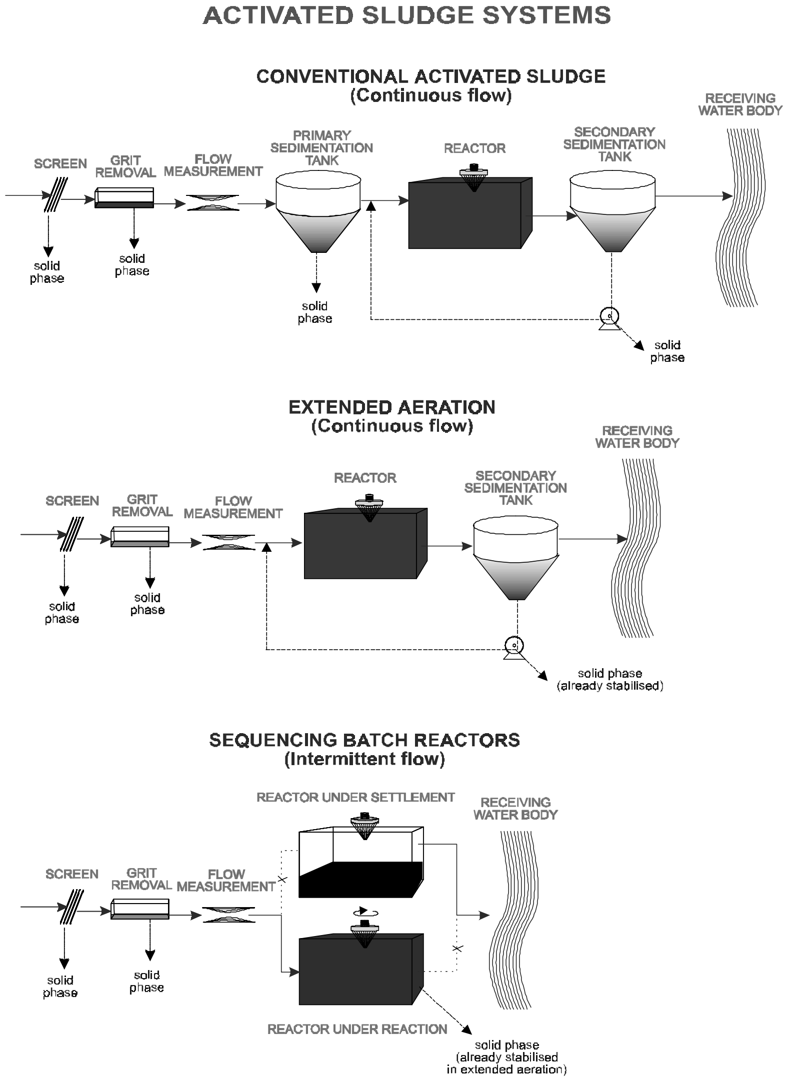


Figure 4.1e. Flowsheet of activated sludge systems (liquid phase only).

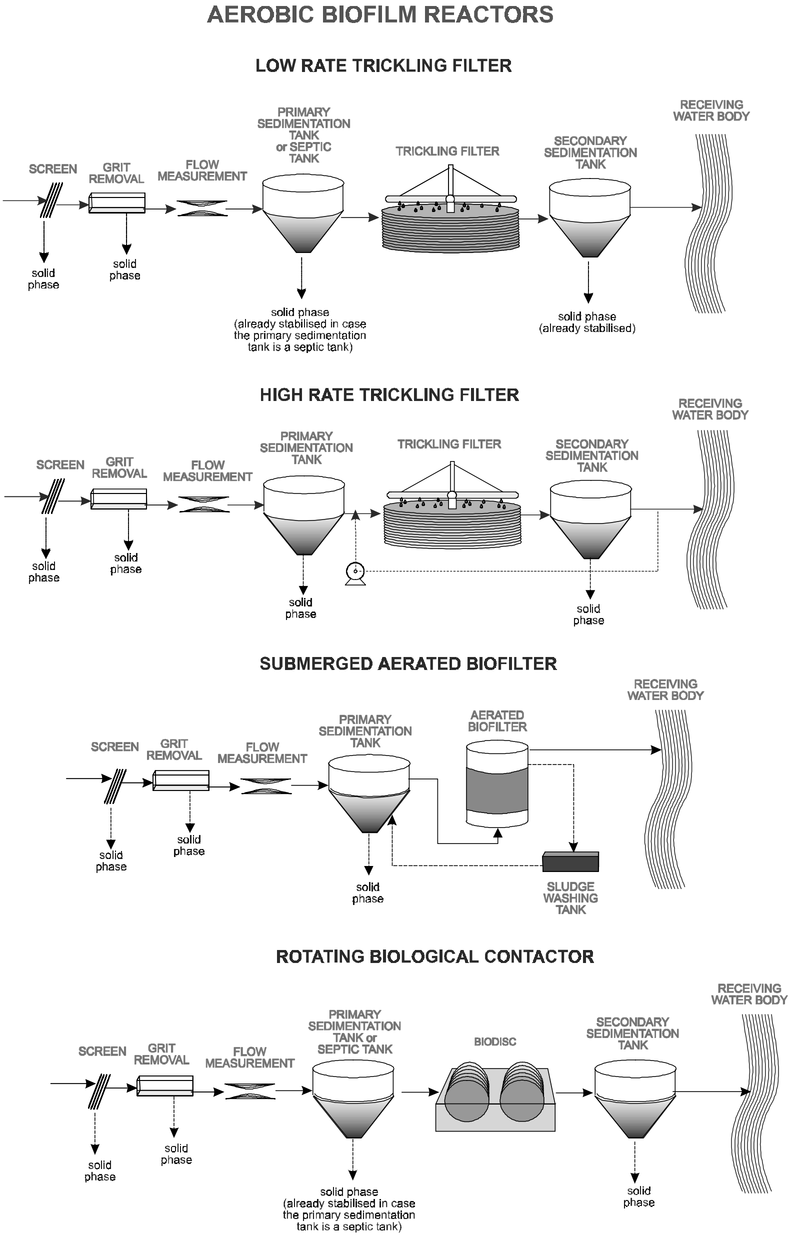


Figure 4.1f. Flowsheet of aerobic biofilm reactors (liquid phase only).

4.3 PRELIMINARY TREATMENT

Preliminary treatment is mainly intended for the removal of:

- *Coarse solids*
- *Grit*

The basic removal mechanisms are of a *physical* order. Besides the coarse solids removal units, there is also a *flow measurement* unit. This usually consists of a standardised flume (e.g. Parshall flume), where the measured liquid level can be correlated with the flow. Weirs (rectangular or triangular) and closed-pipe measurement mechanisms can also be adopted. Figure 4.2 presents a typical flowsheet of the preliminary treatment.

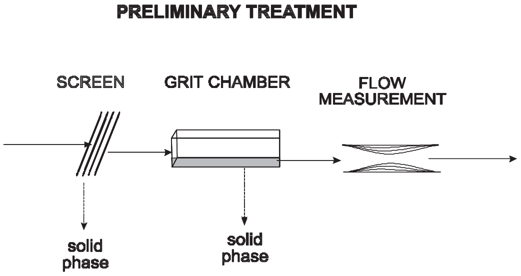


Figure 4.2. Typical flowsheet of the preliminary treatment

The removal of *coarse solids* is frequently done by *screens* or racks, but static or rotating screens and comminutors can also be used. In the screening, material with dimensions larger than the spaces between the bars is removed (see Figure 4.3). There are coarse, medium, and fine screens, depending on the spacing between the bars. The removal of the retained material can be manual or mechanised.

The main objectives of the removal of coarse solids are:

- *protection of the wastewater transport devices (pumps and piping)*
- *protection of the subsequent treatment units*
- *protection of the receiving bodies*

The removal of *sand* contained in the sewage is done through special units called *grit chambers* (see Figure 4.4). The sand removal mechanism is simply by sedimentation: the sand grains go to the bottom of the tank due to their larger dimensions and density, while the organic matter, which settles much slower, stays in suspension and goes on to the downstream units.

There are many processes, from manual to completely mechanised units, for the removal and transportation of the settled grit. The basic purposes of grit removal are:

- *to avoid abrasion of the equipment and piping*
- *to eliminate or reduce the possibility of obstructions in piping, tanks, orifices, siphons, etc*
- *to facilitate the transportation of the liquid, principally the transfer of the sludge in its various phases*

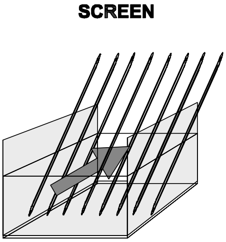


Figure 4.3. Schematics of a screen

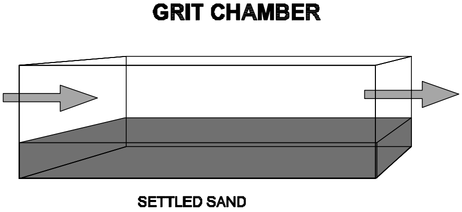


Figure 4.4. Diagram of a grit chamber

4.4 PRIMARY TREATMENT

Primary treatment aims at the removal of:

- *settleable suspended solids*
- *floating solids*

After passing the preliminary treatment units, sewage still contains non-coarse suspended solids, which can be partially removed in sedimentation units. A significant part of these suspended solids is comprised of organic matter in suspension. In this way, its removal by simple processes such as sedimentation implies a reduction in the BOD load directed to the secondary treatment, where its removal is more expensive.

The sedimentation tanks can be circular (Figure 4.5) or rectangular. Sewage flows slowly through the sedimentation tanks, allowing the suspended solids with a greater density than the surrounding liquid to slowly settle to the bottom. The mass of solids accumulated in the bottom is called raw *primary sludge*. This sludge is removed through a single pipe in small sized tanks or through mechanical scrapers and pumps in larger tanks. Floating material, such as grease and oil, tends to have a lower density than the surrounding liquid and rise to the surface of the sedimentation tanks, where they are collected and removed from the tank for subsequent treatment.

The efficiency of primary treatment in the removal of suspended solids, and, as result, BOD, may be enhanced by the addition of coagulants. This is called *advanced primary treatment* or *chemically enhanced primary treatment* (CEPT).

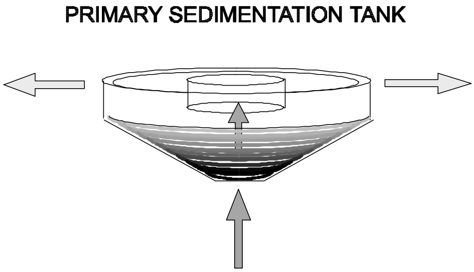


Figure 4.5. Schematics of a circular primary sedimentation tank

Coagulants may be aluminium sulphate, ferric chloride or other, aided or not by a polymer. Phosphorus may be also removed by precipitation. More sludge is formed, resulting from the higher amount of solids removed from the liquid and from the chemical products added. The primary sludge may be digested by conventional digesters, but in some cases it may also be stabilised by lime (simplifying the flowsheet, but further increasing the amount of sludge to be disposed of).

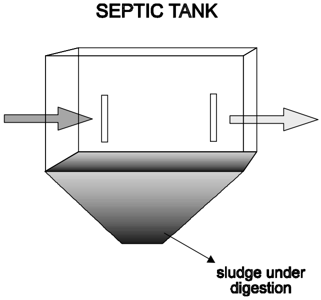


Figure 4.6. Schematics of a single-chamber septic tank

Septic tanks are also a form of primary treatment (Figure 4.6). The septic tanks and their variants, such as Imhoff tanks, are basically sedimentation tanks, where the settleable solids are removed to the bottom. These solids (sludge) remain at the bottom of the tanks for a long period of time (various months) which is sufficient for their digestion. This stabilisation occurs under anaerobic conditions.

4.5 SECONDARY TREATMENT

4.5.1 Introduction

The main objective of secondary treatment is the removal of *organic matter*. Organic matter is present in the following forms:

- *dissolved organic matter* (**soluble** or **filtered BOD**) that is not removed by merely physical operations, such as the sedimentation that occurs in primary treatment;
- *organic matter in suspension* (**suspended** or **particulate BOD**), which is largely removed in the occasionally existing primary treatment, but whose solids with slower settleability (finer solids) remain in the liquid mass.

The secondary treatment processes are conceived in such a way as to accelerate the decomposition mechanisms that naturally occur in the receiving bodies. Thus, the decomposition of the degradable organic pollutants is achieved under controlled conditions, and at smaller time intervals than in the natural systems.

The essence of secondary treatment of domestic sewage is the inclusion of a *biological stage*. While preliminary and primary treatments have predominantly physical mechanisms, the removal of the organic matter in the secondary stage is carried out through biochemical reactions, undertaken by microorganisms.

A great variety of microorganisms take part in the process: bacteria, protozoa, fungi and others. The basis of the whole biological process is the effective contact between these organisms and the organic matter contained in the sewage, in such a way that it can be used as food for the microorganisms. The microorganisms convert the organic matter into carbon dioxide, water and cellular material (growth and reproduction of the microorganisms) (see Figure 4.7). This biological decomposition of the organic matter requires the presence of oxygen as a fundamental component of the aerobic processes, besides the maintenance of other favourable environmental conditions, such as temperature, pH, contact time, etc.

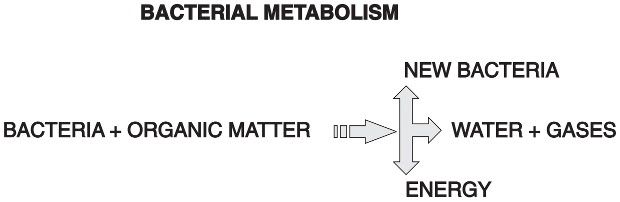


Figure 4.7. Simplified diagram of bacterial metabolism

Secondary treatment generally includes preliminary treatment units, but may or may not include primary treatment units. There exists a large variety of secondary treatment processes, and the most common ones are:

- *Stabilisation ponds*
- *Land disposal systems*
- *Anaerobic reactors*
- *Activated sludge systems*
- *Aerobic biofilm reactors*

These processes have been summarised in Table 4.5, and a simplified description is presented below. In Section 4.7.2, there is a general comparison between all the processes described, including basic data (efficiencies, land requirements, power requirements, costs, sludge production, etc.), together with qualitative comparisons and a list of advantages and disadvantages.

4.5.2 Stabilisation ponds

The following variants of stabilisation ponds are described briefly in this section:

- Facultative ponds
- Anaerobic pond – facultative ponds systems
- Facultative aerated lagoons
- Complete-mix aerated lagoon – sedimentation pond systems
- High rate ponds
- Maturation ponds

a) Facultative ponds

Stabilisation ponds are units specially designed and built with the purpose of treating sewage. However, the construction is simple and is principally based on earth movement for digging, filling and embankment preparation.

When facultative ponds receive raw sewage, they are also called *primary ponds* (a secondary pond would be the one which would receive its influent from a previous treatment unit, such as anaerobic ponds – see item *b* in this section).

Amongst the stabilisation ponds systems, the process of facultative ponds is the simplest, relying only on natural phenomenon. The influent enters continuously in one end of the pond and leaves in the opposite end. During this time, which is of the order of many days, a series of events contribute to the purification of the sewage.

Part of the organic matter in suspension (*particulate BOD*) tends to settle, constituting the bottom sludge. This sludge undergoes a decomposition process by anaerobic microorganisms and is converted into carbon dioxide, methane and other compounds. The inert fraction (non-biodegradable) stays in this bottom layer.

The dissolved organic matter (*soluble BOD*), together with the small-dimension organic matter in suspension (*fine particulate BOD*), does not settle and stays dispersed in the liquid mass. Its decomposition is through **facultative** bacteria that have the capacity to survive, either in the presence or in the absence of free oxygen (but presence of nitrate), hence the designation of facultative, which also defines the name of the pond. These bacteria use the organic matter as energy source, which is released through respiration. The presence of oxygen is necessary in aerobic respiration, and it is supplied to the medium by the photosynthesis carried out by the algae. There is an equilibrium between consumption and the production of oxygen and carbon dioxide (see Figure 4.8).

Bacteria ➔ *respiration*:

- oxygen consumption
- carbon dioxide production

Algae ➔ *photosynthesis*:

- oxygen production
- carbon dioxide consumption

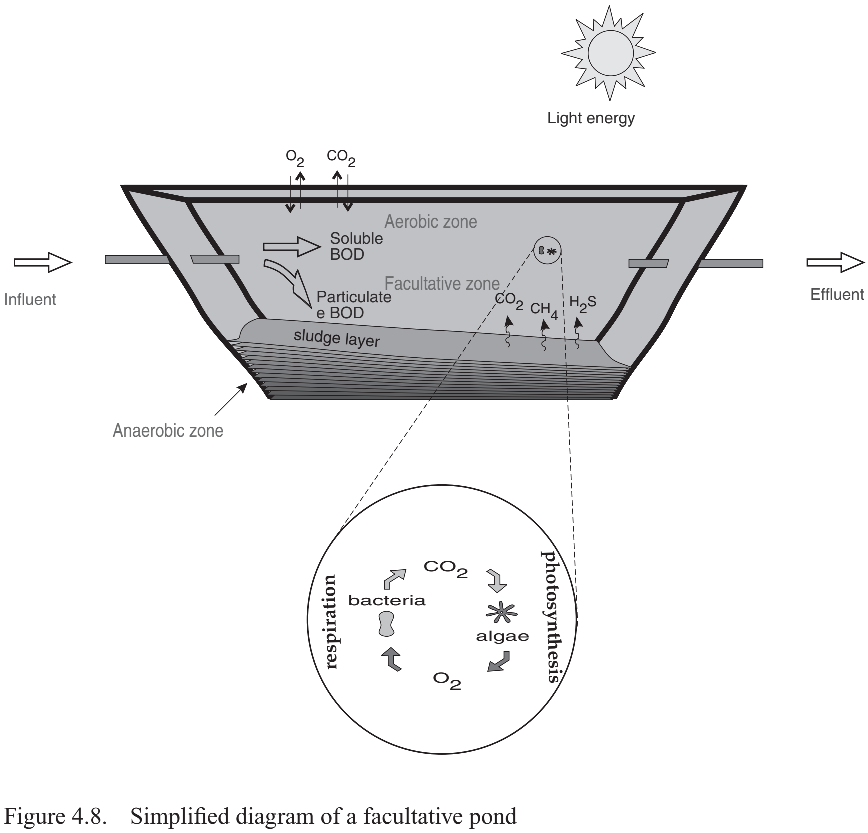


Figure 4.8. Simplified diagram of a facultative pond

A light energy source, in this case represented by the sun, is necessary for photosynthesis to occur. For this reason, locations with high solar radiation and low cloudiness are favourable for the implementation of facultative ponds.

Photosynthesis is higher near the water surface, as it depends on solar energy. Typical pond depths are between 1.5 and 2.0 m. When deep regions in the pond are reached, the light penetration is low, what causes the predominance of oxygen consumption (*respiration*) over its production (*photosynthesis*), with the possible absence of dissolved oxygen at a certain depth. Besides, photosynthesis only occurs during the day, and during the night the absence of oxygen can prevail. Owing to these facts, it is essential that the main bacteria responsible for the stabilisation of the organic matter are facultative, so that they can survive and proliferate, either in the presence or in the absence of oxygen (but only under anoxic, and not strict anaerobic conditions).

The process of facultative ponds is essentially natural, as it does not require any equipment. For this reason, the stabilisation of the organic matter takes place at lower rates, implying the need of a long detention time in the pond (usually greater than 20 days). To be effective, photosynthesis needs a large exposure surface area to make the most of the solar energy by the algae, also implying the need of large units. As a result, the total area required by facultative ponds is the largest within all the wastewater treatment processes (excluding the land disposal processes). On the other hand, because the process is totally natural, it is associated to a high operational simplicity, which is a factor of fundamental importance in developing countries.

Figure 4.9 presents a typical flowsheet of a facultative pond system.

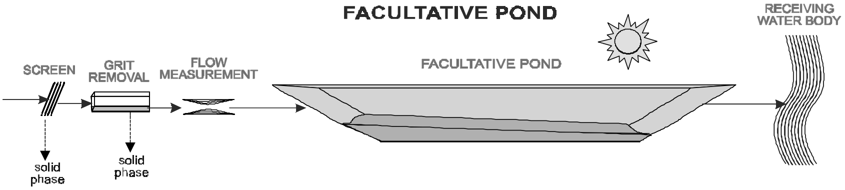


Figure 4.9. Typical flowsheet of a facultative pond system

b) Anaerobic pond – facultative ponds systems

The process of facultative ponds, in spite of having a satisfactory efficiency, requires a large area that is often not available in the locality in question. There is therefore, the need to find solutions that could imply the reduction of the total area required. One of these solutions is the system of anaerobic ponds followed by facultative ponds. In this case, the facultative pond is also called a *secondary* pond, since it receives the influent from an upstream treatment unit, and not the raw sewage.

The raw sewage enters a pond that has smaller dimensions and is deeper (around 4 to 5 m). Owing to the smaller dimensions of this pond, photosynthesis practically does not occur. In the balance between oxygen consumption and production, consumption is much higher. Therefore, anaerobic conditions predominate in this first pond, which is consequently called an *anaerobic pond.*

Anaerobic bacteria have a slower metabolic and reproduction rate than the aerobic bacteria. For a detention time of only 2 to 5 days in the anaerobic pond, there is only partial decomposition of the organic matter. However, the BOD removal of the order of 50 to 70%, even if insufficient, represents a large contribution, substantially reducing the load to the facultative pond that is situated downstream.

The facultative pond receives a load of only 30 to 50% of the raw sewage load, which therefore allows it to have smaller dimensions. The overall area requirement (anaerobic + facultative pond) is such, that land savings in the order of 1/3 are obtained, compared with just a single facultative pond.

The working principles of this facultative pond are exactly as described in item a. Figure 4.10 shows a typical flowsheet of a system of anaerobic ponds followed by facultative ponds.

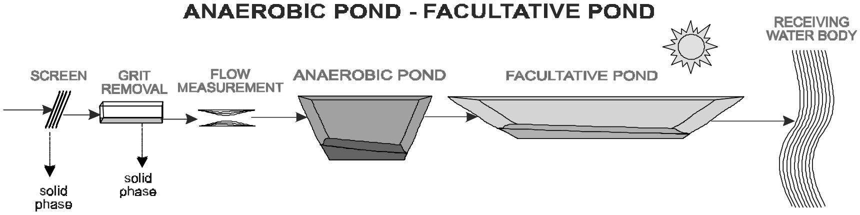


Figure 4.10. Typical flowsheet of a system of anaerobic ponds followed by facultative ponds

The efficiency of the system is similar or only slightly higher than that of a single facultative pond. The system is also conceptually simple and easy to operate. However, the existence of an anaerobic stage in an open unit is always a cause for concern, due to the possibility of the release of malodours. If the system is well balanced, then the generation of bad smells should not occur. However, occasional operational problems could lead to the release of hydrogen sulphide, responsible for a bad smell. For this reason, this system should, whenever possible, be located far from residences.

c) Facultative aerated lagoon

If a predominantly aerobic system is desired, with even smaller dimensions, a *facultative aerated lagoon* can be used. The main difference in relation to a conventional facultative pond is regarding the form of the oxygen supply. While in facultative ponds the oxygen is mainly obtained from photosynthesis, in the case of facultative aerated lagoons the oxygen is supplied by mechanical equipment called *aerators*.

The most commonly used mechanical aerators in aerated ponds are those with a vertical axis that rotates at a high speed, causing great turbulence in the water. This turbulence favours the penetration of atmospheric oxygen into the liquid mass, where it is then dissolved. A greater oxygen introduction is achieved, in comparison with the conventional facultative pond, which leads to a faster decomposition of the organic matter. Because of this, the detention time of the wastewater in the pond can be less (in the order of 5 to 10 days for domestic sewage). Consequently, the land requirements are much smaller.

The pond is called *facultative* by the fact that the level of energy introduced by the aerators is only sufficient for the oxygenation, but not to maintain the solids (biomass and wastewater solids) in suspension in the liquid mass. In this way, the solids tend to settle and constitute a sludge layer at the bottom of the pond, to be decomposed anaerobically. Only the soluble and fine particulate BOD remains in the liquid mass, undergoing aerobic decomposition. Therefore, the pond behaves like a conventional facultative pond (see Figure 4.11).

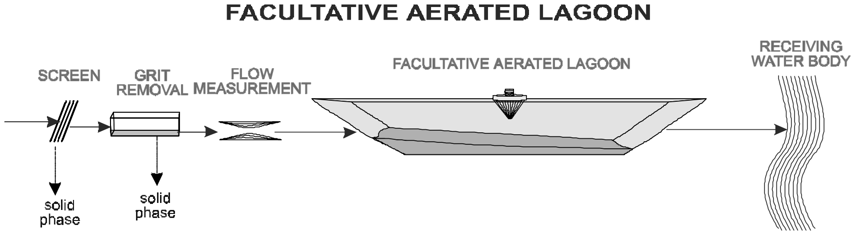


Figure 4.11. Typical flowsheet of a system of facultative aerated lagoons

Aerated lagoons are less simple in terms of operation and maintenance, when compared with the conventional facultative ponds, owing to the introduction of mechanisation. Therefore, the reduction of the reduction of the land requirements is achieved with a certain rising in the operational level, along with the introduction of electricity consumption.

d) Complete-mix aerated lagoon – sedimentation pond systems

A way of reducing the aerated pond volume even further is to increase the aeration level per unit volume of the lagoon, thus creating a turbulence that, besides guaranteeing oxygenation, allows all the solids to be maintained in suspension in the liquid medium. The denomination of *complete mix* is because of the high degree of energy per unit volume, which is responsible for the total mixing of all the constituents in the pond. Amongst the solids maintained in suspension and in complete mixing are the biomass, besides the organic matter of the raw sewage. There is, therefore, a larger concentration of bacteria in the liquid medium as well as a larger organic matter – biomass contact. Consequently, the efficiency of the system increases and allows the volume of the aerated pond to be greatly reduced. The typical detention time in an aerated pond is in the order of 2 to 4 days.

However, in spite of the high efficiency of this lagoon in the removal of the organic matter originally present in the sewage, a new problem is created. The biomass stays in suspension in all the volume and thus leaves with the pond effluent. This biomass is, in a way, also organic matter (particulate BOD), even if of a different nature of the BOD of the raw sewage. If this new organic matter were discharged into the receiving body, it would also exert an oxygen demand and cause a deterioration in the water quality.

Therefore, it is important that there is a unit downstream in which the suspended solids (predominantly the biomass) can settle and be separated from the liquid (final effluent). This unit is a *sedimentation pond*, with the main purpose of permitting the settling and accumulation of the solids.

The sedimentation pond is designed with short detention times, around 2 days. In this period, the solids will go to the bottom where they will undergo digestion and be stored for a period of some years, after which they will be removed. There are also sedimentation ponds with continuous removal of the bottom sludge, using, for instance, pumps mounted on rafts.

The land required for this pond system is the smallest within the pond systems. The energy requirements are similar to or only slightly higher than those in the facultative aerated lagoons. However, the aspects related to sludge handling can be more complicated, due to the fact that there is a smaller storage period in the pond compared with the other systems. If the sludge is removed periodically, this will take place with an approximate frequency of around 2 to 5 years. The removal of the sludge is a laborious and expensive task.

Figure 4.12 illustrates the flowsheet of the system.

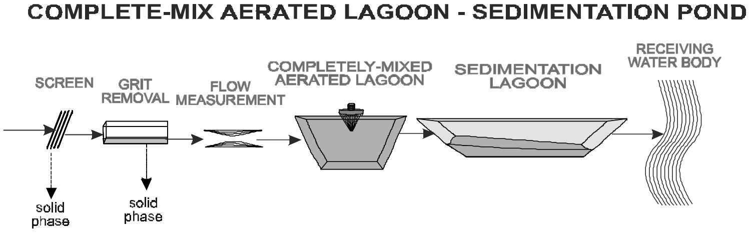


Figure 4.12. Typical flowsheet of a system of complete-mix aerated lagoons – sedimentation ponds

e) High rate ponds

High rate algal ponds are conceived to maximise the production of algae in a totally aerobic environment. To accomplish this, the ponds are shallow (less than 1.0 m depth), thus guaranteeing the penetration of light in all the liquid mass. Consequently, the photosynthetic activity is high, leading to high dissolved oxygen concentrations and an increase in pH (consumption of carbonic acid in the photosynthesis). These factors contribute to the increase in the death rate of the pathogenic microorganisms and the removal of nutrients, which is the main objective of the high rate ponds.

Ammonia removal occurs by stripping of the free ammonia (NH_3), since in high pH conditions the ammonia equilibrium shifts in the direction of free ammonia (with a reduction in the concentration of the ammonium ion NH_4^+). The increase in the NH_3 concentration leads to its release to the atmosphere.

Phosphorus removal also occurs as a result of the high pH, which causes the precipitation of the phosphates into the form of hydroxyapatite or struvite.

The high rate ponds receive a high organic load per unit surface area. There is usually the introduction of moderate agitation in the pond, which is achieved by means of a horizontal-axis rotor or equivalent equipment. Its function is not to aerate, but to gently move the liquid mass. This agitation improves the contact of the influent with the bacteria and algae, reduces dead zones and facilitates the exposure of a larger quantity of algae to sun light.

The configuration of the pond can be in the form of a carrousel, similar to an oxidation ditch (Figure 4.13). The high rate ponds can come after facultative ponds, in which a large part of the BOD is removed, leaving the polishing in terms of pathogen and nutrient removal for the high rate ponds.

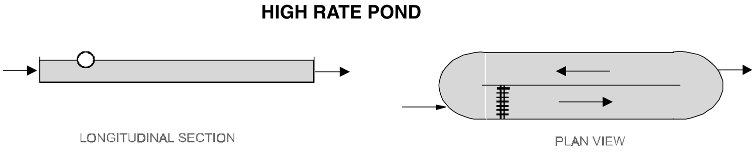


Figure 4.13. Schematics of a high rate pond

f) Maturation ponds

Maturation ponds aim at polishing the effluent from any stabilisation pond system previously described or, in broader terms, from any sewage treatment system. The main objective of maturation ponds is the *removal of pathogenic organisms* and not an additional BOD removal. Maturation ponds are an economic alternative for the disinfection of the effluent, in comparison to more conventional methods, such as chlorination.

The ideal environment for pathogenic microorganisms is the human intestinal tract. Outside it, whether in the sewerage system, in the sewage treatment or in the receiving body, the pathogenic organisms tend to die. Various factors contribute to this, such as temperature, solar radiation, pH, food shortage, predator organisms, competition, toxic compounds, etc. The maturation pond is designed in such a way as to optimise some of these mechanisms. Many of these mechanisms become more effective with lower pond depths, which justifies the fact that maturation ponds are shallower than the other types of ponds. Within the mechanisms associated with the pond depth, the following can be mentioned (van Haandel and Lettinga, 1994; van Buuren et al, 1995):

- Solar radiation (ultraviolet radiation)
- High pH (pH > 8.5)
- High DO concentration (favouring an aerobic community, which is more efficient in the elimination of coliforms)

Maturation ponds should reach extremely high coliform removal efficiencies (E > 99.9 or 99.99%), so that the effluent can comply with usual standards or guidelines for direct use (e.g. for irrigation) or for the maintenance of the various uses of the receiving body (e.g. bathing). The ponds also usually reach total removal of helminth eggs.

In order to maximise the coliform removal efficiency, the maturation ponds are designed with one of the following two configurations: (a) three or four ponds in series (see Figure 4.14) or (b) a single pond with baffles.

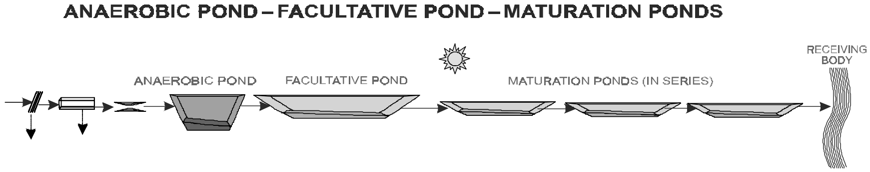


Figure 4.14. Typical flowsheet of a system of stabilisation ponds followed by maturation ponds in series.

4.5.3 Land disposal

The most common destinations for the final disposal of treated liquid effluents are water courses and the sea. However, land disposal is also a viable process, applied in various locations around the world.

The land application of wastewater can be considered as a form of *final disposal*, of *treatment* (primary, secondary or tertiary level) or both. Land application of wastewater leads to groundwater recharge and/or to evapotranspiration. Sewage supplies the plants with water and nutrients.

In the soil, a pollutant has basically four possible destinations:

- *retention in the soil matrix*
- *retention by the plants*
- *appearance in the underground water*
- *collection by underdrains*

Various mechanisms in the soil act in the removal of the pollutants:

- *physical* (settling, filtration, radiation, volatilisation, dehydration)
- *chemical* (oxidation and chemical reactions, precipitation, adsorption, ion exchange, complexation, photochemical breakdown)
- *biological* (biodegradation and predation)

The capacity of the soil to assimilate complex organic compounds depends on its properties and on climatic conditions. Infiltration rates and types of vegetation are important factors in the use of soil as a medium for the degradation of organic compounds. These reactions require good soil aeration, which is inversely related to the humidity of the soil. Insufficient aeration leads to a lower assimilation capacity of the organic compounds by the soil. Almost all soil types are efficient in the removal of organic matter. The removal results from the filtering action of the soil, followed by the biological oxidation of the organic material. Soils with clay (fine texture) or soils with a considerable quantity of organic matter will also retain wastewater constituents through mechanisms of adsorption, precipitation and ion exchange.

The most common types of land application are:

Soil-based systems:

- slow-rate systems
- rapid infiltration
- subsurface infiltration
- overland flow

Aquatic-based systems:

- constructed wetlands

Aquatic-based systems (constructed wetlands) are included in this section for didactic reasons, although they could have been presented in the section of stabilisation ponds, which are also aquatic-based systems.

The selection of the treatment method is a function of various factors, including required efficiency, climatic conditions, depth to ground water, soil permeability, slope etc. The application of wastewater can be done by methods such as sprinklers, furrows, graded border, drip irrigation and others.

a) Slow-rate systems

Depending on the design objective, slow-rate systems can be classified according to two types (WPCF, 1990):

- **Slow infiltration systems**. Main objective: *wastewater treatment*. The amount of wastewater applied is not controlled by crop requirements. For municipal wastewaters, loading rates are controlled either by nitrogen loading or soil permeability. The systems are designed to maximise the amount of wastewater applied per unit land area.
- **Crop irrigation systems**. Main objective: *water reuse for crop production* (wastewater treatment is an additional objective). The systems are designed to apply sufficient wastewater to meet crop irrigation requirements. Loading rates are based on the crop irrigation requirement and the application efficiency of the distribution system. Nitrogen loading must be checked to avoid excess nitrogen.

In *crop irrigation systems*, the objective is to supply the wastewater to the soil in quantities compatible with the *nutrient* requirements of the crops. However, initially the microbiological and biochemical characteristics of the sewage should be evaluated, taking into consideration the type of crop, soil, irrigation system and the form in which the product will be used or consumed. Only after the verification that the sewage meets the conditions specified by the health standards should the evaluation of the chemical components be considered (Mattos, 1998).

Figure 4.15 presents a flowsheet of a slow-rate system. The irrigation with wastewater can be done by flooding, furrows, sprinkler and dripping.

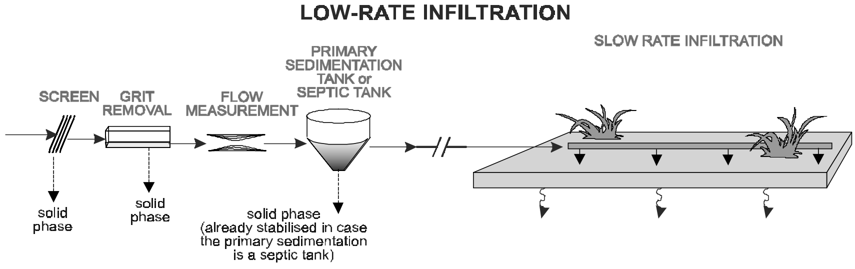


Figure 4.15. Typical flowsheet of a slow-rate system

Loamy soils (medium texture) are indicated because they exhibit the best balance for wastewater renovation and drainage. The depth to the water table should be greater than 1.5 m to prevent groundwater contamination.

The application rates must be compatible with the evapotranspiration of the crop in the period, therefore depending on the type of crop and the climatic conditions. In arid zones, wastewater can be used for irrigation throughout the year. On the other hand, in wet areas, the application of wastewater in rainy periods can lead to anaerobic conditions and consequently odour and insect appearance problems.

Irrigation constitutes a treatment/disposal system that requires the largest surface area per unit of wastewater treated. On the other hand, it is the natural system with the highest efficiency. The plants are those mainly responsible for the removal of nutrients, such as phosphorus and nitrogen, while the microorganisms in the soil perform the removal of the organic substances. There is also a high removal of pathogenic organisms during the percolation through the soil (Mattos, 1998).

b) Rapid infiltration

The objective of the rapid infiltration system is to use the soil as a filtering medium for the wastewater. This system is characterised by the percolation of the wastewater, which is purified by the filtering action of the porous medium. The percolated wastewater may be used for groundwater recharge or be collected by underdrains or wells. The rapid infiltration method requires the lowest area within all the land disposal processes.

Wastewater is applied in shallow infiltration basins, from which wastewater percolates through the soil. The application is intermittent, in order to allow a resting period for the soil, during which the soil dries and re-establishes aerobic conditions. Due to the higher application rates, evaporation losses are small and most of the liquid percolates through the soil, undergoing treatment.

The application can be done by direct discharge (furrows, channels, perforated pipes) or by high capacity sprinklers. Vegetation growth may or may not occur, does not interfere with the efficiency of the process and is not part of the treatment objective (Coraucci Filho et al, 1999).

Figure 4.16 presents a flowsheet of a rapid infiltration system.

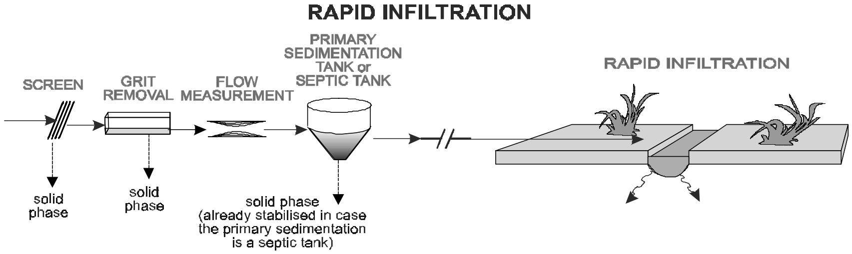


Figure 4.16. Typical flowsheet of a rapid infiltration system (other types of pre-treatment may be applied)

c) Subsurface infiltration

In subsurface infiltration systems, pre-settled or pre-treated sewage is applied below ground level. The infiltration sites are prepared in buried excavations, and filled in with a porous medium. The filling medium maintains the structure of the excavation, allows free sewage flow and provides sewage storage during peak flows. The sewage percolates through unsaturated soil, where additional treatment occurs. This treatment process is similar to rapid infiltration, the main difference residing in the application below ground level.

The subsurface infiltration systems have the following variants:

- *infiltration trenches or pits* (without final effluent: wastewater percolates to groundwater)
- *filtration trenches* (with final effluent: collection by underdrain system)

The subsurface infiltration systems are normally used following septic tanks (Figure 4.17) and, in some cases, after further treatment provided by systems such as anaerobic filters. The applicability is usually for small residential areas or rural dwellings.

d) Overland flow

Overland flow systems consist of the application of untreated (at least screened) or pre-treated wastewater in the upper part of sloped terraces, planted with water resistant grasses. Wastewater flows gently downwards, having contact with the root-soil system, in which biochemical reactions take place. Some evapotranspiration occurs, and the final effluent is collected at the lower end by drainage channels. Application is intermittent.

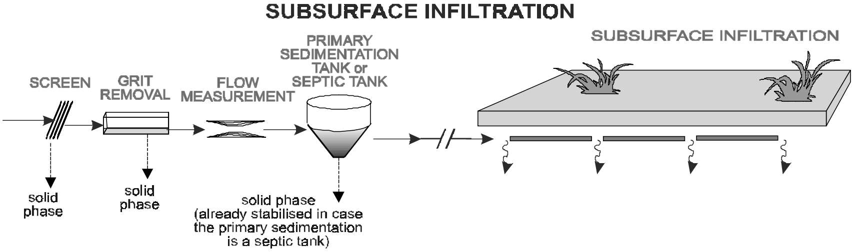


Figure 4.17. Typical flowsheet of a subsurface infiltration system

The soils should have a low permeability (e.g. clay). The slope should be moderate (between 2 and 8%).

The use of vegetation is essential to increase the absorption rate of the nutrients and the loss of water by transpiration. Besides this, the vegetation represents a barrier to the free surface flow of the liquid in the soil, increasing the retention of suspended solids and avoiding erosion. This gives better conditions for the development of the microorganisms that will carry out the various biochemical reactions. The vegetation should be perennial, water tolerant grasses. Local agricultural extension agents should be consulted.

Wastewater application can be done by sprinklers, gated pipes, slotted or perforated pipes or bubbling orifices (WPCF, 1990).

Figure 4.18 presents a flowsheet of an overland flow system.

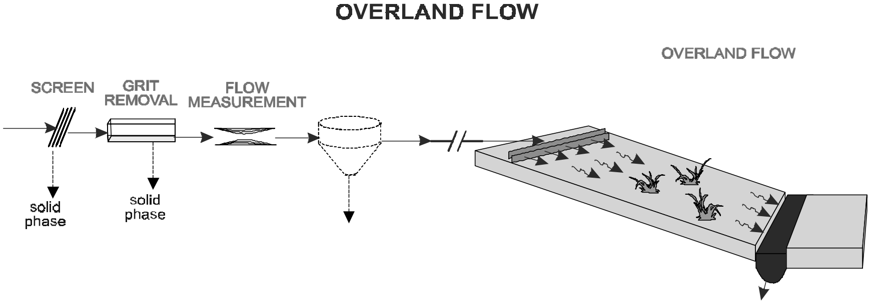


Figure 4.18. Typical flowsheet of an overland flow system

e) Constructed wetlands

Most of the following concepts were extracted from Marques (1999), OPS/OMS (1999) and mainly EPA (2000). **Natural** wetlands are areas inundated or saturated by surface or groundwater that support a vegetation adapted to these conditions. The natural wetlands include marshes, swamps and similar areas, that shelter diverse forms of aquatic life.

Constructed wetlands are purposely built wastewater treatment processes, which consist of ponds, basins or shallow canals (usually with a depth of less than 1.0 m) that shelter aquatic plants, and use biological, chemical and physical mechanisms to treat the sewage. The constructed wetlands usually have an impermeable layer of clay or synthetic membrane, and structures to control the flow direction, hydraulic detention time and water level. Depending on the system, they can contain an inert porous medium such as stones, gravel or sand.

Constructed wetlands are different from natural wetlands because of human interference, such as landfills, drainage, flow alterations and physical properties. The direct use of natural wetlands for sewage treatment has great environmental impacts and must not be encouraged.

There are basically two types of constructed wetlands:

- **Surface flow (free water surface) wetlands**. These resemble natural wetlands in appearance, because they have plants which can be floating and/or rooted (emergent or submerged) in a soil layer at the bottom, and water flows freely between the leaves and the stems of these plants. There can be open areas dominated by these plants or islands exerting habitat functions. Plant genera in use include: (a) emergent: *Typha*, *Phragmites*, *Scirpus*, (b) submerged: *Potamogeon*, *Elodea*, etc., (c) floating: *Eichornia* (water hyacinth), *Lemna* (duckweed). Native plants are preferred. These wetlands present a very complex aquatic ecology. They may or may not have a lined bottom, depending on the environmental requirements. Water depth is between 0.6 and 0.9 m for the vegetated zones (or less, in the case of certain emergent plants), and 1.2 to 1.5 m for free water zones. This type of wetlands is adequate to receive effluent from stabilisation ponds. In these conditions, they occupy an area between 1.5 to 3.0 m^2/inhab.
- **Subsurface flow wetlands (vegetated submersed bed systems)**. These do not resemble natural wetlands because there is no free water on the surface. There is a bed composed of small stones, gravel, sand or soil that gives support to the growth of aquatic plants. The water level stays below the surface of the bed, and sewage flows in contact with the roots and the rhizomes of the plants (where a bacterial biofilm is developed), not being visible or available for the aquatic biota. Plant genera that have been used are: *Typha*, *Scirpus*, *Carex* and *Phragmites*. The medium height is between 0.5 and 0.6 m and water depth is between 0.4 and 0.5 m. The gravel should have a size that allows the continuos flow of the sewage without clogging problems. A large part of the subsurface zone is anaerobic, with aerobic sites immediately adjacent to the rhizomes and roots. There is a lower potential for the generation of bad odours and the appearance of mosquitoes and rats. These wetlands are suited to receive effluents from septic tanks and anaerobic reactors, but not from stabilisation ponds, because of the presence of algae. For effluents from septic tanks, the land requirements are around 5.0 to 6.0 m^2/hab, and for effluents from anaerobic reactors, between 2.5 and 4.0 m^2/hab.

Regarding the direction of the water flow, the wetlands can be classified as:

- **Vertical flow**. Typically, a filter of sand or gravel planted with vegetation. At the bottom of the filter medium there is a series of underdrains that collect the treated sewage. The operation resembles the routine of a filter, with dosing and draining cycles, therefore, differing from the conventional conception of wetlands. With intermittent dosing, the flow is normally through unsaturated media.
- **Horizontal flow**. The most classical conception of constructed wetlands. May be with surface or subsurface flow.

Figure 4.19 illustrates the main variants of constructed wetlands.

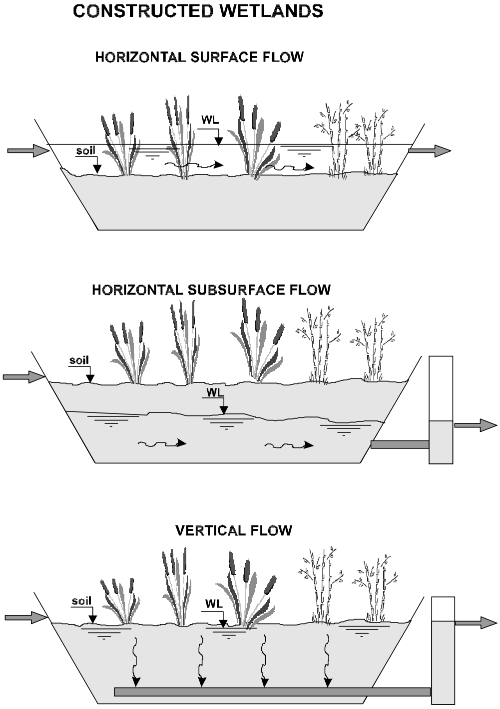


Figure 4.19. Diagram showing the main variants of constructed wetlands

Constructed wetlands do not perform well in the treatment of raw sewage. Some form of primary or secondary treatment (e.g. stabilisation ponds or anaerobic reactors) must precede this process (Figure 4.20). In the case of having previous secondary treatment, low values of BOD, SS and nitrogen can be reached.

The layout of wetlands is usually in cells in series or in parallel.

A surface flow system receiving the effluent from a stabilisation pond can operate for 10 to 15 years without the need to remove the material composed of

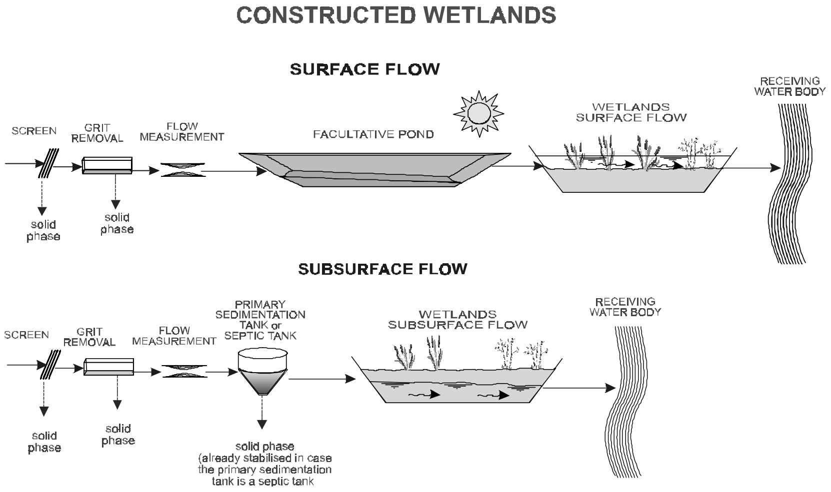


Figure 4.20. Typical flowsheets of wetlands systems

plants and inert settled solids. Of these solids, the largest part tends to accumulate at the inlet end of the unit.

The operation and maintenance of constructed wetlands is very simple. Besides the activities related with the preceding treatment, the maintenance of the wetlands is usually associated with the control of undesired aquatic plants and mosquitoes (which are not normally a problem in well designed and operated subsurface flow systems). The removal of the plants is not normally necessary, but a certain pruning or replanting can be necessary to maintain the desired flow conditions and treatment.

4.5.4 Anaerobic reactors

There are many variants of anaerobic reactors. This section presents only the two most widely applied for domestic sewage treatment:

- anaerobic filter (frequently treating septic tank effluents)
- UASB (upflow anaerobic sludge blanket) reactor

a) Septic tank – anaerobic filter system

The system of septic tanks followed by anaerobic filters (Figure 4.21) has been widely used in rural areas and in small sized communities. The septic tanks remove most of the suspended solids, which settle and undergo anaerobic digestion at the bottom of the tank.

The septic tank can be a single-chamber tank or a two-compartment tank (called an Imhoff tank). In the single chamber tank, there is no physical separation between the regions of the raw sewage solids sedimentation and bottom sludge digestion. The single chamber tanks can be single or in series.

In the Imhoff tank, settling occurs in the upper compartment (settling compartment). The settled solids pass through an opening at the bottom of the compartment and are directed to the bottom compartment (digestion compartment). The accumulated sludge then undergoes anaerobic digestion. The gases originating from the anaerobic digestion do not interfere with the settling process, as they cannot penetrate inside the sedimentation chamber.

Because septic tanks are sedimentation tanks (no biochemical reactions in the liquid phase), BOD removal is limited. The effluent, still with high organic matter concentration, goes to the anaerobic filter, where further removal takes place under anaerobic conditions. The filter is a biofilm reactor: the biomass grows attached to a support medium, usually stone. The following points are characteristic of anaerobic filters, differing from the trickling filters, which are also biofilm reactors (see Section 4.5.6):

- the liquid flow is upwards, i.e. the inlet is at the bottom and the outlet at the top of the anaerobic filter
- the anaerobic filter works submerged, i.e. the free spaces are filled with liquid
- the unit is closed
- the BOD load applied per unit volume is very high, which guarantees anaerobic conditions

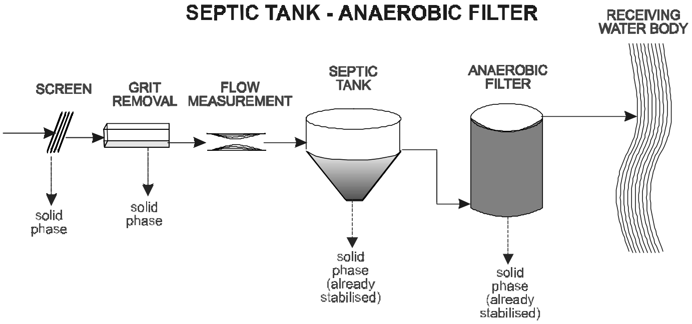


Figure 4.21. Typical flowsheet of a system of a septic tank followed by an anaerobic filter (liquid phase)

The efficiency of a septic tank – anaerobic filter is usually less compared with fully aerobic systems, although in most situations sufficient. The system has been widely used for small populations, but there has been a trend in terms of anaerobic treatment favouring the use of anaerobic sludge blanket reactors (described below).

Sludge production in anaerobic systems is very low. The excess sludge is already digested and can go directly to dewatering (in this system, typically by drying beds).

Being an anaerobic system, there is always a risk of generation of bad odours. However, proper design and operational procedures can contribute to the reduction of these risks. It should also be remembered that the septic tank and the anaerobic reactors are closed units.

b) Upflow anaerobic sludge blanket (UASB) reactors

The *upflow anaerobic sludge blanket* (UASB) reactors are currently the main trend in wastewater treatment in some warm-climate countries, either as single units, or followed by some form of post treatment.

In the UASB reactors, the biomass grows dispersed in the liquid, and not attached to a support medium, as in the case of anaerobic filters. When biomass grows it can form small granules, which are a result of the agglutination of various microorganisms. These small granules tend to serve as a support medium for other organisms. The granulation increases the efficiency of the system, but it is not essential for the working of the reactor, and is actually difficult to be obtained with domestic wastewater.

The concentration of the biomass in the reactor is very high, justifying the name of sludge blanket. Owing to this high concentration, the volume required for the UASB reactor is greatly reduced in comparison with all other treatment systems.

The liquid enters at the bottom, where it meets the sludge blanket, leading to the adsorption of the organic matter by the biomass. The flow is upward. As a result of the anaerobic activity, gases are formed (mainly methane and carbon dioxide) and the bubbles also present a rising tendency. The upper part of the anaerobic sludge blanket reactor presents a structure, whose functions are the separation and accumulation of the gas and the separation and return of the solids (biomass). In this way, the biomass is kept in the system (leading to high concentrations in the reactor), and only a minor fraction leaves with the effluent. This structure is called a three-phase separator, as it separates the liquid, solids, and gases. The form of the separator is frequently that of an inverted cone or pyramid.

The gas is collected in the upper part of the separator, in the gas compartment, from where it can be removed for reuse (energy from methane) or burning.

The solids settle in the upper part of the separator, in the settling compartment, and drain down the steeply inclined walls until they return to the reactor body. In this way, a large part of the biomass is retained by the system by simple gravitational return (differently from the activated sludge process, which requires pumping of the return sludge). Owing to the high solids retention, the hydraulic detention time can be low (in the order of 6 to 10 h). Because the gas bubbles do not penetrate the settling zone, the separation of the solids-liquid is not impaired. The effluent is relatively clarified when it leaves the settling compartment, and the concentration of the biomass in the reactor is maintained at a high level.

Figure 4.22 presents a schematic view of a UASB reactor. Various configurations are possible, including circular, square or rectangular tanks.

The sludge production is very low. The sludge wasted from the reactor is already digested and thickened, and may be simply dewatered in drying beds or other dewatering process. The dewaterability of the sludge is very good.

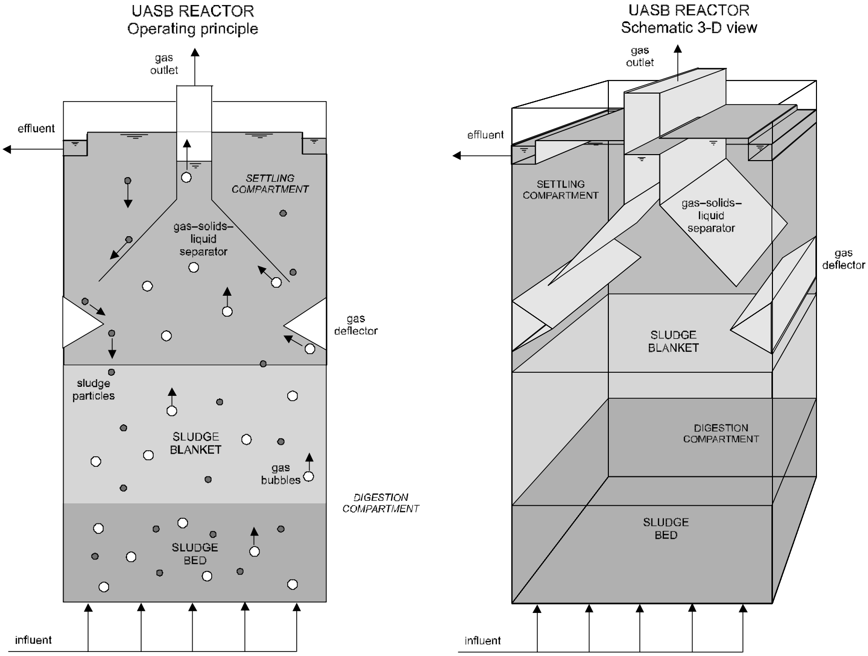


Figure 4.22. Schematics of a UASB reactor (working principle and schematic view)

The plant flowsheet is simplified even more by the fact that, differently from anaerobic filters and other systems, there is no need for primary settling.

Figure 4.23 presents the flowsheet of a wastewater treatment system comprised by a UASB reactor.

The risk of generation or release of malodours can be greatly reduced by a careful design, not only in the kinetics calculations, but mainly in the hydraulic aspects. The complete sealing-off of the reactor, including a submerged exit of the effluent and the reduction of weirs, contributes noticeably to the reduction of these risks. The adequate operation of the reactor also contributes to this.

A characteristic aspect of this process is the limitation in the BOD removal efficiency, which is around 70% , therefore lower than in most of the other systems. This must not be considered a disadvantage in itself, but as a characteristic of the process. To reach the desired efficiency, some form of post-treatment must follow the UASB reactors. The post-treatment process can be any of the secondary processes (aerobic or anaerobic) covered in this chapter, or a physical–chemical process, such as dissolved air flotation. The difference is that the post treatment stage is much more compact, since around 70% of the organic load has been previously removed in the anaerobic stage. Besides this, in the case of post-treatment processes that incorporate aeration, the consumption of energy is less, by virtue of the lower influent organic load to the aerated tank. Overall sludge production will be also lower. The total size (volume) of all the treatment units in the

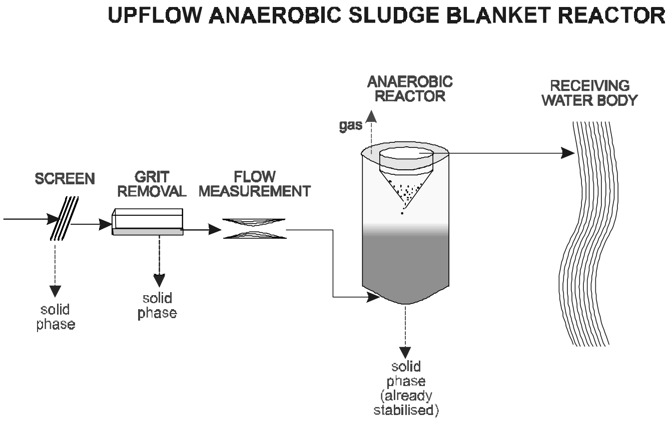


Figure 4.23. Typical flowsheet of a UASB reactor system (liquid phase)

UASB – post-treatment system will be slightly smaller compared with the alternative of no UASB reactor. Therefore, an economy in the construction and operation costs is usually obtained, in comparison with conventional systems not preceded by an anaerobic stage.

Figure 4.24 illustrates some of the main possible combinations of UASB reactors with post-treatment systems. It can be observed that in the UASB – activated sludge and UASB – biofilm aerobic reactor systems, the aerobic biological excess sludge is simply returned to the UASB reactor, where it undergoes digestion and thickening with the anaerobic sludge, dispensing with the separate digestion and thickening units for the aerobic sludge. Thus a large simplification in the overall flowsheet is obtained, including the liquid (sewage) and solid (sludge) phases.

4.5.5 Activated sludge system

There are many variants of the activated sludge process, and the present section covers only the main ones. Under this perspective, activated sludge systems may be classified according to the following categories:

- *Division according to the sludge age* (see concept of sludge age in item a below):
  - Conventional activated sludge
  - Extended aeration

- *Division according to flow:*
  - Continuous flow
  - Intermittent flow (sequencing batch reactors and variants)

- *Division regarding the treatment objectives:*
  - Removal of carbon (BOD)
  - Removal of carbon and nutrients (N and/or phosphorus)

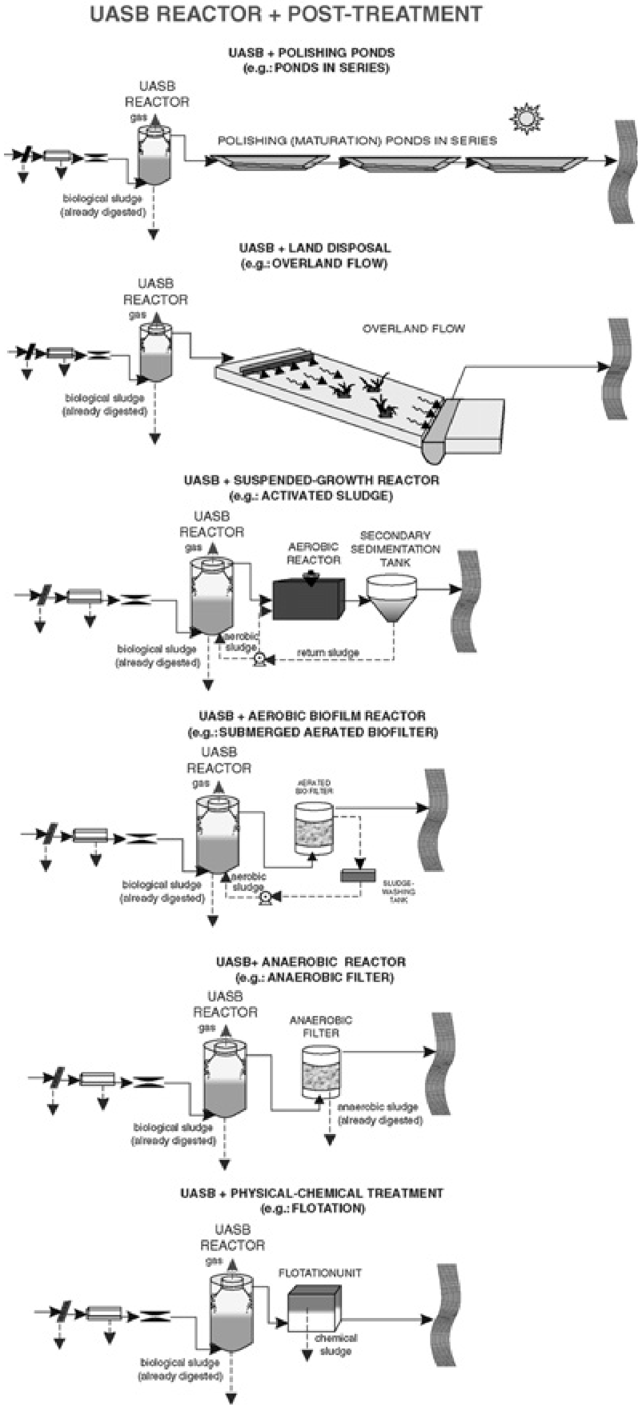


Figure 4.24. Examples of flowsheets of UASB reactors followed by post-treatment processes (liquid phase)

This section presents a brief description of the main variants of the activated sludge process, which are a combination of the above divisions:

- conventional activated sludge (continuous flow)
- extended aeration (continuous flow)
- sequencing batch reactors(intermittent operation)
- activated sludge with biological nitrogen removal
- activated sludge with biological nitrogen and phosphorus removal

All the systems above may be used as post-treatment of the effluent from anaerobic (UASB) reactors. In this case, primary sedimentation tanks (if existing) are substituted by the anaerobic reactor, and the excess sludge from the aerobic stage, if not yet stabilised, is pumped back to the anaerobic reactor, where it undergoes thickening and digestion. Biological nutrient removal is less efficient with the anaerobic pre-treatment, and adaptations or incorporation of physical-chemical treatment may be necessary.

a) Conventional activated sludge

When analysing the aerated pond systems described in the previous items, it becomes evident that a reduction of the volume required could be reached by increasing the biomass concentration in suspension in the liquid. The more bacteria there are in suspension, the greater the food consumption is going to be, thus the greater the assimilation of the organic matter present in the raw sewage.

Within this concept, analysing the previously described aerated ponds – settling ponds system, it can be observed that there is a storage of bacteria still active in the settling unit. If part of these bacteria is returned to the aeration unit, the concentration of the bacteria in this unit will be greatly increased. This is the basic principle of the activated sludge system, in that the solids are recycled by pumping, from the bottom of the settling unit, to the aeration unit. The following items are therefore essential in the activated sludge system (liquid flow) (see Figure 4.25):

- *aeration tank (reactor)*
- *settling tank (secondary sedimentation tank, also called final clarifiers)*
- *pumps for the sludge recirculation*
- *removal of the biological excess sludge*

The biomass can be separated in the secondary sedimentation tank because of its property of flocculating. This is due to the fact that many bacteria have a gelatinous matrix that permits their agglutination. The floc has larger dimensions, which facilitates settling (see Figure 4.26).

The concentration of the suspended solids in the aeration tank of the activated sludge system is more than 10 times greater than in a complete-mix aerated pond. The detention time of the liquid is very low, of the order of 6 to 8 hours in the conventional activated sludge system, which implies that the aeration tank volume is very small. However, owing to the sludge recirculation, the solids (biomass)

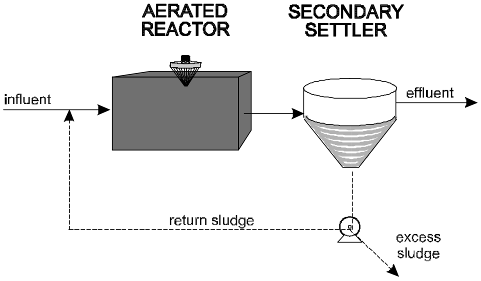


Figure 4.25. Schematics of the units of the biological stage of the activated sludge system

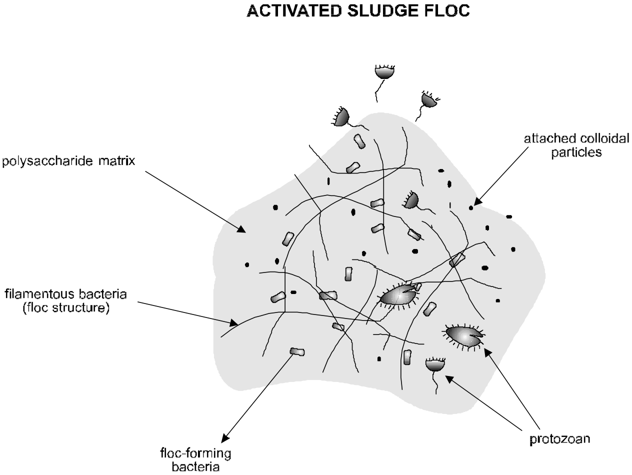


Figure 4.26. Schematics of bacteria and other microorganisms forming an activated sludge floc

stay in the system for a time longer than that of the liquid. The retention time of the solids in the system is called **sludge age** or **solids retention time**, which is of the order of 4 to 10 days in the conventional activated sludge system. It is this longer retention of the solids in the system that guarantees the high efficiency of the activated sludge, as the biomass has sufficient time to metabolise practically all of the organic matter in the sewage. In the UASB reactor described in the previous section, the biomass is returned to the digestion compartment by gravity from the settling compartment situated on top of the digestion compartment and, therefore, the solids retention time is also greater than the hydraulic detention time.

In the activated sludge system, the tanks are typically made of concrete, different from stabilisation ponds. To save in terms of energy for the aeration, part of the organic matter (in suspension, settleable) of the sewage is removed before the aeration tank, in the primary sedimentation tank. Therefore, the conventional activated sludge systems have as an integral part also the *primary treatment* (Figure 4.27).

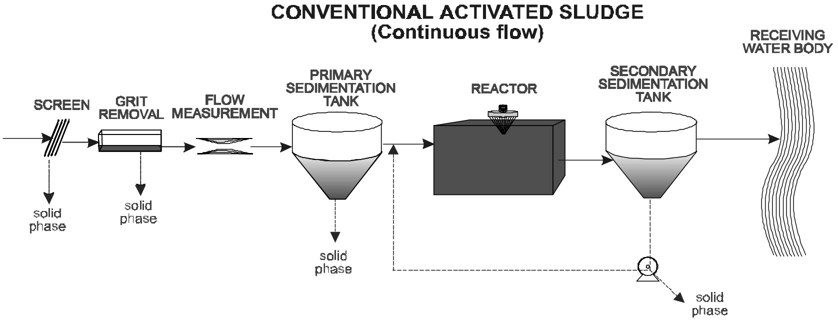


Figure 4.27. Typical flowsheet of the conventional activated sludge system (liquid phase)

Owing to the continuous arrival of food in the form of BOD to the aeration tank, bacteria grow and reproduce continuously. If an indefinite population growth were allowed, the bacteria would reach excessive concentrations in the aeration tank, making the transfer of oxygen to all bacterial cells difficult. Besides this, the secondary sedimentation tank would become overloaded, the solids would not settle well and they would start to leave with the final effluent, thus deteriorating its quality. To maintain the system in equilibrium, it is necessary to draw approximately the same quantity of biomass that has increased by reproduction. This is, therefore, the *biological excess sludge* that can be wasted directly from the reactor or the recirculation line. The excess sludge must undergo additional treatment in the sludge treatment line.

The conventional activated sludge system has low land requirements and has very good removal efficiencies. However, the flowsheet of the system is more complex than in most other treatment systems, requiring more skill for its control and operation. Energy costs for aeration are higher than for aerated ponds.

b) Extended aeration

In the *conventional* activated sludge system, the average retention time of the sludge in the system is between 4 to 10 days. With this sludge age, the biomass removed in the excess sludge still requires a stabilisation stage in the sludge treatment. This is due to the high level of biodegradable organic matter in their cell composition.

However, if the biomass is retained in the system for a longer period, with a sludge age around 18 to 30 days (thus the name *extended aeration*), receiving the same BOD load as the conventional system, there is a lower food availability for the bacteria. Owing to the higher sludge age, the reactor has a greater volume (the liquid detention time is around 16 to 24 hours). Therefore, there is less organic matter per unit volume of the aeration tank, and per unit microbial mass. Consequently, in order to survive, the bacteria start to use in their metabolic processes the organic matter from their cellular material. This cellular organic matter is converted into carbon dioxide and water through respiration. This corresponds to a stabilisation (digestion) of the biomass, taking place in the aeration tank. While in the conventional system the sludge stabilisation is carried out separately (in sludge digesters in the sludge treatment line), in extended aeration systems the digestion is done concurrently with the BOD stabilisation in the reactor.

As there is no need to stabilise separately the excess biological sludge, the generation of another type of sludge in the system that would require subsequent treatment is also avoided. Consequently, extended aeration systems do not usually have primary sedimentation tanks. A great simplification in the flowsheet is obtained: there are no primary sedimentation tanks and no sludge digestion units (Figure 4.28).

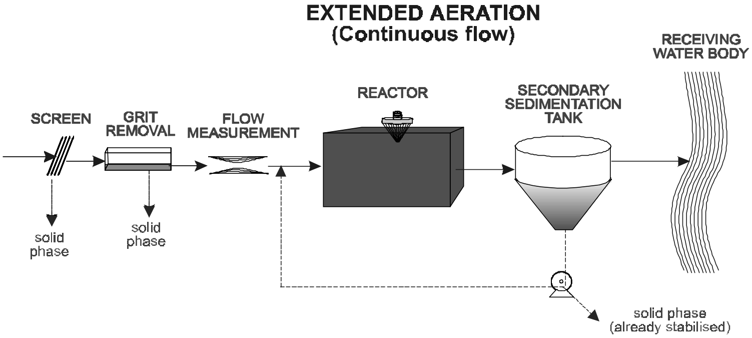


Figure 4.28. Flowsheet of the extended aeration system (liquid phase)

The consequence of this simplification in the system is the energy expenditure for aeration, which is due, not only to the removal of the incoming BOD, but also for the aerobic digestion of the sludge in the reactor. On the other hand, the reduction in the availability of food and its practically complete assimilation by the biomass makes extended aeration one of the most efficient sewage treatment processes in terms of BOD removal.

c) Intermittent operation (sequencing batch reactors)

The activated sludge systems described above are of continuous flow with relation to the sewage, that is, the sewage is always entering and leaving the reactor. However, there is a variant of the system which has *intermittent* flow and operation, also called a *sequencing batch reactor* (SBR).

The principle of the activated sludge process with intermittent operation consists in the incorporation of all the units, processes and operations normally associated to the conventional activated sludge (primary sedimentation, biological oxidation, secondary sedimentation, sludge pumping) within a single tank. Using only a single tank, these processes and operations become sequences in time and not separated units, such as in conventional processes with continuous flow. The process of activated sludge with intermittent flow can be used in the conventional or in the extended aeration sludge ages, although the latter is more common, due to its greater operational simplicity. In the extended aeration mode, the tank also incorporates the role of the sludge digestion (aerobic) unit.

The process consists of a complete-mix reactor where all the treatment stages occur. This is accomplished by the establishment of operating cycles with defined duration. The biological mass stays in the reactor, eliminating the need for separate sedimentation and sludge pumping. The retention of biomass occurs because it is not withdrawn with the supernatant (final effluent) after the sedimentation stage, remaining in the tank. The normal treatment cycle is composed of the following stages:

- *Fill* (entrance of the influent in the reactor)
- *React* (aeration/mixture of the liquid/biomass contained in the reactor)
- *Settle* (sedimentation and separation of the suspended solids from the treated sewage)
- *Draw* (removal of the supernatant, which is the treated effluent from the reactor)
- *Idle* (cycle adjustment and removal of the excess sludge)

The usual duration of each stage within the cycle can be altered as a function of the influent flow variations, the treatment needs and the sewage and biomass characteristics.

The wasting of excess sludge generally occurs during the last stage (idle), whose purpose is to allow the adjustment of the stages within the operating cycles of each reactor. However, as this stage is optional or may be short, sludge wasting can happen in other phases of the process. The sludge wasting quantity and frequency are established in function with the performance requirements, in the same way as in the conventional continuous flow processes.

The flowsheet of the process is greatly simplified due to the elimination of various units, compared with the continuous flow activated sludge systems. The only units in an SBR operating in the extended aeration mode are: *screens*, *grit chamber, reactors, sludge thickener (optional) and sludge drying* (Figure 4.29). With domestic sewage, which arrives at the treatment plant 24 hours per day, more than one tank is necessary, since only the tank in the fill stage is apt to receive the incoming sewage.

There are some variants of the sequencing batch reactor systems related to its operation (continuous feeding and discontinuous supernatant withdrawal) as well as in the sequence and duration of the stages within each cycle. These variations may lead to additional simplifications in the process or to biological nutrient removal.

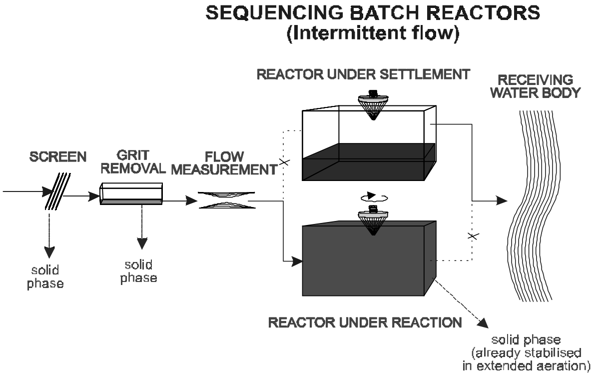


Figure 4.29. Flowsheet of a sequencing batch reactor system in the extended aeration mode (liquid phase)

d) Activated sludge with biological nitrogen removal

The activated sludge system is capable of producing, without process alterations, a satisfactory conversion of ammonia to nitrate (**nitrification**). In this case, *only ammonia and not nitrogen is removed*, as there is only a conversion of the nitrogen form. Nitrification occurs almost systematically in warm-climate regions unless there is some environmental problem in the aeration tank, such as lack of dissolved oxygen, low pH, little biomass or the presence of toxic or inhibiting substances.

Biological nitrogen removal is achieved in the absence of dissolved oxygen, but presence of nitrates (called *anoxic conditions*). In these conditions, a group of bacteria uses the nitrates in their respiration process, converting them to nitrogen gas, which escapes into the atmosphere. This process is called **denitrification**. To achieve denitrification in the activated sludge, process modifications are necessary, including the creation of anoxic zones and possible internal recycles.

In spite of nitrogen removal being considered as tertiary treatment, biological removal is presented in this item of secondary treatment, as it consists of essentially biological processes and can be achieved through adaptations in the flowsheet of the activated sludge process at a secondary level.

In activated sludge systems where nitrification occurs (mainly in warm-climate regions), it is interesting to induce denitrification to take place intentionally in the reactor. The reasons are usually associated to purely operational aspects, as well as to the final effluent quality:

- *Savings in oxygen* (energy economy in the aeration). Under anoxic conditions, facultative bacteria remove BOD by using the nitrate in their respiratory processes, therefore leading to an economy of oxygen, or in other words, in the energy used for aeration. This economy partially compensates the energy expenditure for nitrification, which occurs, necessarily, under aerobic conditions.

- *Savings in alkalinity* (preservation of the buffering capacity). During nitrification, H^+ ions are generated and alkalinity is consumed, which can lead to a decrease in the pH in the aeration tank. Conversely, denitrification consumes H^+ and generates alkalinity, partially compensating the pH reduction mechanisms that occur in nitrification.
- *Operation of the secondary sedimentation tank* (to avoid rising sludge). If denitrification occurs in the anoxic conditions in the secondary sedimentation tanks, there will be a production of small nitrogen gas bubbles. These bubbles tend to adhere to the settling flocs, dragging them to the surface and causing a loss of biomass and deterioration in the final effluent quality.
- *Nutrient control* (eutrophication). The reduction of the nitrogen levels is important when the effluent is discharged into sensitive water bodies that are subjected to eutrophication (see Chapter 3).

The main process variants for nitrification and denitrification combined in a single reactor are listed below. These systems are not described in this chapter, but they are detailed in Chapters 35 and 36 of this book.

- Pre-denitrification (nitrogen removal with carbon from the raw sewage)
- Post-denitrification (nitrogen removal with carbon from endogenous respiration)
- Bardenpho four-stage process
- Oxidation ditches
- Sequencing batch reactors

e) Activated sludge with biological nitrogen and phosphorus removal

Although phosphorus removal can be considered as a tertiary treatment, biological removal is presented in this section on secondary treatment because it consists of essentially biological mechanisms and can be achieved through adaptations of the activated sludge process flowsheet at a secondary level.

It is *essential to have anaerobic and aerobic zones* in the treatment line for the biological removal of phosphorus. The anaerobic zone is considered a *biological selector* for the phosphorus accumulating organisms. This zone allows an advantage in terms of competition for the phosphorus accumulating organisms, since they can assimilate the substrate of this zone before the other microorganisms. In this way, the anaerobic zone gives good conditions for the development or selection of a large population of phosphorus accumulating organisms in the system, which absorb substantial quantities of phosphorus from the liquid, much higher than the normal metabolic requirements. When the biological excess sludge is wasted from the system, phosphorus is removed, since it is present at high concentrations in the phosphorus accumulating organisms that are part of the withdrawn sludge.

Some of the main processes used for both nitrogen and phosphorus removal in the activated sludge system are listed below. Processes employed to remove just phosphorus are not listed due to the difficulties that these undergo with the presence of nitrates in the anaerobic zone. Nitrification occurs almost systematically in the activated sludge plants in warm-climate regions. If efficient denitrification is not provided in the reactor, substantial quantities of nitrates are returned to the anaerobic zone through the recycle lines, impairing the maintenance of strict anaerobic conditions in the anaerobic zone. For this reason, nitrogen removal is encouraged, even if, in terms of the water body requirements, there is only the need to remove phosphorus.

The processes are only listed here, but are further detailed in Chapters 35 and 36. The literature presents a diverging nomenclature in relation to some processes, as a function of variations between commercial and scientific designations.

- A^2O process (three-stage Phoredox)
- Five-stage Bardenpho process (Phoredox)
- UCT process
- Modified UCT process
- Sequencing batch reactors

If higher efficiencies are still desired for phosphorus removal, effluent polishing can adopted. Methods employed are:

- *addition of coagulants* (metallic ions): phosphorus precipitation
- *effluent filtration*: removal of the phosphorus present in the suspended solids in the effluent
- *combination of the addition of coagulants and filtration*

These physical–chemical polishing methods can also be employed for P removal from other biological wastewater treatment process, and not only from the activated sludge process.

4.5.6 Aerobic biofilm reactors

In this section, the aerobic units are biofilm reactors, in which the biomass grows attached to a support medium. There are many variants within this broad concept, and the following ones are presented in this section:

- Low rate trickling filter
- High rate trickling filters
- Submerged aerated biofilters
- Rotating biological contactors

All systems may be used as post-treatment of the effluent from anaerobic reactors. In this case, primary sedimentation tanks are substituted by the anaerobic reactor, and the excess sludge from the aerobic stage, if not yet stabilised, is pumped back to the anaerobic reactor, where it undergoes thickening and digestion.

a) Low rate trickling filter

A trickling filter consists of a coarse material bed, such as stones, gravel, blast furnace slag, plastic material or other, over which the wastewater is applied, in the form of drops or jets. After application, the wastewater percolates in the direction of the drains at the bottom. This percolation allows the bacterial growth on the surface of the support medium, forming an attached biofilm. With the passage of the wastewater, there is a contact between the microorganisms and the organic matter.

The trickling filters are aerobic systems because the air circulates between the empty spaces between the stones, supplying the oxygen for the respiration of the microorganisms. The ventilation is usually natural.

Wastewater is usually applied over the medium through rotating distributors, moved by the hydraulic head of the wastewater. The liquid percolates rapidly through the support medium. However, the microbial film adsorbs the organic matter, which stays adhered for a time sufficient for its stabilisation (see Figure 4.30).

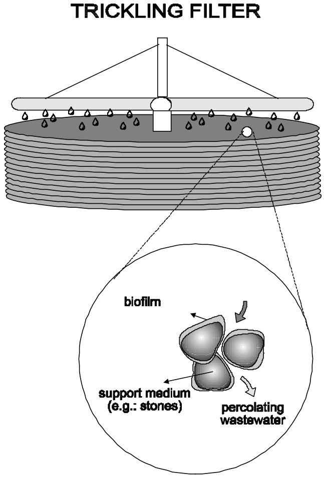


Figure 4.30. Schematics of a trickling filter

The filters are normally circular and can be of various sizes in diameter (several metres). Contrary to what the name suggests, the primary function of the filter is not to filter. The diameters of the stones used are of the order of a few centimetres, which allows a large void space that is inefficient for the act of filtration by screening. The function of the medium is only to supply support for the formation of the microbial film. There are also synthetic media of various materials and forms, which present the advantage of being lighter than stone, besides having a higher surface area. However, the synthetic media are more expensive. The savings in construction costs must be analysed together with the greater expenditure in purchasing the synthetic media.

With the biomass growth on the surface of the stones, the empty spaces tend to decrease, increasing the liquid velocity through the pores. When growth reaches a certain level, the velocity causes a shearing stress that dislodges part of the attached material. This is a natural form of controlling the microbial population in the medium. The dislodged sludge must be removed by the secondary sedimentation tanks to reduce the level of suspended solids in the final effluent.

The applied BOD load per unit area and volume is lower in the *low rate* trickling filters. Therefore, food availability is low, which results in a partial self digestion of the sludge (self consumption of the cellular organic matter) and a higher BOD removal efficiency in the system. This is analogous to what happens in the extended aeration activated sludge system. This lower BOD load per surface unit of the tank is associated with higher area requirements when compared with high rate systems, which are described in the following item. The low rate trickling filters are still more efficient in the removal of ammonia by nitrification.

The low rate system is conceptually simple. Although the efficiency of the system is comparable with the conventional activated sludge system, the operation is simpler, although less flexible. The trickling filters have a lower capacity to adjust to influent variations, besides requiring a slightly higher total area. In terms of energy consumption, the filters present a very low consumption in relation to the activated sludge system. Figure 4.31 presents a typical flowsheet of low rate trickling filters.

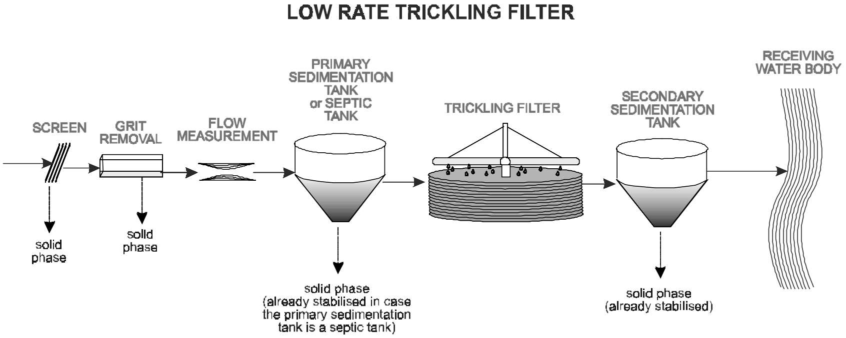


Figure 4.31. Typical flowsheet of a low rate trickling filter (liquid phase)

b) High rate trickling filters

High rate trickling filters are conceptually similar to the low rate filters. However, because the high rate units receive a higher BOD load per unit volume of the bed, there are the following main differences: (a) the area requirements are lower; (b) there is a slight reduction in the organic matter removal efficiency; (c) sludge is not digested in the filter.

Another difference is with respect to the existence of a recirculation of the final effluent. This is done with the main objectives of: (a) maintaining an approximately uniform flow during all the day (at night, the distributors could not rotate, due to the low flow, eventually drying the sludge); (b) balancing the influent load; (c) giving a new contact chance of the effluent organic matter with the biomass; and (d) bringing dissolved oxygen into the incoming liquid. The difference from the activated sludge system is that the recirculation of the high rate filters is of the liquid effluent and not of the sludge from the secondary sedimentation tanks (Figure 4.32).

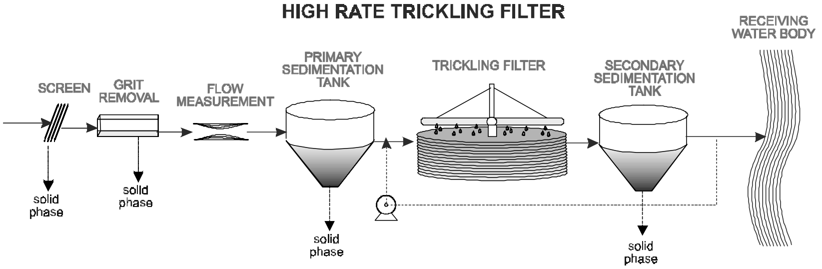


Figure 4.32. Typical flowsheet of a high rate trickling filter (liquid phase)

Another way of improving the efficiency of trickling filters or to treat wastewaters with high concentrations of organic waste is by using two filters in series. This is called a two-stage trickling filter system. There are various possible configurations with different forms of effluent recirculation.

Some of the limitations of stone-bed trickling filters when operating with high organic loads refer to clogging of the void spaces, due to the excessive growth of the biofilm. In these conditions, flooding (ponding) and system failures may occur.

If land availability is of concern, a careful consideration of the filter media must be exercised. The most commonly used material is still stones and gravel. However, the empty volume is limited in a trickling filter with stones, thus restricting the air circulation in the filter and consequently the quantity of oxygen available for the microorganisms and the quantity of wastewater that can be treated. The specific surface area (exposure area per unit volume of the medium) is also low, reducing the available sites for biofilm attachment and growth.

To overcome these limitations, other materials can be used. These materials include corrugated plastic modules, plastic rings and others. These materials offer larger surface areas for the bacterial growth (approximately double that of typical stones), besides the significant increase in the empty spaces for air circulation. These materials are much lighter than stones (around 30 times), which allows the filters to be much higher without causing structural problems. While filters with stones are usually less than 3 metres in height, filters with synthetic media can be more than 6 metres high, substantially reducing the land required for the installation of the filters.

c) Submerged aerated biofilters

A submerged aerated biofilter consists of a tank filled with a porous material, through which wastewater and air permanently flow. In almost all of the existing processes, the porous medium is maintained under total immersion. The biofilter is a three-phase reactor composed of (Gonçalves, 1996):

- *Solid phase:* consists of a support medium and microorganism colonies that develop in the form of a biofilm

- *Liquid phase:* consists of the liquid in permanent flow through the porous medium
- *Gas phase:* formed by artificial aeration and, in a reduced scale, by the gaseous by-products of the biological activity

The airflow in the submerged aerated biofilter is always upflow, while the liquid flow can be upflow or downflow.

Biofilters with granular media remove, in the same reactor, soluble organic compounds and suspended solids from the wastewater. Besides serving as support medium for the microorganisms, the granular material performs as an effective filter. In this type of biofilter, periodic washing is necessary to eliminate the accumulated biomass, reducing the hydraulic head losses through the medium. During washing, the feeding with the wastewater is interrupted, and various sequential hydraulic discharges are made with air and cleaning water (Gonçalves, 1996).

The flowsheet of a system composed of a submerged aerated filter is presented in Figure 4.33. The two sources of sludge generation are the primary sedimentation tanks and the washing of the biofilter. The sludge from the washing is collected in a storage tank and pumped to the primary sedimentation tank for clarification outside peak flow times. Therefore, the sludge sent to the sludge treatment stage is a mixed sludge, comprising primary sludge and biological sludge (Gonçalves, 1996). Submerged aerated biofilters are also being successfully applied for the post-treatment of UASB reactors. The aerobic sludge is returned to the UASB reactor, where it undergoes thickening and digestion, thereby simplifying substantially the overall flowsheet (see Figure 4.24) (Chernicharo et al, 2001).

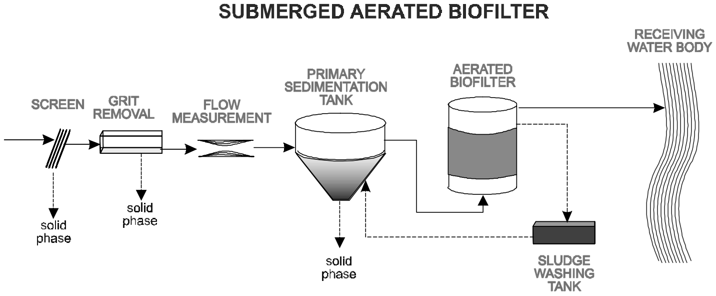


Figure 4.33. Typical flowsheet of a conventional submerged aerated biofilter system (liquid phase)

Submerged aerated biofilters achieve good nitrification efficiencies and can be modified for the biological removal of nitrogen, through the incorporation of an anoxic zone in the reactor (zone below the air injection).

d) Rotating biological contactors

The most widely version of rotating biological contactors are the biodiscs, a process that consists of a series of spaced discs, mounted on a horizontal axis. The discs rotate slowly and maintain at each instant around half the surface immersed in the sewage and the other half exposed to the air. Biomass grows attached to the discs, forming a biofilm (see Figure 4.34).

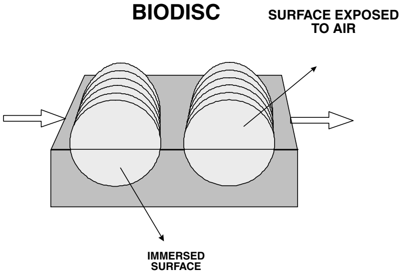


Figure 4.34. Schematics of a tank with biodiscs

The discs usually are less than 3.6 metres in diameter and are generally constructed of low weight plastic. When the system is put into operation, the microorganisms of the sewage start to adhere to the rotating surfaces, where they grow until the entire disk surface is covered with a fine biological layer, a few millimetres thick. As the discs rotate, the part of the disc exposed to the air brings a thin layer of wastewater, allowing oxygen absorption through the drops and percolation on the surface of the discs. After the discs complete a rotation, this film mixes itself with the wastewater, bringing still some oxygen and mixing the partially and fully treated sewage. With the passage of the microorganisms attached to the disc surface through the wastewater, they absorb a new quantity of organic matter that is used as food.

When the biological layer reaches an excessive thickness, it detaches from the discs. Part of these detached microorganisms is maintained in suspension in the liquid due to the movement of the disk, which increases the efficiency of the system.

The main purposes of the discs are:

- to serve as the surface for microbial film growth;
- to promote the contact between the microbial film and the sewage;
- to maintain the biomass that detached from the discs in suspension in the liquid;
- to promote the aeration of the sewage that is adhered to the disc and the sewage immersed in the liquid.

The growth of the biofilm is similar in concept to the trickling filter, with the difference that the microorganisms pass through the sewage, instead of the sewage passing through the microorganisms, like in the filters. Like the trickling filter process, secondary sedimentation tanks are also necessary, with the objetive of removing the suspended solids.

Biodisc systems are mainly used for the treatment of sewage from small communities. Due to the limitations in the diameter of the discs, it would be necessary to have a large number of discs, often impractical, for the treatment of high flows. The system presents good BOD removal efficiency, although it sometimes shows signs of instability. DO in the effluent may be high. The operational level is moderate and the construction costs are usually high. The flowsheet of the system is presented in Figure 4.35.

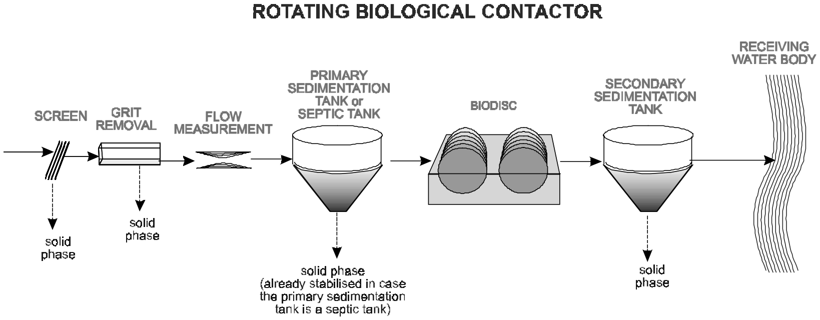


Figure 4.35. Typical flowsheet of a biodisc system (liquid phase)

4.6 REMOVAL OF PATHOGENIC ORGANISMS

The main processes used for removal of pathogenic organisms are listed in Table 4.6. Only short comments are made, since the removal of pathogenic organisms, especially by artifical methods, is outside the scope of this book. However, maturation ponds are treated in detail in Chapter 17.

The processes listed above are capable of reaching a coliform removal of 99.99% or more. Regarding pathogenic organisms, bacteria removal efficiency is very high (equal to or higher than coliform removal), and the other pathogens (protozoa, virus, helminths) are usually high, but variable, depending on the removal mechanism and the resistance of each species.

4.7 ANALYSIS AND SELECTION OF THE WASTEWATER TREATMENT PROCESS

4.7.1 Criteria for the analysis

The decision regarding the wastewater treatment process to be adopted should be derived from a balance between technical and economical criteria, taking into account quantitative and qualitative aspects of each alternative. If the decision regarding economic aspects may seem relatively simple, the same may not be the case with the financial aspects. Besides, the technical points are in many cases intangible and in a large number of situations, the final decision can still have

Table 4.6. Main processes for the removal of pathogenic microorganisms in sewage treatment

Type	Process	Comment
Natural	Maturation ponds	• Shallow ponds, where the penetration of solar ultraviolet radiation and unfavourable environmental conditions causes a high mortality of the pathogens. • The maturation ponds do not need chemical products or energy, but require large areas. • They are highly recommended systems (if there is area available), owing to their great simplicity and low costs.
	Land treatment (infiltration in soil)	• The unfavourable environmental conditions in the soil favour the mortality of the pathogens. • In slow-rate systems, there is the possibility of plant contamination, depending on the type of application. • Chemical products are not needed. • Requires large areas.
Artificial	Chlorination	• Chlorine kills pathogenic microorganisms (although protozoan cysts and helminth eggs are not much affected). • High dosages are necessary, which may increase operational costs. The larger the previous organic matter removal, the lower the chlorine dosage required. • There is a concern regarding the generation of toxic by-products to human beings. However, the great benefit to public health in the removal of pathogens must be taken into consideration. • The toxicity caused by the residual chlorine in the water bodies is also of concern. The residual chlorine must have very low levels, frequently requiring dechlorination. • There is much experience with chlorination in the area of water treatment in various developing countries.
	Ozonisation	• Ozone is a very effective agent for the removal of pathogens. • Ozonisation is usually expensive, although the costs are reducing, making this alternative a competitive option in certain specific circumstances. • There is less experience with ozonisation in most developing countries.
	Ultraviolet radiation	• Ultraviolet radiation, generated by special lamps, affects the reproduction of the pathogenic agents. • Toxic by-products are not generated. • Ideally, the effluent must be well clarified for the radiation to penetrate well in the liquid mass. • This process has recently shown substantial development, which has made it more competitive or more advantageous than chlorination in various applications.

Table 4.6 (*Continued*)

Type	Process	Comment
	Membranes	• The passage of treated sewage through membranes of minute dimensions (e.g. ultrafiltration, nanofiltration) constitutes a physical barrier for the pathogenic microorganisms, which have larger dimensions than the pores. • The process is highly interesting and does not introduce chemical products into the liquid. • The costs are still high, but they have been reducing significantly in recent years.

subjectivity. Criteria or weightings can be attributed to the various aspects connected essentially with the reality in focus, so that the selection really leads to the most adequate alternative for the system under analysis. There are no such generalised formulas for this, and common sense and experience when attributing the relative importance of each technical aspect are essential. While the economic side is fundamental, it needs to be remembered that the best alternative is not always the one that simply presents the lowest cost in economic–financial studies.

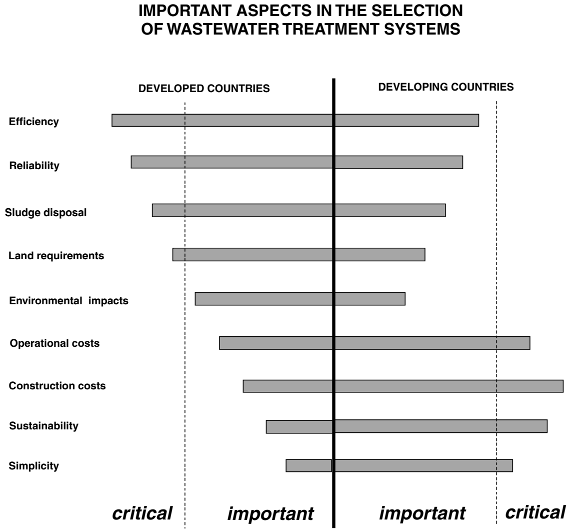


Figure 4.36. Critical and important aspects in the selection of wastewater treatment systems in developed and developing regions (von Sperling, 1996)

Figure 4.36 presents a comparison between important aspects in the selection of treatment systems, analysed in terms of developed and developing regions (von Sperling, 1996). The comparison is unavoidably general, owing to the specificity of each region or country and the high contrasts usually observed in developing countries. The items are organised in a decreasing order of importance for the developed regions. In these regions, the critical items are usually: efficiency, reliability, sludge disposal aspects and land requirements. In developing regions, these first items are organised in a similar manner of decreasing importance, but have a lower magnitude, in comparison with the developed regions. The main difference resides in what are considered the critical items for the developing regions: construction costs, sustainability, simplicity and operational costs. These items are of course important in developed regions, but cannot be usually considered critical.

Table 4.7 presents general factors to be taken into account when selecting and evaluating unit operations and processes in wastewater treatment, while Table 4.8 presents environmental aspects to be considered in the selection of processes for wastewater treatment and sludge management.

Each of these factors must be evaluated in terms of the local conditions and the technology employed. The reliability of the monitoring system must also be considered.

4.7.2 Comparison between the wastewater treatment systems

Presented below is a comparative analysis between the main wastewater treatment systems (liquid and solid phases) applied to domestic sewage. The analysis is summarised in various tables and figures:

- *Quantitative comparison* (Table 4.9): average effluent concentrations and typical removal efficiencies of the main pollutants of interest in domestic sewage
- *Quantitative comparison* (Table 4.10): typical characteristics of the main sewage treatment systems, expressed in per-capita values
- *Diagrammatic comparison* (Tables 4.11 to 4.13): capacity of the various sewage treatment systems in consistently reaching different quality levels in terms of BOD, COD, SS, ammonia, total nitrogen, total phosphorus, faecal coliforms and helminth eggs (based on von Sperling & Chernicharo, 2002)
- *Diagrammatic comparison* (Tables 4.14 to 4.18): per capita values of land requirement, power for aeration, production of sludge to be disposed of, construction costs and operation and maintenance costs, for various sewage treatment processes.
- *Qualitative comparison* (Table 4.19): a qualitative comparative analysis that covers various relevant aspects in the evaluation of the sewage treatment systems. The aspects of efficiency, economy, process and environmental problems are analysed.
- *Description* (Table 4.20): a list of the basic equipment usually necessary in the main sewage treatment systems.
- *Advantages and disadvantages* (Table 4.21): main advantages and disadvantages of the various sewage treatments systems. This analysis is principally oriented for the comparison of the processes within the same system, although it still permits, within certain limitations, the comparison between distinct systems.

Table 4.7. Important factors to be considered when evaluating and selecting unit operations and processes

Condition	Factor
Process applicability	The applicability of a process is evaluated based on past experience, published data, data from operating works and from pilot plants. If new or unusual conditions are found, pilot scale studies are necessary.
Applicable flow	The process must be adequate for the expected flow range.
Acceptable flow variation	The majority of the operations and processes must be designed to operate over a wide flow range. The highest efficiency is usually obtained with a constant flow, although some variation can be tolerated. Equalisation of the flow could be necessary if the variation is very large.
Influent characteristics	The characteristics of the influent wastewater affect the process types to be used (e.g. chemical or biological) and the requirements for their adequate operation.
Inhibiting or refractory compounds	What are the constituents in the wastewater that could be inhibitory or toxic, and under what conditions? What constituents are not affected during the treatment?
Climatic aspects	Temperature affects reaction rates of most chemical and biological processes. Temperature can also affect the physical operation of the units. High temperatures can accelerate odour generation.
Process kinetics and reactor hydraulics	The design of the reactor is based on reaction kinetics. Kinetic data are normally obtained from experience, literature or pilot studies. Reactor configuration also plays an important role in the removal of some constituents.
Performance	Performance is normally measured in terms of the quality of the effluent, which should be consistent with the discharge requirements and/or the discharge standards.
Treatment residuals	The type, quality and quantity of the solids, liquids and gaseous by-products need to be known or estimated. If necessary, undertake a pilot study.
Sludge processing	Are there limitations that could make the sludge processing and disposal expensive or unfeasible? What is the influence in the liquid phase of the loads recycled from the sludge treatment units? The selection of the sludge-processing system must be done in parallel with the selection of the treatment processes of the liquid phase.
Environmental constraints	Environmental factors, such as prevailing winds and proximity to residential areas could restrict the use of certain processes, especially when odours are released. Noise and traffic could affect the selection of the works location.
Chemical product requirements	What resources and quantities must be guaranteed for the satisfactory operation of the unit for a long period of time?
Energy requirements	The energy requirements, together with the probable future energy costs, need to be estimated if it is desired to design cost-effective treatment systems.
Requirements of other resources	What additional resources are necessary to guarantee a satisfactory implementation and operation of the system?

(*Continued*)

Table 4.7 (*Continued*)

Condition	Factor
Personnel requirements	How many people and what levels of skills are necessary to operate the system? Are the skills easily found? What level of training will be necessary?
Operating and maintenance requirements	What are the special operational requirements that need to be provided? Which and how many spare parts will be required, and what is their availability and cost?
Ancillary processes	What support processes are necessary? How do they affect the effluent quality, especially when they become inoperative?
Reliability	What is the reliability of the operation and process in consideration? Is the unit likely to present frequent problems? Can the process resist periodical shock loads? If yes, is the effluent quality affected?
Complexity	What is the complexity of the process in routine and emergency operation? What is the level of training that an operator needs to operate the process?
Compatibility	Can the unit operation or process be used satisfactorily with the existing units? Can plant expansion be easily accomplished?
Area availability	Is there space availability to accommodate, not only the currently required units, but possible future expansions? Is there a buffer zone available to provide landscaping to minimise the aesthetical and environmental impacts in the neighbourhood?

Source: adapted from Metcalf & Eddy (1991)

Table 4.8. Some environmental impacts to be considered in wastewater treatment and sludge management

Item	Comment
Odours	Must be considered in the wastewater treatment and in the processing and disposal of the sludge. Important factor, mainly in urbanised areas.
Vector attraction	Vector (e.g. insects) attraction is connected with odour and can be one of the biggest problems in the sludge processing and disposal.
Noise	Important factor, principally in urbanised areas.
Sludge transportation	Transportation form and route need to be considered.
Sanitary risks	Although difficult to be evaluated objectively, the risk is related to the number of people exposed to the sewage, receiving body and sludge, their qualities and the infection routes.
Air contamination	Air can be contaminated by particulated material from aerosols and sprinkling.
Soil and subsoil contamination	Highly variable in function of the type of wastewater treatment and sewage and sludge disposals, and the processes employed.
Surface or ground water contamination	One of the main aspects of the disposal of wastewater and sludge. Risk depends on the technology employed.
Devaluation of nearby areas	The cost of land and property may be affected by the implementation of a wastewater treatment plant or a disposal site.
Inconvenience to the nearby population	Besides affecting many people, some solutions can generate opposition groups against the implementation of a certain system.

Source: adapted from Fernandes et al (2001)

Table 4.9. Average effluent concentrations and typical removal efficiencies of the main pollutants of interest in domestic sewage

	Average quality of the effluent								Average removal efficiency						
System	BOD$_5$ (mg/L)	COD (mg/L)	SS (mg/L)	Ammonia (mg/L)	Total N (mg/L)	Total P (mg/L)	FC (FC/100mL)	Helminth eggs (eggs/L)	BOD$_5$ (%)	COD (%)	SS (%)	Ammonia (%)	Total N (%)	Total P (%)	FC (log units)
Primary treatment (septic tanks)	200–250	400–450	100–150	>20	>30	>4	10^7–10^8	>1	30–35	25–35	55–65	<30	<30	<35	<1
Conventional primary treatment	200–250	400–450	100–150	>20	>30	>4	10^7–10^8	>1	30–35	25–35	55–65	<30	<30	<35	<1
Advanced primary treatment (chemically enhanced)	60–150	150–250	30–90	>20	>30	<2	10^6–10^7	>1	45–80	55–75	60–90	<30	<30	75–90	≈1
Facultative pond	50–80	120–200	60–90	>15	>20	>4	10^6–10^7	<1	75–85	65–80	70–80	<50	<60	<35	1–2
Anaerobic pond + facultative pond	50–80	120–200	60–90	>15	>20	>4	10^6–10^7	<1	75–85	65–80	70–80	<50	<60	<35	1–2
Facultative aerated lagoon	50–80	120–200	60–90	>20	>30	>4	10^6–10^7	>1	75–85	65–80	70–80	<30	<30	<35	1–2
Complete-mix aerated lagoon + sedimentation pond	50–80	120–200	40–60	>20	>30	>4	10^6–10^7	>1	75–85	65–80	80–87	<30	<30	<35	1–2
Anaerobic pond + facult. pond + maturation pond	40–70	100–180	50–80	10–15	15–20	<4	10^2–10^4	<1	80–85	70–83	73–83	50–65	50-65	>50	3–5
Anaerobic pond + facultative pond + high rate pond	40–70	100–180	50–80	5–10	10–15	3-4	10^4–10^5	>1	80–85	70–83	73–83	65–85	75-90	50–60	3–4
Anaerobic pond – facultative pond + algae removal	30–50	100–150	<30	>15	>20	>4	10^4–10^5	>1	85–90	75–83	>90	<50	<60	<35	3–4
Slow rate treatment	<20	<80	<20	<5	<10	<1	10^2–10^4	<1	90–99	85–95	>93	>80	>75	>85	3–5
Rapid infiltration	<20	<80	<20	<10	<15	<4	10^3–10^4	<1	85–98	80–93	>93	>65	>65	>50	4–5
Overland flow	30–70	100–150	20–60	10–20	>15	>4	10^4–10^6	<1	80–90	75–85	80–93	35–65	<65	<35	2–3
Constructed wetlands	30–70	100–150	20–40	>15	>20	>4	10^4–10^5	<1	80–90	75–85	87–93	<50	<60	<35	3–4
Septic tank + anaerobic filter	40–80	100–200	30–60	>15	>20	>4	10^6–10^7	>1	80–85	70–80	80–90	<45	<60	<35	1–2
Septic tank + infiltration	<20	<80	<20	<10	<15	<4	10^3–10^4	<1	90–98	85–95	>93	>65	>65	>50	4–5

(*Continued*)

Table 4.9. (*Continued*)

System	Average quality of the effluent								Average removal efficiency						
	BOD_5 (mg/L)	COD (mg/L)	SS (mg/L)	Ammonia (mg/L)	Total N (mg/L)	Total P (mg/L)	FC (FC/100mL)	Helminth eggs (eggs/L)	BOD_5 (%)	COD (%)	SS (%)	Ammonia (%)	Total N (%)	Total P (%)	FC (log units)
UASB reactor	70–100	180–270	60–100	>15	>20	>4	10^6–10^7	>1	60–75	55–70	65–80	<50	<60	<35	1–2
UASB + activated sludge	20–50	60–150	20–40	5–15	>20	>4	10^6–10^7	>1	83–93	75–88	87–93	50–85	<60	<35	1–2
UASB + submerged aerated biofilter	20–50	60–150	20–40	5–15	>20	>4	10^6–10^7	>1	83–93	75–88	87–93	50–85	<60	<35	1–2
UASB + anaerobic filter	40–80	100–200	30–60	>15	>20	>4	10^6–10^7	>1	75–87	70–80	80–90	<50	<60	<35	1–2
UASB + high rate trickling filter	20–60	70–180	20–40	>15	>20	>4	10^6–10^7	>1	80–93	73–88	87–93	<50	<60	<35	1–2
UASB + dissolved-air flotation	20–50	60–100	10–30	>20	>30	1–2	10^6–10^7	>1	83–93	83–90	90–97	<30	<30	75–88	1–2
UASB + maturation ponds	40–70	100–180	50–80	10–15	15-20	<4	10^2–10^4	<1	77–87	70–83	73–83	50–65	50–65	>50	3–5
UASB + facultative aerated pond	50–80	120–200	60–90	>20	>30	>4	10^6–10^7	>1	75–85	65–80	70–80	<30	<30	<35	1–2
UASB + compl.mix. aerated lagoon + sedim. pond	50–80	120–200	40–60	>20	>30	>4	10^6–10^7	>1	75–85	65–80	80–87	<30	<30	<35	1–2
UASB + overland flow	30–70	90–180	20–60	10–20	>15	>4	10^4–10^6	<1	77–90	70–85	80–93	35–65	<65	<35	2–3
Conventional activated sludge	15–40	45–120	20–40	<5	>20	>4	10^6–10^7	>1	85–93	80–90	87–93	>80	<60	<35	1–2
Activated sludge – extended aeration	10–35	30–100	20–40	<5	>20	>4	10^6–10^7	>1	90–97	83–93	87–93	>80	<60	<35	1–2
Sequencing batch reactor (extended aeration)	10–35	30–100	20–40	<5	>20	>4	10^6–10^7	>1	90–97	83–93	87–93	>80	<60	<35	1–2
Convent. activated sludge with biological N removal	15–40	45–120	20–40	<5	<10	>4	10^6–10^7	>1	85–93	80–90	87–93	>80	>75	<35	1–2
Convent. activated sludge with biolog. N/P removal	15–40	45–120	20–40	<5	<10	1–2	10^6–10^7	>1	85–93	80–90	87–93	>80	>75	75–88	1–2
Conventional activated sludge + tertiary filtration	10–20	30–60	10–20	<5	>20	3–4	10^2–10^4	<1	93–98	90–95	93–97	>80	<60	50–60	3–5

Low rate trickling filter	15–40	30–120	20–40	5–10	>20	>4	10^6–10^7	>1	85–93	80–90	87–93	65–85	<60	<35	1–2
High rate trickling filter	30–60	80–180	20–40	>15	>20	>4	10^6–10^7	>1	80–90	70–87	87–93	<50	<60	<35	1–2
Submerged aerated biofilter with nitrification	15–35	30–100	20–40	<5	>20	>4	10^6–10^7	>1	88–95	83–90	87–93	>80	<60	<35	1–2
Submerged aerated biofilter with biolog. N removal	15–35	30–100	20–40	<5	<10	>4	10^6–10^7	>1	88–95	83–90	87–93	>80	>75	<35	1–2
Rotating biological contactor	15–35	30–100	20–40	5–10	>20	>4	10^6–10^7	>1	88–95	83–90	87–93	65–85	<60	<35	1–2

- Chemical precipitation of phosphorus with any of the technologies above: $P < 1$ mg/L
- Disinfection: e.g. chlorination, ozonisation, UV radiation; Barrier: e.g. membranes (provided the disinfection/barrier process is compatible with the quality of the effluent from the preceding treatment): $CF < 10^3$ FC/100ml; helminth eggs: variable
- Advanced primary treatment: the removal efficiencies vary depending on the coagulant dosage

Table 4.10. Typical characteristics of the main sewage treatment systems, expressed as per capita values

		Power for aeration		Sludge volume		Costs	
System	Land requirements (m^2/inhab)	Installed power (W/inhab.)	Consumed power (kWh/inhab.year)	Liquid sludge to be treated (L/ inhab.year)	Dewatered sludge to be disposed of (L/ inhab.year)	Construction (US$/inhab.)	Operation and maintenance (US$/inhab.year)
Primary treatment (septic tanks)	0.03–0.05	0	0	110–360	15–35	12–20	0.5–1.0
Conventional primary treatment	0.02–0.04	0	0	330–730	15–40	12–20	0.5–1.0
Advanced primary treatment (chemically enhanced) (a)	0.04–0.06	0	0	730–2500	40–110	15–25	3.0–6.0
Facultative pond	2.0–4.0	0	0	35–90	15–30	15–30	0.8–1.5
Anaerobic pond + facultative pond	1.2–3.0	0	0	55–160	20–60	12–30	0.8–1.5
Facultative aerated lagoon	0.25–0.5	1.2–2.0	11–18	30–220	7–30	20–35	2.0–3.5
Complete-mix aerated lagoon + sedimentation pond	0.2–0.4	1.8–2.5	16–22	55–360	10–35	20–35	2.0–3.5
Anaerobic pond + facultative pond + maturation pond	3.0–5.0	0	0	55–160	20–60	20–40	1.0–2.0
Anaerobic pond + facultative pond + high rate pond	2.0–3.5	<0.3	<2	55–160	20–60	20–35	1.5–2.5
Anaerobic pond – facultative pond + algae removal	1.7–3.2	0	0	60–190	25–70	20–35	1.5–2.5
Slow rate treatment	10–50	0	0	–	–	8–25	0.4–1.2
Rapid infiltration	1.0–6.0	0	0	–	–	12–30	0.5–1.5
Overland flow	2.0–3.5	0	0	–	–	15–30	0.8–1.5
Constructed wetlands	3.0–5.0	0	0	–	–	20–30	1.0–1.5
Septic tank + anaerobic filter	0.2–0.35	0	0	180–1000	25–50	30–50	2.5–4.0
Septic tank + infiltration	1.0–1.5	0	0	110–360	15–35	25–40	1.2–2.0
UASB reactor	0.03–0.10	0	0	70–220	10–35	12–20	1.0–1.5
UASB + activated sludge	0.08–0.2	1.8–3.5	14–20	180–400	15–60	30–45	2.5–5.0
UASB + submerged aerated biofilter	0.05–0.15	1.8–3.5	14–20	180–400	15–55	25–40	2.5–5.0
UASB + anaerobic filter	0.05–0.15	0	0	150–300	10–50	20–30	1.5–2.2
UASB + high rate trickling filter	0.1–0.2	0	0	180–400	15–55	25–35	2.0–3.0

UASB + dissolved-air flotation	0.05–0.15	1.0–1.5	8–12	300–470	25–75	25–35	2.5–3.5
UASB + maturation ponds	1.5–2.5	0	0	150–250	10–35	15–30	1.8–3.0
UASB + facultative aerated pond	0.15–0.3	0.3–0.6	2–5	150–300	15–50	15–35	2.0–3.5
UASB + compl.mix aerated lagoon + sediment. pond	0.1–0.3	0.5–0.9	4–8	150–300	15–50	15–35	2.0–3.5
UASB + overland flow	1.5–3.0	0	0	70–220	10–35	20–35	2.0–3.0
Conventional activated sludge	0.12–0.25	2.5–4.5	18–26	1100–3000	35–90	40–65	4.0–8.0
Activated sludge – extended aeration	0.12–0.25	3.5–5.5	20–35	1200–2000	40–105	35–50	4.0–8.0
Sequencing batch reactor (extended aeration) (b)	0.12–0.25	4.5–6.0	20–35	1200–2000	40–105	35–50	4.0–8.0
Conventional activated sludge with biological N removal	0.12–0.25	2.2–4.2	15–22	1100–3000	35–90	45–70	4.0–9.0
Convention. activated sludge with biological N/P removal	0.12–0.25	2.2–4.2	15–22	1100–3000	35–90	50–75	6.0–10.0
Conventional activated sludge + tertiary filtration	0.15–0.30	2.5–4.5	18–26	1200–3100	40–100	50–75	6.0–10.0
Low rate trickling filter	0.15–0.3	0	0	360–1100	35–80	50–60	4.0–6.0
High rate trickling filter	0.12–0.25	0	0	500–1900	35–80	50–60	4.0–6.0
Submerged aerated biofilter with nitrification	0.1–0.15	2.5–4.5	18–26	1100–3000	35–90	30–50	3.0–6.0
Submerged aerated biofilter with biological N removal	0.1–0.15	2.2–4.2	15–22	11000–3000	35–90	30–50	3.0–6.0
Rotating biological contactor	0.1–0.2	0	0	330–1500	20–75	50–60	4.0–6.0

- Costs based on Brazilian experience (basis: year 2002 – US$1.00 = R$2.50)
- Per capita costs are applicable inside the typical population ranges within which each treatment system is usually applied (usually, for a certain system, the lower the population, the greater the per capita costs)
- Additional disinfection: construction costs – increase US$2.0 to 4.0/inhab.; operational and maintenance costs: increase US$0.2 to 0.6/inhab.year
- In compact aerated systems (e.g.: activated sludge, submerged aerated biofilters) or after treatment with a UASB reactor, aeration control allows a certain economy (not all the installed power is consumed)

(a) Advanced primary treatment: the operational costs depend on the dosage of the chemical product

(b) Sequencing batch reactors (activated sludge) have a greater installed power compared with the consumed power, because all reactors have aerators, but not all aerators are turned on simultaneously

Table 4.11. Capacity of various wastewater treatment technologies in consistently achieving the indicated levels of effluent quality in terms of BOD, COD and SS

	BOD					COD			SS		
System	100 mg/L	80 mg/L	60 mg/L	40 mg/L	20 mg/L	200 mg/L	150 mg/L	100 mg/L	90 mg/L	60 mg/L	30 mg/L
Facultative pond	■	■				▨			▩		
Anaerobic pond + facultative pond	■	■				▨			▩		
Facultative aerated lagoon	■	■	■			▨			▩		
Complete-mix aerated lagoon + sedimentation pond	■	■	■			▨			▩		
Anaerobic pond + facultat. pond + maturation pond	■	■	■			▨			▩		
Anaerobic pond + facultative pond + high rate pond	■	■	■			▨			▩		
Anaerobic pond – facultative pond + algae removal	■	■	■	■		▨	▨		▩	▩	
Slow rate treatment	■	■	■	■	■	▨	▨	▨	▩	▩	▩
Rapid infiltration	■	■	■	■	■	▨	▨	▨	▩	▩	▩
Overland flow	■	■	■			▨			▩	▩	
Constructed wetlands	■	■	■			▨			▩		
Septic tank + anaerobic filter	■	■	■			▨			▩	▩	
Septic tank + infiltration	■	■	■	■	■	▨	▨	▨	▩	▩	▩
UASB reactor	■					▨			▩		
UASB + activated sludge	■	■	■	■		▨	▨	▨	▩	▩	▩
UASB + submerged aerated biofilter	■	■	■	■		▨	▨	▨	▩	▩	▩
UASB + anaerobic filter	■	■	■			▨			▩	▩	
UASB + high rate trickling filter	■	■	■	■		▨	▨	▨	▩	▩	▩
UASB + maturation ponds	■	■	■			▨			▩		
UASB + overland flow	■	■	■			▨			▩	▩	
Conventional activated sludge	■	■	■	■		▨	▨	▨	▩	▩	▩
Extended aeration	■	■	■	■		▨	▨	▨	▩	▩	▩
Sequencing batch reactor	■	■	■	■		▨	▨	▨	▩	▩	▩
Activated sludge with biological N removal	■	■	■	■		▨	▨	▨	▩	▩	▩
Activated sludge with biological N/P removal	■	■	■	■		▨	▨	▨	▩	▩	▩
Activated sludge + tertiary filtration	■	■	■	■	■	▨	▨	▨	▩	▩	▩
Low rate trickling filter	■	■	■	■		▨	▨	▨	▩	▩	▩
High rate trickling filter	■	■	■			▨			▩	▩	▩
Submerged aerated biofilter	■	■	■	■	■	▨	▨	▨	▩	▩	▩
Submerged aerated biofilter with biological N removal	■	■	■	■	■	▨	▨	▨	▩	▩	▩
Rotating biological contactor	■	■	■	■		▨	▨	▨	▩	▩	▩

Table 4.12. Capacity of various wastewater treatment technologies in consistently achieving the indicated levels of effluent quality in terms of Ammonia, total N and total P

	Ammonia – N			Total N			Total P			
System	15 mg/L	10 mg/L	5 mg/L	20 mg/L	15 mg/L	10 mg/L	4.0 mg/L	3.0 mg/L	2.0 mg/L	1.0 mg/L
Facultative pond										
Anaerobic pond + facultativepond										
Facultative aerated lagoon										
Complete-mix aerated lagoon + sedimentation pond										
Anaerobic pond + facultative pond + maturation pond	■			▨						
Anaerobic pond + facultative pond + high rate pond	■	■		▨	▨		▦			
Anaerobic pond – facultative pond + algae removal										
Slow rate treatment	■	■	■	▨	▨	▨	▦	▦	▦	
Rapid infiltration	■	■	■	▨	▨		▦			
Overland flow	■	■		▨						
Constructed wetlands										
Septictank + anaerobic filter										
Septic tank + infiltration	■	■	■	▨	▨	▨				
UASB reactor										
UASB + activated sludge	■	■								
UASB + submerged aerated biofilter	■	■								
UASB + anaerobic filter										
UASB + high rate trickling filter										
UASB + maturation ponds	■			▨						
UASB + overland flow	■	■		▨						
Conventional activated sludge	■	■	■							
Extended aeration	■	■	■							
Sequencing batch reactor	■	■	■							
Activated sludge with biological N removal	■	■	■	▨	▨	▨				
Activated sludge with biological N/P removal	■	■	■	▨	▨	▨	▦	▦	▦	
Activated sludge + tertiary filtration	■	■	■				▦			
Low rate trickling filter	■	■								
High rate trickling filter										
Submerged aerated biofilter	■	■	■							
Submerged aerated biofilter with biological N removal	■	■	■	▨	▨					
Rotating biological contactor	■	■								
Any of the technologies above + chemical P precipitation							▦	▦	▦	▦

Table 4.13. Capacity of various wastewater treatment technologies in consistently achieving the indicated levels of effluent quality in terms of Faecal (thermotolerant) Coliforms and Helminth Eggs

	Faecal (thermotolerant) Coliforms (FC/100mL)				Helminth eggs
System	1 × 10^6	1 × 10^5	1 × 10^4	1 × 10^3	≤ 1 egg/L
Facultative pond	■				▨
Anaerobic pond + facultative pond	■	■			▨
Facultative aerated lagoon	■				
Complete-mix aerated lagoon + sedimentation pond	■				
Anaerobic pond + facultative pond + maturation pond	■	■	■	■	▨
Anaerobic pond + facultative pond + high rate pond	■	■			
Anaerobic pond – facultative pond + algae removal	■	■			
Slow rate treatment	■	■	■	■	▨
Rapid infiltration	■	■	■		▨
Overland flow	■				▨
Constructed wetlands	■	■			▨
Septic tank + anaerobic filter	■				
Septic tank + infiltration	■	■	■	■	▨
UASB reactor					
UASB + activated sludge	■				
UASB + submerged aerated biofilter	■				
UASB + anaerobic filter	■				
UASB + high rate trickling filter	■				
UASB + maturation ponds	■	■	■	■	▨
UASB + overland flow	■	■			▨
Conventional activated sludge					
Extended aeration	■				
Sequencing batch reactor					
Activated sludge with biological N removal					
Activated sludge with biological N/P removal					
Activated sludge + tertiary filtration	■	■	■	■	▨
Low rate trickling filter					
High rate trickling filter					
Submerged aerated biofilter					
Submerged aerated biofilter with biological N removal					
Rotating biological contactor					
Any of the technologies above + disinfection/barrier	■	■	■	■	Variable

Disinfection: e.g. chlorinating, ozonisation, UV radiation; Barrier: e.g. membranes (as long as the disinfection process/barrier is compatible with the quality of the effluent from the preceding treatment)

Table 4.14. Graphical representation of the per capita land requirements for various sewage treatment systems.

System	Land requirement (m^2/inhab.)	Land requirements (m^2/inhab.)							
		0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
Primary treatment (septic tanks)	0.03–0.05	▨							
Conventional primary treatment	0.02–0.04	▨							
Advanced primary treatment (chemically enhanced)	0.04–0.06	▨							
Facultative pond	2.0–4.0					▨	▨	▨	▨
Anaerobic pond + facultative pond	1.2–3.0			▨	▨	▨	▨		
Facultative aerated lagoon	0.25–0.5	▨							
Complete-mix aerated lagoon + sedimentation pond	0.2–0.4	▨							
Anaerobic pond + facultative pond + maturation pond	3.0–5.0							▨	▨
Anaerobic pond + facultative pond + high rate pond	2.0–3.5					▨	▨	▨	
Anaerobic pond + facultative pond + algae removal	1.7–3.2				▨	▨	▨	▨	
Slow rate treatment	10–50								▨ →
Rapid infiltration	1.0–6.0			▨	▨	▨	▨	▨	▨ →
Overland flow	2.0–3.5					▨	▨	▨	
Constructed wetlands	3.0–5.0							▨	▨ →
Septic tank + anaerobic filter	0.2–0.35	▨							
Septic tank + infiltration	1.0–1.5		▨	▨					
UASB reactor	0.03–0.10	▨							
UASB + activated sludge	0.08–0.2	▨							
UASB + submerged aerated biofilter	0.05–0.15	▨							
UASB + anaerobic filter	0.05–0.15	▨							
UASB + high rate trickling filter	0.1–0.2	▨							
UASB + dissolved-air flotation	0.05–0.15	▨							
UASB + maturation ponds	1.5–2.5				▨	▨			
UASB + facultative aerated pond	0.15–0.3	▨							
UASB + compl.mix aerated lagoon + sediment. pond	0.1–0.3	▨							
UASB + overland flow	0.12–0.25				▨	▨	▨		
Conventional activated sludge	0.12–0.25	▨							
Activated sludge – extended aeration	0.12–0.25	▨							
Sequencing batch reactor (extended aeration)	0.12–0.25	▨							
Conventional activated sludge with biological N removal	0.12–0.25	▨							
Convention. activated sludge with biological N/P removal	0.12–0.25	▨							
Conventional activated sludge + tertiary filtration	0.15–0.30	▨							
Low rate trickling filter	0.15–0.3	▨							
High rate trickling filter	0.12–0.25	▨							
Submerged aerated biofilter with nitrification	0.1–0.15	▨							
Submerged aerated biofilter with biological N removal	0.1–0.15	▨							
Rotating biological contactor	0.1–0.2	▨							

Table 4.15. Graphical representation of the per capita installed power for aeration in various sewage treatment systems.

System	Installed power for aeration(W/inhab.)	Installed power for aeration (W/inhab.)							
		0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
Primary treatment (septic tanks)	0								
Conventional primary treatment	0								
Advanced primary treatment (chemically enhanced)	0								
Facultative pond	0								
Anaerobic pond + facultative pond	0								
Facultative aerated lagoon	1.2–2.0			▨	▨				
Complete-mix aerated lagoon + sedimentation pond	1.8–2.5				▨	▨			
Anaerobic pond + facultative pond + maturation pond	0								
Anaerobic pond + facultative pond + high rate pond	<0.3	▨							
Anaerobic pond + facultative pond + algae removal	0								
Slow rate treatment	0								
Rapid infiltration	0								
Overland flow	0								
Constructed wetlands	0								
Septic tank + anaerobic filter	0								
Septic tank + infiltration	0								
UASB reactor	0								
UASB + activated sludge	1.8–3.5				▨	▨	▨	▨	
UASB + submerged aerated biofilter	1.8–3.5				▨	▨	▨	▨	
UASB + anaerobic filter	0								
UASB + high rate trickling filter	0								
UASB + dissolved-air flotation	1.0–1.5			▨					
UASB + maturation ponds	0								
UASB + facultative aerated pond	0.3–0.6	▨	▨						
UASB + compl.mix aerated lagoon + sediment. pond	0.5–0.9		▨						
UASB + overland flow	0								
Conventional activated sludge	2.5–4.5						▨	▨	▨ →
Activated sludge – extended aeration	3.5–5.5								▨ →
Sequencing batch reactor (extended aeration)	4.5–6.0								▨ →
Conventional activated sludge with biological N removal	2.2–4.2					▨	▨	▨	▨ →
Convention. activated sludge with biological N/P removal	2.2–4.2					▨	▨	▨	▨ →
Conventional activated sludge + tertiary filtration	2.5–4.5						▨	▨	▨ →
Low rate trickling filter	0								
High rate trickling filter	0								
Submerged aerated biofilter with nitrification	2.5–4.5								→
Submerged aerated biofilter with biological N removal	2.2–4.2						▨	▨	▨ →
Rotating biological contactor	0					▨	▨	▨	▨

Table 4.16. Graphical representation of the per capita dewatered sludge to be disposed of in various sewage treatment systems.

System	Volume of sludge to be disposed of (L/inhab.year)	Volume of dewatered sludge to be disposed of (L/inhab.year) (≈kg of sludge/inhab.year, on a wet basis)							
		12.5	25.0	37.5	50.0	62.5	75.0	87.5	100.0
Primary treatment (septic tanks)	15–35		▨	▨					
Conventional primary treatment	15–40		▨	▨	▨				
Advanced primary treatment (chemically enhanced)	40–110				▨	▨	▨	▨	▨ →
Facultative pond	15–30		▨	▨					
Anaerobic pond + facultative pond	20–60		▨	▨	▨	▨			
Facultative aerated lagoon	7–30	▨	▨	▨					
Complete-mix aerated lagoon + sedimentation pond	10–35	▨	▨	▨					
Anaerobic pond + facultative pond + maturation pond	20–60		▨	▨	▨	▨			
Anaerobic pond + facultative pond + highrate pond	20–60		▨	▨	▨	▨			
Anaerobic pond + facultative pond + algae removal	25–70			▨	▨	▨	▨		
Slow rate treatment	–								
Rapid infiltration	–								
Overland flow	–								
Constructed wetlands	–								
Septic tank + anaerobic filter	25–50			▨	▨				
Septic tank + infiltration	15–35		▨	▨					
UASB reactor	10–35	▨	▨	▨					
UASB + activated sludge	15–60		▨	▨	▨	▨			
UASB + submerged aerated biofilter	15–55		▨	▨	▨	▨			
UASB + anaerobic filter	10–50	▨	▨	▨	▨	▨			
UASB + high rate trickling filter	15–55		▨	▨	▨	▨			
UASB + dissolved-air flotationt	25–75			▨	▨	▨	▨		
UASB + maturation ponds	10–35	▨	▨	▨					
UASB + facultative aerated pond	15–50		▨	▨	▨	▨			
UASB + compl.mix aerated lagoon + sediment.pond	15–50		▨	▨	▨	▨			
UASB + overland flow	10–35	▨	▨	▨					
Conventional activated sludge	35–90			▨	▨	▨	▨	▨	▨
Activated sludge – extended aeration	40–105				▨	▨	▨	▨	▨ →
Sequencing batch reactor (extended aeration)	40–105				▨	▨	▨	▨	▨ →
Conventional activated sludge with biological N removal	35–90			▨	▨	▨	▨	▨	▨
Convention. activated sludge with biological N/P removal	35–90			▨	▨	▨	▨	▨	▨
Conventional activated sludge + tertiary filtration	40–100				▨	▨	▨	▨	▨
Low rate trickling filter	35–80			▨	▨	▨	▨	▨	
High rate trickling filter	35–80			▨	▨	▨	▨	▨	
Submerged aerated biofilter with nitrification	35–90			▨	▨	▨	▨	▨	▨
Submerged aerated biofilter with biological N removal	35–90			▨	▨	▨	▨	▨	▨
Rotating biological contactor	20–75		▨	▨	▨	▨	▨		

Table 4.17. Graphical representation of the per capita construction costs in various sewage treatment systems.

System	Construction costs (US$/inhab.)
Primary treatment (septic tanks)	12–20
Conventional primary treatment	12–20
Advanced primary treatment (chemically enhanced)	15–25
Facultative pond	15–30
Anaerobic pond + facultative pond	20–30
Facultative aerated lagoon	20–35
Complete-mix aerated lagoon + sedimentation pond	20–35
Anaerobic pond + facultative pond + maturation pond	20–40
Anaerobic pond + facultative pond + high rate pond	20–35
Anaerobic pond + facultative pond + algae removal	20–35
Slow rate treatment	8–25
Rapid infiltration	12–30
Overland flow	15–30
Constructed wetlands	20–30
Septic tank + anaerobic filter	30–50
Septic tank + infiltration	25–40
UASB reactor	12–20
UASB + activated sludge	30–45
UASB + submerged aerated biofilter	25–40
UASB + anaerobic filter	20–30
UASB + high rate trickling filter	25–35
UASB + dissolved-air flotation	25–35
UASB + maturation ponds	15–30
UASB + facultative aerated pond	15–35
UASB + compl.mix aerated lagoon + sediment. pond	15–35
UASB + overland flow	20–35
Conventional activated sludge	40–65
Activated sludge – extended aeration	35–50
Sequencing batch reactor (extended aeration)	35–50
Conventional activated sludge with biological N removal	45–70
Convention. activated sludge with biological N/P removal	50–75
Conventional activated sludge + tertiary filtration	50–75
Low rate trickling filter	50–60
High rate trickling filter	50–60
Submerged aerated biofilter with nitrification	30–50
Submerged aerated biofilter with biological N removal	30–50
Rotating biological contactor	50–60

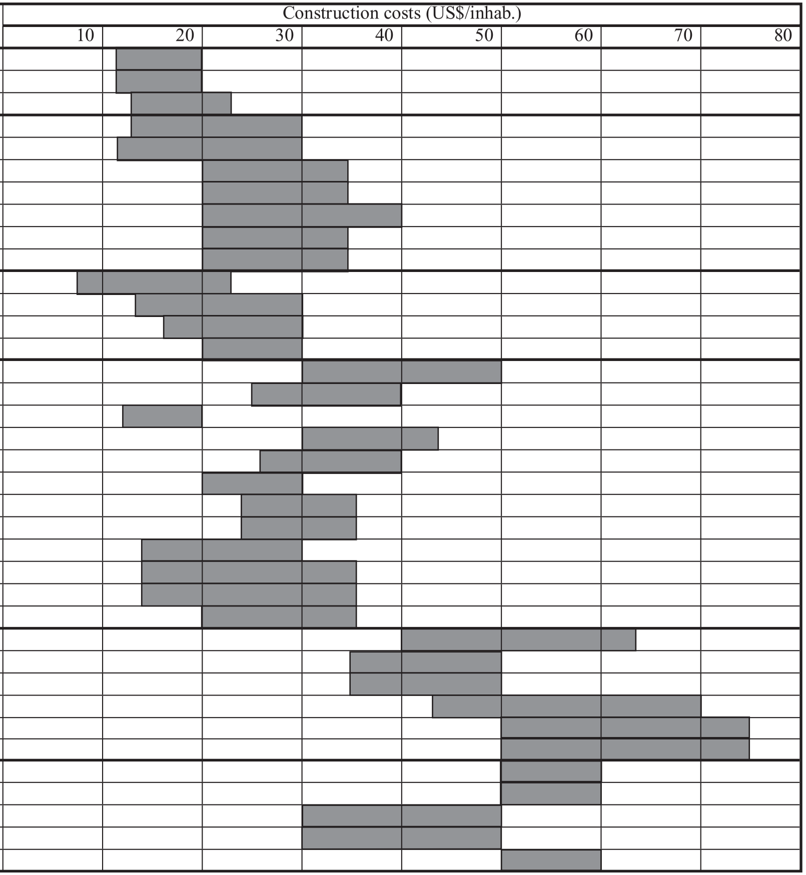


Table 4.18. Graphical representation of the per capita annual operation and maintenance costs in various sewage treatment systems.

System	Operation and maintenance costs (US$/inhab.year)
Primary treatment (septic tanks)	0.5–1.0
Conventional primary treatment	0.5–1.0
Advanced primary treatment (chemically enhanced)	3.0–6.0
Facultative pond	0.8–1.5
Anaerobic pond + facultative pond	0.8–1.5
Facultative aerated lagoon	2.0–3.5
Complete-mix aerated lagoon + sedimentation pond	2.0–3.5
Anaerobic pond + facultative pond + maturation pond	1.0–2.0
Anaerobic pond + facultative pond + high rate pond	1.5–2.5
Anaerobic pond + facultative pond + algae removal	1.5–2.5
Slow rate treatment	0.4–1.2
Rapid infiltration	0.5–1.5
Overland flow	0.8–1.5
Constructed wetlands	1.0–1.5
Septic tank + anaerobic filter	2.5–4.0
Septic tank + infiltration	1.2–2.0
UASB reactor	1.0–1.5
UASB + activated sludge	2.5–5.0
UASB + submerged aerated biofilter	2.5–5.0
UASB + anaerobic filter	1.5–2.2
UASB + high rate trickling filter	2.0–3.0
UASB + dissolved-air flotation	2.5–3.5
UASB + maturation ponds	1.8–3.0
UASB + facultative aerated pond	2.0–3.5
UASB + compl.mix aerated lagoon + sediment. pond	2.0–3.5
UASB + overland flow	2.0–3.0
Conventional activated sludge	4.0–8.0
Activated sludge – extended aeration	4.0–8.0
Sequencing batch reactor (extended aeration)	4.0–8.0
Conventional activated sludge with biological N removal	4.0–9.0
Convention. activated sludge with biological N/P removal	6.0–10.0
Conventional activated sludge + tertiary filtration	6.0–10.0
Low rate trickling filter	4.0–6.0
High rate trickling filter	4.0–6.0
Submerged aerated biofilter with nitrification	3.0–6.0
Submerged aerated biofilter with biological N removal	3.0–6.0
Rotating biological contactor	4.0–6.0

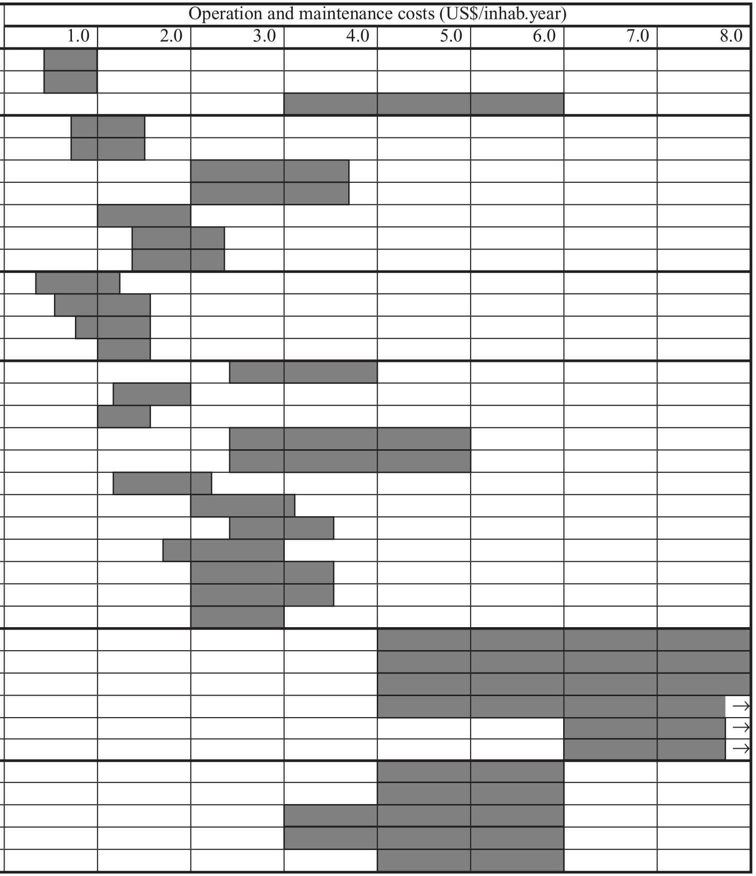


Table 4.19. Relative evaluation of the main domestic sewage treatment systems (liquid phase)

	Removal efficiency			Economy					Resistance capacity to influent variations and shock loads			Reliability	Simplicity in O&M.	Independence of other charact.for good perform.		Lower possibility of environmental problems			
				Requirements		Costs		Generation											
	BOD	Nutrients	Coliforms	Land	Energy	Constr.	O & M	Sludge	Flow	Quality	Toxic			Climate	Soil	Bad odours	Noise	Aerosols	Insects and worms
Preliminary treatment	0	0	0	+++++	+++++	+++++	++++	+++++	+++++	+++++	+++++	+++++	+++	+++++	+++++	+	++++	+++++	+++
Primary treatment	+	+	+	+++++	++++	++++	+++	+++	++++	+++++	++++	++++	+++	++++	+++++	++	++++	+++++	+++
Advanced primary treatment	++	+/++++	++	+++++	++++	+++	++	+	++++	+++++	++++	++++	+++	+++++	+++++	+++	++++	+++++	+++
Facultative pond	+++	++	++/++++	+	+++++	+++	+++++	+++++	++++	++++	+++	++++	+++++	++	+++	+++	+++++	+++++	++
Anaerobic pond – facultative pond	+++	++	++/++++	++	+++++	++++	+++++	+++++	++++	++++	+++	++++	+++++	++	+++	+	+++++	+++++	++
Facultative aerated lagoon	+++	++	++/++++	++	+++	+++	++++	+++++	++++	++++	+++	++++	++++	+++	+++	++++	+	+	+++
Compl. mix aerated – sedim. pond	+++	++	++/++++	+++	+++	+++	+++	+++	+++	++++	+++	+++	+++	+++	++++	+++	+	+	++
Pond – maturation pond	+++	+++	+++++	+	+++++	+++	+++++	+++++	++++	++++	+++	++++	+++++	++	+++	+++	+++++	+++++	++
Pond –high rate pond	+++	++++	++++	++	++++	+++	++++	+++++	++++	++++	+++	++++	+++	+++	+++	+++	++	++	++
Pond – algae removal	++++	++	++/++++	++	+++++	+++	++++	+++	++++	++++	+++	++++	+++	+++	+++	+++	+++++	+++++	++
Slow rate treatment	+++++	++++	++++	+	+++++	+++	+++++	+++++	++++	++++	++++	++++	++++	++	+	++	+++++	+/+++++	++
Rapid infiltration	+++++	++++	++++	+	+++++	++++	+++++	+++++	++++	++++	++++	++++	++++	++	+	++	+++++	+++++	++
Overland flow	++++	+++	++/+++	+	+++++	+++	++++	+++++	++++	++++	+++	++++	+++++	+++	++	++	+++++	+/+++++	++
Constructed wetlands	++++	++	+++	+	+++++	+++	++++	+++++	++++	++++	+++	++++	+++++	++	++	++	+++++	+++++	++
Septic tank – anaerobic filter	+++	+	++	+++++	+++++	+++	+++	++++	+++	+++	++	+++	++++	++	+++++	++	++++	+++++	++++
UASB reactor	+++	+	++	+++++	+++++	++++	++++	++++	++	++	++	+++	++++	++	+++++	++	++++	+++++	++++
UASB reactor – post-treatment	(a)	(a)	(a)	(a)	(a)	(a)	(a)	(a)	(b)	(b)	(b)	(a)	(a)	(a)	(a)	(b)	(a)	(a)	(a)
Conventional activated sludge	++++	++/++++	++	++++	++	+	++	+	+++	+++	++	++++	+	+++	+++++	++++	+	+/+++++	++++
Activated sludge (extended aeration)	+++++	++/++++	++	++++	+	++	+	++	++++	++++	+++	++++	++	++++	+++++	+++++	+	+/+++++	++++
Sequencing batch reactor	++++	++/++++	++	++++	+/++	+	+	+/++	++++	++++	+++	++++	+++	++++	+++++	+++	+	+/+++++	++++
Trickling filter (low rate)	++++	++/++++	++	+++	++++	+	+++	++	+++	++	++	++++	+++	++	+++++	++++	++++	++++	++
Trickling filter (high rate)	++++	++/+++	++	++++	+++	++	+++	+	++++	+++	+++	++++	+++	++	+++++	++++	++++	++++	+++
Submerged aerated biofilter	+++++	++/+++	++	+++++	++		+++	+	+++	+++	++	++++	++	++++	+++++	+++++	++	+++++	++++
Rotating biological contactor	++++	++/+++	++	++++	+++	+	+++	+	+++	+++	++	+++	+++	++	+++++	++++	++++	+++++	+++

Notes: the grading is only relative in each column and is not generalised for all the items. The grading can vary widely with the local conditions

+++++ : most favourable   + : least favourable   ++++, +++, ++: intermediate grades, in decreasing order   0 : zero effect

+ / +++++: variable with the type of process, equipment, variant or design

UASB reactor + post-treatment: (a) post-treatment characteristics prevail; (b) UASB reactor characteristics prevail

Table 4.20. Minimum equipment necessary for the main wastewater treatment processes

Treatment process	Basic equipment required
Preliminary treatment	Screens; grit chamber; flowmeter
Primary treatment	Sludge scraper (larger systems); mixers in the digesters; gas equipment
Facultative pond	–
Anaerobic pond – facult. pond	Effluent recycle pump (optional)
Facultative aerated lagoon	Aerators
Compl. mix. aerated – sedim. pond	Aerators
High rate pond	Rotors for movement of liquid
Maturation pond	–
Slow rate treatment	Sprinklers (optional)
Rapid infiltration	–
Subsurface infiltration	–
Overland flow	Sprinklers (optional)
Constructed wetlands	–
Septic tank – anaerobic filter	–
UASB reactor	–
UASB reactor + post-treatment	The equipment depends on the post-treatment process used. However, there is no need for equipment relating to primary sedimentation tanks, thickeners and sludge digesters.
Conventional activated sludge	Aerators; sludge recycle pumps; sludge scrapers in sedimentation tanks; sludge scrapers in thickeners; mixers in digesters; gas equipment; pumps for the return of supernatants and drained liquids from sludge treatment.
Activated sludge (extended aeration)	Aerators; sludge recycle pumps; sludge scrapers in sedimentation tanks; thickening equipment; pumps for the return of supernatants and drained liquids from sludge treatment.
Sequencing batch reactors	Aerators; thickening equipment; pumps for the return of supernatants and drained liquids from sludge treatment.
Trickling filter (low rate)	Rotating distributor; sludge scrapers in sedimentation tanks; sludge scrapers in thickeners; pumps for the return of supernatants and drained liquids from sludge treatment.
Trickling filter (high rate)	Rotating distributor; effluent recycle pumps; sludge scrapers in sedimentation tanks; sludge scrapers in thickeners; mixers in digesters; gas equipment; pumps for the return of supernatants and drained liquids from sludge treatment.
Submerged aerated biofilters	Aeration system; filter washing system; sludge scrapers in sedimentation tanks; thickening equipment; mixers in digesters; gas equipment; pumps for the return of supernatants and drained liquids from sludge treatment.
Rotating biological contactors	Motor for the rotation of the discs; thickening equipment; pumps for the return of supernatants and drained liquids from sludge treatment.

Table 4.21. Comparative analysis of the main wastewater treatment systems. Balance of the advantages and disadvantages

STABILISATION PONDS SYSTEMS		
System	Advantages	Disadvantages
Facultative pond	• Satisfactory BOD removal efficiency • Reasonable pathogen removal efficiency • Simple construction, operation and maintenance • Reduced construction and operating costs • Absence of mechanical equipment • Practically no energy requirements • Satisfactory resistance to load variations • Sludge removal only necessary after periods greater than 20 years	• High land requirements • Difficulty in satisfying restrictive discharge standards • Operational simplicity can bring a disregard to maintenance (e.g. vegetation growth) • Possible need for removing algae from effluent to comply with stringent discharge standards • Variable performance with climatic conditions (temperature and sunlight) • Possible insect growth
Anaerobic pond – facultative pond system	• The same as facultative ponds • Lower land requirements than single facultative ponds	• The same as facultative ponds • Possibility of bad odours in the anaerobic pond • Occasional need for effluent recycling to control bad odours • Need for a safe distance from surrounding neighbourhoods • Need for periodic (few years interval) removal of sludge from anaerobic pond
Facultative aerated lagoon	• Relatively simple construction, operation and maintenance • Lower land requirements than the facultative and anaerobic-facultative pond systems • Greater independence from climatic conditions than the facultative and anaerobic-facultative pond systems • Satisfactory resistance to load variations • Reduced possibilities of bad odours	• Introduction of equipment • Slight increase in the sophistication level • Land requirements still high • Relatively high energy requirements • Low coliform removal efficiency • Need for periodic (some years interval) removal of sludge from aerated pond
Completely-mixing aerated lagoon – sedimentation pond system	• Same as facultative aerated lagoons • Lowest land requirements for all the ponds systems	• Same as facultative aerated lagoons (exception: land requirements) • Rapid filling of the sedimentation pond with sludge (2 to 5 years) • Need for continuous or periodic (few years interval) removal of sludge from sedimentation pond

Table 4.21 (*Continued*)

System	Advantages	Disadvantages
Ponds – maturation pond system	• Same as the preceding ponds • High pathogen removal efficiency • Reasonable nutrient removal efficiency	• Same as the preceding ponds • Very high land requirements
Ponds – high rate pond	• Same as the preceding ponds • Good pathogen removal efficiency • High nutrient removal efficiency	• Same as the preceding ponds
	LAND DISPOSAL SYSTEMS	
Slow rate treatment	• High removal efficiency of BOD and coliforms • Satisfactory removal efficiency of N and P • Combined treatment and final disposal methods • Practically no energy requirements • Simple construction, operation and maintenance • Reduced construction and operation costs • Good resistance to load variations • No sludge to be treated • Provides soil fertilisation and conditioning • Financial return from irrigation in agricultural areas • Recharge of groundwater	• Very high land requirements • Possibility of bad odours • Possibility of vector attraction • Relatively dependent on the climate and the nutrient requirements of the plants • Dependent on the soil characteristics • Contamination risk to the plants to be consumed if applied indiscriminately • Possibility of the contamination of the farm workers (e.g. in application by sprinklers) • Possibility of chemical effects in the soil, plants or groundwater (in the case of industrial wastewater) • Difficult inspection and control of the irrigated vegetables • The application must be suspended or reduced in rainy periods
Rapid infiltration	• The same as slow rate treatment (although the removal efficiency of pollutants is lower) • Much lower land requirements than slow rate treatment • Reduced dependence on the slope of the ground • Application during all the year	• Same as slow rate treatment (but with lower land requirements and the possibility of application during all the year) • Potential contamination of groundwater with nitrates
Subsurface infiltration	• Same as rapid infiltration • Possible economy in the implementation of interceptors • Absence of bad odours • The above ground can be used as green area or parks • Independent of climatic conditions • Absence of problems related to the contamination of plants and workers	• Same as rapid infiltration • Requires spare units to allow switching between units (operation and rest) • The larger systems require very permeable soil to reduce land requirements

(*Continued*)

Table 4.21 (*Continued*)

System	Advantages	Disadvantages
Overland flow	• Same as rapid infiltration (but with the generation of a final effluent and with a greater dependence on the ground slope) • Lowest dependence on the soil characteristics among the land disposal systems	• Same as rapid infiltration • Greater dependence on the ground slope • Generation of a final effluent
Constructed wetlands	• High removal efficiency of BOD and coliforms • Practically no energy requirements • Simple construction, operation and maintenance • Reduced construction and operational costs • Good resistance to load variations • No sludge to be treated • Possibility of using the produced plant biomass	• High land requirements • Wastewater requires previous treatment (primary or simplified secondary) • Need for a substrate, such as gravel or sand • Susceptible to clogging • Need of macrophytes handling • Possibility of mosquitoes in surface flow systems
	ANAEROBIC REACTORS	
UASB reactor	• Reasonable BOD removal efficiency • Low land requirements • Low construction and operational costs • Tolerance to influents highly concentrated in organic matter • Practically no energy consumption • Possibility of energy use of the biogas • Support medium not required • Simple construction, operation and maintenance • Very low sludge production • Sludge stabilisation in the reactor itself • Sludge with good dewaterability • Sludge requires only dewatering and final disposal • Rapid start up after periods of no use (biomass preservation for various months)	• Difficulty in complying with restrictive discharge standards • Low coliform removal efficiency • Practically no N and P removal • Possibility of the generation of an effluent with an unpleasant aspect • Possibility of the generation of bad odours, although controllable • Initial start up is generally slow (but can be accelerated with the use of seeding) • Relatively sensitive to load variations and toxic compounds • Usually needs post-treatment
Septic tank – anaerobic filter	• Same as UASB reactors (exception: support medium required) • Good adaptation to different wastewater types and concentrations • Good resistance to load variations	• Difficulty in complying with restrictive discharge standards • Low coliform removal efficiency • Practically no N and P removal • Possibility of the generation of an effluent with an unpleasant aspect • Possibility of the generation of bad odours, although controllable

Table 4.21 (*Continued*)

System	Advantages	Disadvantages
		• Risks of clogging • Restricted to the treatment of influents without high solids concentrations
UASB reactor – post-treatment system	• Maintenance of the inherent advantages of the UASB reactor • Maintenance of the inherent advantages of the post-treatment system • Reduction in the volume in the biological reactors in the post-treatment system (and frequently in the overall volume of the whole system) • Reduction in the energy consumption for aerated post-treatment systems • Reduction in the sludge production in the post-treatment system	• Maintenance of the inherent disadvantages of the UASB reactor (with the exception of the effluent quality, that assumes the characteristics of the post-treatment system) • Maintenance of the inherent disadvantages of the post-treatment system • Greater difficulty in the biological removal of nutrients in the post-treatment system
	ACTIVATED SLUDGE SYSTEMS	
Conventional activated sludge	• High BOD removal efficiency • Nitrification usually obtained • Biological removal of N and P is possible • Low land requirements • Reliable process, as long as it is supervised • Reduced possibilities of bad odours, insects and worms • Operational flexibility	• Low coliform removal efficiency • High construction and operational costs • High energy consumption • Sophisticated operation required • High mechanisation level • Relatively sensitive to toxic discharges • Requires complete treatment and final disposal of the sludge • Possible environmental problems with noise and aerosols
Extended aeration	• Same as conventional activated sludge • Variant with the highest BOD removal efficiency • Consistent nitrification • Conceptually simpler than conventional activated sludge (simpler operation) • Lower sludge production than conventional activated sludge • Sludge digestion in the reactor itself • High resistance to load variations and toxic loads • Satisfactory independence from climatic conditions	• Low coliform removal efficiency • High construction and operational costs • System with the highest energy consumption • High mechanisation level (although less than conventional activated sludge) • Thickening / dewatering and final disposal of the sludge required
Sequencing batch reactors	• High BOD removal efficiency • Satisfactory removal of N and possibly P • Low land requirement	• Low coliform removal efficiency • High construction and operational costs

(*Continued*)

Table 4.21 (*Continued*)

System	Advantages	Disadvantages
	• Conceptually simpler than the other activated sludge systems • Less equipment than the other activated sludge systems • Operational flexibility (through cycle variation) • Secondary sedimentation tanks and sludge recycle pumps are not necessary (operation as extended aeration: primary clarifiers and sludge digesters also not necessary)	• Greater installed power than the other activated sludge systems • Treatment and disposal of the sludge is required (variable with the conventional or extended aeration mode, although the latter is more frequent) • Usually economically more competitive for small to medium-size populations
Activated sludge with biological nutrient removal	• Same as conventional activated sludge • High nutrient removal efficiency	• Same as conventional activated sludge • Requirement of internal recycles • Increase in the operational complexity
	AEROBIC BIOFILM REACTORS	
System	Advantages	Disadvantages
Low rate trickling filter	• High BOD removal efficiency • Frequent nitrification • Relatively low land requirements • Conceptually simpler than activated sludge • Relatively low mechanisation level • Simple mechanical equipment • Sludge digestion in the filter itself	• Low coliform removal efficiency • Lower operational flexibility than activated sludge • High construction costs • Land requirements higher than high rate trickling filters • Relative dependence from the air temperature • Relatively sensitive to toxic discharges • Thickening / dewatering and final disposal of the sludge required • Possible problems with flies • High head loss
High rate trickling filter	• Good BOD removal efficiency (although slightly less than the low rate filters) • Low land requirements • Conceptually simpler than activated sludge • Greater operational flexibility than low rate filters • Better resistance to load variations than low rate filters • Reduced possibilities of bad odours	• Low coliform removal efficiency • Operation slightly more sophisticated than low rate filters • High construction costs • Relative dependence from the air temperature • Complete sludge treatment and final disposal required • High head loss
Submerged aerated biofilters	• High BOD removal efficiency • Optional nitrification (frequent, when desired) • Very low land requirements • Reduced possibilities of bad odours • Reduced head loss	• Low coliform removal efficiency • Relatively high construction and operational costs • High energy consumption • Requirement of a slightly more careful operation compared to trickling filters (aeration and washing of the filters) • Complete sludge treatment and final disposal required

Table 4.21 (*Continued*)

System	Advantages	Disadvantages
Rotating biological contactors	• High BOD removal efficiency • Frequent nitrification • Very low land requirements • Conceptually simpler than activated sludge • Simple mechanical equipment • Reduced possibilities of bad odours • Reduced head loss	• Low coliform removal efficiency • High construction and operational costs • Mainly indicated for small populations (avoid excessive number of discs) • Usually the discs need to be covered (protection against rain, wind and vandalism) • Relative dependence from the air temperature • Complete sludge treatment (possibly without digestion if the discs are installed on top of septic tanks) and final disposal required

5

Overview of sludge treatment and disposal

5.1 INTRODUCTION

The main solid by-products produced in wastewater treatment are:

- *screened material*
- *grit*
- *scum*
- *primary sludge*
- *secondary sludge*
- *chemical sludge (if a physical–chemical stage is included)*

The treatment of the solid by-products generated in the various units is an essential stage in wastewater treatment. Even though the sludge, in most of its handling stages, is constituted by more than 95% water, it is only by convention that it is called a **solid phase**, with the aim at distinguishing it from the wastewater, or the liquid flow being treated (**liquid phase**). Owing to the greater volume and mass generated, compared with the other solid by-products, the present book covers the problems of sludge in greater depth.

The following aspects need to be taken into consideration and quantified when planning the sludge management:

- **production** of the sludge in the *liquid phase*

Biological Wastewater Treatment in Warm Climate Regions by Marcos von Sperling and Carlos Augusto de Lemos Chernicharo. ISBN: 1 84339 002 7. Published by IWA Publishing, London, UK.

- **wastage** of the sludge from the *liquid phase* (removal to the sludge processing line)
- **wastage** of the sludge from the *solid phase* (removal from the wastewater treatment plant to the sludge disposal or reuse site)

The sludge *production* is a function of the wastewater treatment system used for the liquid phase. In principle, all the biological treatment processes generate sludge. The processes that receive raw wastewater in primary settling tanks generate the **primary sludge**, which is composed of the settleable solids of the raw wastewater. In the biological treatment stage, there is the so-called **biological sludge** or **secondary sludge**. This sludge is the biomass that grows at the expense of the food supplied by the incoming sewage. If the biomass is not removed, it tends to accumulate in the system and eventually leaves with the final effluent, deteriorating its quality in terms of suspended solids and organic matter. Depending on the treatment system, the primary sludge can be sent for treatment together with the secondary sludge. In this case, the resultant sludge of the mixture is called **mixed sludge**. In treatment systems that incorporate a physical–chemical stage for improving the performance of primary or secondary settling tanks, a **chemical sludge** is produced.

Since sludge is produced, its *wastage* from the liquid phase is necessary. However, not all the wastewater treatment systems need the continuous removal of this biomass. Some treatment systems can store the sludge for all the operating horizon of the works (e.g. facultative ponds), others require only an occasional withdrawal (e.g. anaerobic reactors) and others need the continuous or very frequent removal (e.g. activated sludge). The *biological sludge* withdrawn is also called **excess sludge**, **surplus sludge**, **waste sludge** or **secondary sludge**.

Finally, the sludge is treated and processed in the solid phase stage, from where it is removed or wasted, going to *final disposal* or *reuse* routes.

Table 5.1 presents a summary description of the main types of solid by-products and their origin in wastewater treatment. All of the treatment processes start with preliminary treatment, where there is, necessarily, the generation of *screened material* and *grit*. The *scum* is variable from process to process and can or cannot occur systematically. The *primary sludge* is only generated in plants that have a primary treatment stage (primary sedimentation tank). The *secondary sludge* is generated in all biological treatment processes. The sludge type varies, and the table makes a distinction between aerobic sludge (non-stabilised), aerobic sludge (stabilised) and anaerobic sludge (stabilised). The biological treatment processes are described in Chapter 4. The *chemical sludge* is only produced in plants that explicitly incorporate a physical–chemical stage in the treatment of the liquid phase.

Wastewater treatment processes based on land application also generate biomass, which is, in this case, mainly composed by the plant biomass related to the irrigated culture. This plant biomass can be used or disposed of after cutting or harvesting and possible processing. The analysis of the management of this biomass is outside the scope of the present text.

Table 5.1. Origin and description of the main solid by-products generated in wastewater treatment

Solid by-product	Origin	Description
Coarse solids	• Screen	The solids removed in the screens include all of the organic and inorganic solids with dimensions greater than the free space between the bars. The organic material varies in function of the characteristics of the sewerage system and the season of the year. The removal can be manual or mechanical.
Grit	• Grit chamber	The grit usually consists of heavier inorganic solids that settle with relatively high velocities. The grit is removed in units called grit chambers that are settling tanks with a low hydraulic detention time, which is only sufficient for the grit to settle. However, depending on the operating conditions, organic matter, mainly fats and grease, can also be removed.
Scum	• Grit chamber • Primary settling tank • Secondary settling tank • Stabilisation pond • Anaerobic tank	The scum removed from primary settling tanks consists of floating material that has been scraped from the surface; this includes grease, vegetable and mineral oils, animal fats, soaps, food wastes, vegetable and fruits peelings, hair, paper, cotton, cigarette tips and similar materials. The specific gravity of scum is less than 1.0 (generally around 0.95). The grit chambers do not usually have scum removal equipment. In secondary treatment, biological reactors also produce scum, which includes scum-forming microorganisms that develop under specific environmental conditions. This scum is usually removed in the secondary settling tanks by scraping the surface. Stabilisation ponds and anaerobic reactors can also present scum.
Primary sludge	• Septic tank • Primary settling tank	The solids removed by settling from primary sedimentation tanks constitute the primary sludge. Primary sludge can have a strong odour, principally if retained in the primary settling tank for a long time in high temperature conditions. The primary sludge removed from septic tanks stays a time long enough for its anaerobic digestion, under controlled conditions (closed tanks).

Table 5.1 (*Continued*)

Solid by-product	Origin	Description
Aerobic biological sludge (non-stabilised)	• Conventional activated sludge • Aerobic biofilm reactors – high rate (high rate trickling filter, submerged aerated biofilter, rotating biological contactor)	The excess biological sludge (secondary sludge) consists of a biomass of aerobic microorganisms generated at the expense of the removal of the organic matter (substrate) from the wastewater. This biomass is in constant growth, resulting from the continuous input of organic matter into the biological reactors. To maintain the system in equilibrium, approximately the same mass of biological solids generated must be removed from the system. If the residence time of the solids in the system is low and there is a satisfactory level of substrate available, the biological solids will contain greater levels of organic matter in their cellular composition. These solids are not stabilised (digested), requiring a subsequent separate digestion stage. If no digestion is included, release of bad odours by the sludge during its treatment and final disposal is likely to occur, because of the anaerobic decomposition of the organic matter under uncontrolled conditions.
Aerobic biological sludge (stabilised)	• Activated sludge – extended aeration • Aerobic biofilm reactors – low rate (low rate trickling filter, rotating biological contactor, submerged aerated biofilter)	This biological sludge is also predominantly composed by aerobic microorganisms that grow and multiply themselves at the expense of the organic matter in the raw wastewater. However, in low loading rate systems, the availability of substrate is lower and the biomass is retained longer in the system, thus prevailing endogenous respiration conditions. Under these conditions, the biomass uses its own reserves of organic matter in the composition of the cellular protoplasm, which leads to a sludge with a lower level of organic matter (digested sludge) and higher level of inorganic solids. This sludge does not require a subsequent separate digestion stage.
Anaerobic biological sludge (stabilised)	• Stabilisation ponds (facultative ponds, anaerobic-facultative ponds, facultative aerated lagoons, complete-mix aerated lagoons – sedimentation lagoons)	Anaerobic reactors and the sludge at the bottom of stabilisation ponds are in predominantly anaerobic conditions. The anaerobic biomass also grows and multiplies itself at the expense of the organic matter. In these treatment processes, the biomass is usually retained for a long time, in which anaerobic digestion of their own cellular material occurs. In stabilisation ponds, the sludge

(*Continued*)

Table 5.1 (*Continued*)

Solid by-product	Origin	Description
	• Anaerobic reactors (UASB reactors, anaerobic filters)	is also composed by settled solids from the raw sewage, together with dead algae. This sludge does not require a subsequent digestion stage.
Chemical sludge	• Primary settling tanks with chemical precipitation • Activated sludge with chemical phosphorus precipitation	This sludge is usually a result of chemical precipitation with metallic salts or lime. The concern with odours is less than with the primary sludge, although it still can occur (only in the case of the use of lime as a coagulant). The decomposition rate of the chemical sludge in the tanks is less than the primary sludge.

Source: adapted from Metcalf & Eddy (1991) and von Sperling and Gonçalves (2001)

The term '**sludge**' has been used to designate the solid by-products from wastewater treatment. In the biological treatment processes, part of the organic matter is absorbed and converted into microbial biomass, generically called biological or secondary sludge. This is principally composed of biological solids, and for this reason it is also called a **biosolid**. To adopt this term, it is still necessary that its chemical and biological characteristics are compatible with a productive use, such as for example in agriculture. The term 'biosolid' is a way of emphasising its beneficial aspects, giving more value to productive uses, in comparison with the mere final non-productive disposal by means of landfills or incineration.

5.2 RELATIONSHIPS IN SLUDGE: SOLIDS LEVELS, CONCENTRATION AND FLOW

To express the characteristics of the sludge, as well as to calculate the sludge production in terms of mass and volume, the understanding of certain fundamental relations that are covered below are essential.

a) Relation between solids levels and water content

The relation between the level of dry solids and the water content in the sludge is given by:

$$\boxed{\text{Water content (\%)} = 100 - \text{Dry solids level (\%)}} \qquad (5.1)$$

A sludge with a level of dry solids of 2% has a water content of 98%. Therefore, in every 100 kg of sludge, 98 kg are water and 2 kg are solids.

The water content influences the mechanical properties of the sludge and these influence the handling processes and the final disposal of the sludge. The relation between the water content and the mechanical properties in most forms of sludges is (van Haandel and Lettinga, 1994):

Water content	Dry-solids content	Mechanical properties of the sludge
100% to 75%	0% to 25%	fluid sludge
75% to 65%	25% to 35%	semi-solid cake
65% to 40%	35% to 60%	hard solid
40% to 15%	60% to 85%	sludge in granules
15% to 0%	85% to 100%	sludge disintegrating into a fine powder

In the present context, dry solids ***(d.s.)*** *are equivalent to total solids (****TS****), which, in the case of sludges, are very similar to total suspended solids (****TSS*** *or simply* ***SS****). These variables may be used interchangeably in this book, when representing solids concentration in the sludge.*

The water content has a large influence on the volume to be handled, as detailed in item d below.

The water in the sludge can be divided into four distinct classes, with different degrees in the easiness of separation (van Haandel and Lettinga, 1994):

- *Free water*. Can be removed by gravity (thickening, flotation)
- *Adsorbed water*. Can be removed by mechanical forces or by the use of a flocculating agent
- *Capillary water*. Maintains itself adsorbed in the solid phase by capillary forces, and is distinguished from the adsorbed water by the need of a greater separation force
- *Cellular water*. Is part of the solid phase and can only be removed by the change of the water aggregation state, that is, through freezing or evaporation

b) Sludge density

The density of the sludge during most of its processing is very close to water. Usual values are between 1.02 and 1.03 (1020 to 1030 kg/m^3) for the liquid sludge during its treatment, and between 1.05 and 1.08 (1050 to 1080 kg/m^3) for the dewatered sludge going to final disposal.

c) Expression of the concentration of dry solids

The concentration of solids in the sludge is expressed in the form of dry solids, that is, excluding the water content of the sludge. The concentration can be in mg/L or in % (the latter being more frequent for sludge processing) and both are related by:

$$\boxed{\text{Concentration (\%)} = \frac{\text{Concentration (mg/L)} \times 100}{1 \times 10^6 \text{(mg/kg)} \times \text{Density (kg/L)}}} \tag{5.2}$$

Since in most of the sludge processing stages the specific gravity is very close to 1.0 (except for the dewatered sludge), Equation 5.2 can be simplified to the following:

$$\boxed{\text{Concentration (\%)} \approx \frac{\text{Concentration (mg/L)}}{10{,}000}} \tag{5.3}$$

For instance, a sludge with a concentration of 20,000 mg/L could have this same concentration expressed as 20,000/10,000 = 2.0% of dry solids. Thus, each 100 kg (or 100 litres) of sludge has 2 kg of dry solids (and 98 kg of water). To clarify the example even further, it can be said 1000 kg (or 1000 litres, or 1 m^3) of sludge have 20 kg of dry solids (or 20,000 g of dry solids). Hence, there are 20,000 g of dry solids in 1 m^3 of sludge or 20,000 gTS/m^3 or 20,000 mgTS/L (mg/L = g/m^3).

d) Relation between flow, concentration and load

The design of the sludge treatment and final disposal stages is based on the sludge flow (volume per unit time) or in many cases, the dry solids load (mass per unit time). The sludge flow is related to the SS load and concentration by:

$$\text{Flow} = \text{Load / Concentration} \tag{5.4}$$

$$\boxed{\text{Sludge flow (m}^3\text{/d)} = \frac{\text{SS load (kgSS/d)}}{\frac{\text{Dry solids (\%)}}{100} \times \text{Sludge density (kg sludge/m}^3\text{sludge)}}} \tag{5.5}$$

Considering that the density of the sludge in practically all of its processing stages is very close to 1000 kg/m^3, Equation 5.5 can be simplified to:

$$\boxed{\text{Sludge flow (m}^3\text{/d)} = \frac{\text{SS load (kgSS/d)}}{\text{Dry solids (\%)} \times 10}} \tag{5.6}$$

A sludge with a solids load of 120 kgSS/d and a solids concentration of 2.0 % (20,000 mg/L) will have a flow of $120/(2.0 \times 10) = 120/20 = 6.0$ m^3/d.

To estimate the SS load from the sludge flow and SS concentration, the rearrangement of the previous equations can be used:

Load = Flow × Concentration

$$\boxed{\text{Load (kgSS/d)} = \frac{\text{Flow (m}^3\text{/d)} \times \text{Concentration (g/m}^3\text{)}}{\text{1000 (g/kg)}}} \qquad (5.7)$$

The conversion of the units is based on the fact that mg/L is the same as g/m^3 (as seen above). A sludge with 20,000 mg/L is the same as a with 20,000 g/m^3. If the flow is 6 m^3/d, the solids load will be $6 \times 20{,}000/1000 = 120$ kg of dry solids per day (or 120 kgSS/d or 120 kgTS/d)

For an approximate calculation, it can still be said that the sludge volume (flow) varies inversely with the dry solids concentration (for a sludge with a specific gravity equal to 1.0):

$$\boxed{\frac{\text{Flow sludge}_1}{\text{Flow sludge}_2} = \frac{\text{Conc. SS}_2\ (\%)}{\text{Conc. SS}_1\ (\%)}} \qquad (5.8)$$

Therefore, a sludge with a SS concentration of 2.0% and a flow of 6 m^3/d will have the following flow, if the SS concentration is raised to 5.0%: (6 m^3/d ×2.0)/5.0 = 2.4 m^3/d. The other 3.6 m^3/d (= 6.0 – 2.4) are the removed liquid from this stage, which needs to be returned to the head of the works.

5.3 QUANTITY OF SLUDGE GENERATED IN THE WASTEWATER TREATMENT PROCESSES

Table 5.2 presents typical sludge removal intervals from the treatment units of the liquid phase, from where the sludge goes to the treatment stage. The intervals are expressed as: *continuous, hours, days, weeks, months, years*, and *decades*. For example, the classification of 'months' indicates that the sludge must be removed in the order of a few months from the treatment unit in the liquid phase to go on to the processing stage in the solid phase. The storage period has a large influence on the sludge characteristics and, as a result, on its treatment. Sludges removed in intervals of weeks, months, years or decades are usually thicker and already digested.

The quantity of sludge generated in sewage treatment, and that should be directed to the sludge treatment stage, can be expressed in terms of mass and volume. Various chapters in the present book detail the methodology of calculating the masses and volumes of the sludge produced in each treatment system. In the present section, a simplified approach of expressing the sludge production in *per capita* terms is adopted.

Table 5.2 also presents typical values for the production of liquid sludge (to be treated) and the dewatered sludge (to be disposed of or reused). As mentioned in

Table 5.2. Frequency of removal, treatment stages and characteristics of the sludge generated and to be disposed of, according to various sewage treatment processes

System	Sludge removal interval from the liquid phase		Usual stages of sludge processing				Liquid sludge (to be treated)			Dewatered sludge (to be disposed of)		
	Primary sludge	Biological sludge	Thickening	Digestion	Dewatering	Final disposal	Dry solids level (%) (a)	Sludge mass (gSS/inhab.d) (b)	Sludge volume (L/ inhab.d) (c)	Dry solids level (%) (a) (d)	Sludge mass (gSS/inhab.d)	Sludge volume (L/inhab.d) (c)
Primary treatment (septic tank)	Months	–	–	–	PS	PS	3–6	20–30	0.3–1.0	30–40	20–30	0.05–0.10
Conventional primary treatment	Hours	–	PS	PS	PS	PS	2–4	35–45	0.9–2.0	25–45	25–28	0.05–0.11
Advanced primary treatment	Hours	–	PS/CS	PS/CS	PS/CS	PS/CS	1–3	60–70	2.0–7.0	20–35	40–60	0.11–0.30
Facultative pond	–	Decades	–	–	BS (e)	BS	5–15	20–25	0.1–0.25	30–40	20–25	0.05–0.08
Anaerobic pond – facultative pond	–	Years	–	–	BS (e)	BS	–	26–55	0.15–0.45	30–40	26–55	0.06–0.17
Facultative aerated lagoon	–	Years	–	–	BS (e)	BS	4–10	8–24	0.08–0.60	30–40	8–24	0.02–0.08
Complete-mix aerated lagoon + sedim. pond	–	Years	–	–	BS (e)	BS	3–8	12–30	0.15–1.0	30–40	12–30	0.03–0.10
Anaerobic pond + facult. pond + maturation pond	–	Years	–	–	BS (e)	BS	–	26–55	0.15–0.45	30–40	26–55	0.06–0.17
Anaerobic pond + facultative pond + high rate pond	–	Years	–	–	BS (e)	BS	–	26–55	0.15–0.45	30–40	26–55	0.06–0.17
Anaerobic pond + facult. pond + algae removal	–	Years	–	–	BS (e)	BS	–	30–60	0.17–0.52	30–40	30–60	0.07–0.20
Slow rate treatment	–	–	–	–	–	–	–	–	–	–	–	–
Rapid infiltration	–	–	–	–	–	–	–	–	–	–	–	–
Overland flow	–	–	–	–	–	–	–	–	–	–	–	–
Wetlands	–	–	–	–	–	–	–	–	–	–	–	–
Septic tank + anaerobic filter	Months	Months	–	–	PS/BS	PS/BS	1.4–5.4	27–39	0.5–2.8	30–40	27–39	0.07–0.13
Septic tank + infiltration	Months	–	–	–	PS	PS	3–6	20–30	0.3–1.0	30–40	20–30	0.05–0.10
UASB reactor	–	Weeks	–	–	BS	BS	3–6	12–18	0.2–0.6	20–45	12–18	0.03–0.09
UASB + activated sludge	–	Weeks	–	–	BS	BS	3–4	20–32	0.5–1.1	20–45	20–32	0.04–0.16
UASB + submerged aerated biofilter	–	Weeks	–	–	BS	BS	3–4	20–32	0.5–1.1	20–45	18–30	0.04–0.15
UASB + anaerobic filter	–	Weeks	–	–	BS	BS	3–4	15–25	0.4–0.8	20–45	15–25	0.03–0.13
UASB + high rate trickling filter	–	Weeks	–	–	BS	BS	3–4	20–32	0.5–1.1	20–45	18–30	0.04–0.15
UASB + dissolved air flotation	–	Weeks	–	–	BS/CS	BS/CS	3–4	33–40	0.8–1.3	20–45	33–40	0.07–0.20

UASB + polishing pond	–	Weeks	–	–	BS	BS	3–4	15–20	0.4–0.7	20–45	15–20	0.03–0.10
UASB + facultative aerated lagoon	–	Weeks	–	–	BS	BS	–	20–25	0.4–0.8	20–45	20–25	0.04–0.13
UASB + compl.-mix. aerated lagoon + sedim. pond	–	Weeks	–	–	BS	BS	–	20–25	0.4–0.8	20–45	20–25	0.04–0.13
UASB + overland flow	–	Weeks	–	–	BS	BS	3–6	12–18	0.2–0.6	20–45	12–18	0.03–0.09
Conventional activated sludge	Hours	~Continuous	MS	MS	MS	MS	1–2	60–80	3.1–8.2	20–40	38–50	0.10–0.25
Activated sludge – extended aeration	–	~Continuous	BS	–	BS	BS	0.8–1.2	40–45	3.3–5.6	15–35	40–45	0.11–0.29
Sequencing batch reactor (extended aeration)	–	Hours	BS	–	BS	BS	0.8–1.2	40–45	3.3–5.6	15–35	40–45	0.11–0.29
Convent. activ. sludge with biological N removal	Hours	~Continuous	MS	MS	MS	MS	1–2	60–80	3.1–8.2	20–40	38–50	0.10–0.25
Convent. activ. sludge with biological N/P removal	Hours	~Continuous	MS	MS	MS	MS	1–2	60–80	3.1–8.2	20–40	38–50	0.10–0.25
Conventional activated sludge + tertiary filtration	Hours	~Continuous	MS	MS	MS	MS	1–2	65–85	3.2–8.5	20–40	43–55	0.11–0.28
Low rate trickling filter	Hours	Hours	–	–	MS	MS	1.5–4.0	38–47	1.0–3.1	20–40	38–47	0.09–0.22
High rate trickling filter	Hours	Hours	MS	MS	MS	MS	1.5–4.0	55–75	1.4–5.2	20–40	38–47	0.09–0.22
Submerged aerated biofilter with nitrification	Hours	Hours	MS	MS	MS	MS	1–2	60–80	3.1–8.2	20–40	38–50	0.10–0.25
Submerged aerated biofilter with biol. N removal	Hours	Hours	MS	MS	MS	MS	1–2	60–80	3.1–8.2	20–40	38–50	0.10–0.25
Septic tank + rotating biological contactor	Months	Hours	–	–	PS/BS	PS/BS	1–4	25–40	0.9–4.0	20–40	25–40	0.06–0.20

Source: Qasim (1985), EPA (1979, 1987), Metcalf & Eddy (1991), Jordão and Pessoa (1995), Franci (1996), Aisse et al (1999), Chernicharo (1997), Franci (1999), Alem Sobrinho and Jordão (2001), Alem Sobrinho (2001), von Sperling and Gonçalves (2001)

Notes: PS = primary sludge; BS = biological sludge; CS = chemical sludge; MS = mixed sludge

(a) Dry solids content (%) = 100 – water content (%)

(b) In units with a long sludge detention time (e.g. ponds, septic tanks, UASB reactors, anaerobic filters), the presented values include the digestion and the thickening that occurs in the unit (which reduce the mass and the volume of the sludge).

(c) Litres of sludge / inhab.d = [(gSS/inhab.d) / (dry solids (%))] x (100/1000) (assuming a density of 1000 kg/m^3)

(d) The broad variation of the ranges of dry solids reflects different technologies (natural and mechanised), operating in distinct climatic conditions

(e) The mass of solids in the dewatered sludge becomes lower than the sludge to be treated when the sludge undergoes digestion in the treatment process. In the UASB reactor process followed by aerobic post treatment (e.g. activated sludge, trickling filters and submerged aerated biofilters), the aerobic sludge is returned to the UASB reactor, where it undergoes digestion and thickening, together with the anaerobic sludge. In these cases, the presented values correspond to the combined sludge mixture extracted from the UASB reactor.

the previous item, the sludge mass, expressed as solids, represents the fraction of solids of the sludge generated. The rest of the sludge consists of pure water. The calculation of the sludge volume produced per capita per day is done based on the daily per capita load and the dry solids concentration.

5.4 SLUDGE TREATMENT STAGES

The main stages in sludge management, with their respective objectives are:

- **Thickening**: removal of water (volume reduction)
- **Stabilisation**: removal of organic matter – volatile solids (mass reduction)
- **Conditioning**: preparation for dewatering (principally mechanical)
- **Dewatering**: removal of water (volume reduction)
- **Disinfection**: removal of pathogenic organisms
- **Final disposal**: final destination of the by-products

The incorporation of each of these stages in the sludge-processing flowsheet depends on the characteristics of the sludge produced or, in other words, on the treatment system used for the liquid phase, as well as on the subsequent sludge-treatment stage and on the final disposal.

Thickening is a physical process of concentrating the sludge, with the aim of reducing its water content and, as a result, its volume, facilitating the subsequent sludge treatment stages.

Stabilisation aims at attenuating the inconveniences associated with the generation of bad odours during processing and disposing of the sludge. This is accomplished through the removal of the biodegradable organic matter of the sludge, what also brings about a reduction in the solids mass in the sludge.

Conditioning is a sludge preparation process, based on the addition of chemical products (coagulants, polyelectrolytes) to increase its dewatering capability and to improve the capture of solids in the sludge dewatering systems.

The next stage is the *dewatering* of the sludge, which can be done through natural or mechanical methods. The objective of this phase is to remove water and reduce the volume even further, producing a sludge with a mechanical behaviour close to solids. The dewatering of the sludge has an important impact in its transport and final disposal costs, besides influencing its subsequent handling, since the mechanical behaviour varies with the water content level.

The *disinfection* of the sludge is necessary if its destination is for agricultural recycling, since the anaerobic or aerobic digestion processes usually employed do not reduce the pathogens content to acceptable levels. Disinfection is not necessary if the sludge is to be incinerated or disposed of in landfills.

Table 5.2 presents the sludge management stages usually adopted for the most frequently used sewage treatment systems. There are process variants within each stage, with the main ones being presented in Table 5.3.

Table 5.3. Sludge management stages and main processes used

Thickening	Stabilisation	Conditioning	Dewatering	Disinfection	Final disposal
• Gravity thickening	• Anaerobic digestion	• Chemical conditioning	• Drying beds	• Lime addition	• Agricultural recycling
• Flotation	• Aerobic digestion	• Thermal conditioning	• Sludge lagoons	• Thermal treatment	• Recovery of degraded areas
• Centrifuge	• Thermal treatment		• Filter press	• Composting	• Land farming
• Belt filter press	• Chemical stabilisation		• Centrifuge	• Wet air oxidation	• Non agricultural use (small slabs manufacturing, fuel, etc.)
			• Belt filter press	• Others (gamma radiation, etc.)	
			• Vacuum filter		• Incineration
			• Thermal drying		• Wet air oxidation
					• Sanitary landfill

The flowsheets of sludge treatment systems allow various combinations of unit operations and process, comprising different sequences, as a function of the sludge characteristics, sewage treatment processes and final disposal methods. Figure 5.1 shows examples of sludge treatment flowsheets frequently used.

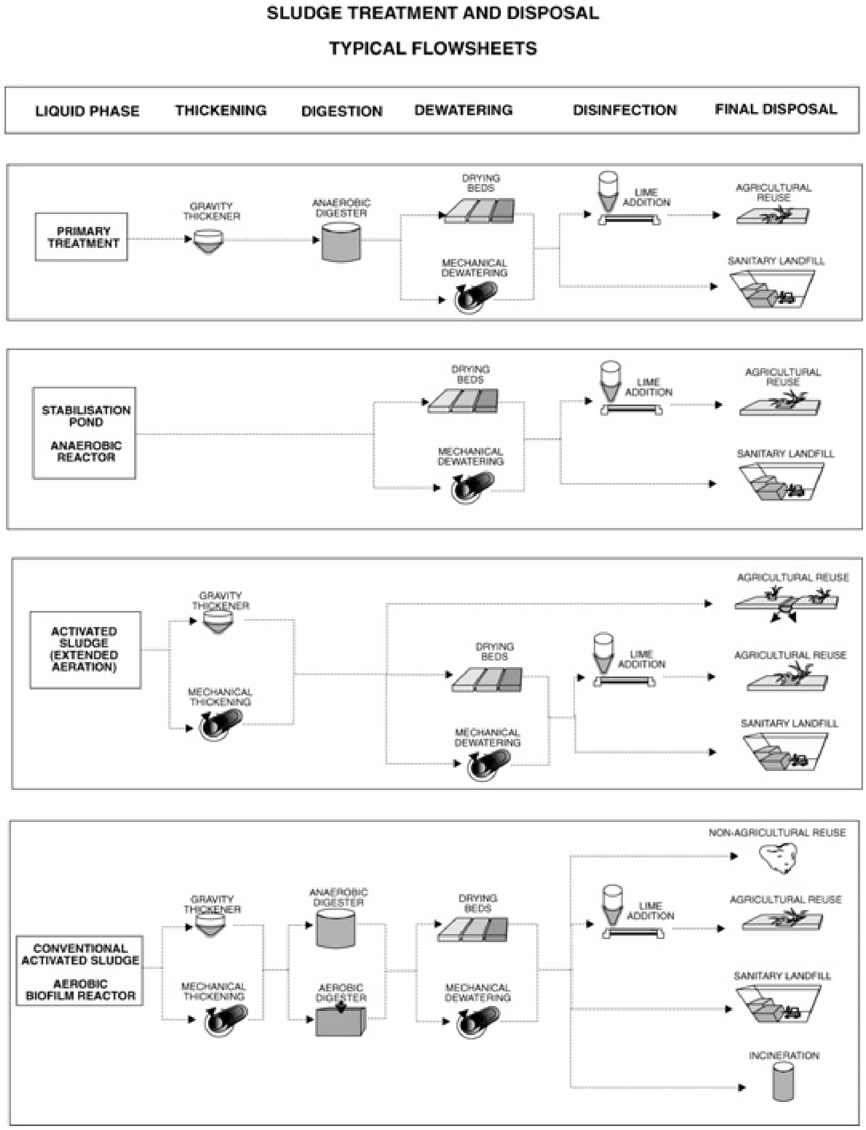


Figure 5.1. Usual sludge treatment and disposal flowsheets (schematic representation of processes frequently employed; certain stages can be optional and in each stage there are various process variants; see Tables 5.2 and 5.3)

Table 5.4. Typical uses of the main sludge thickening methods

Thickening method	Sludge type	Comment
Gravity	Primary	Frequently used, with excellent results
	Activated sludge	Less frequent, owing to the small increase in the solid levels
	Mixed sludge (primary sludge and activated sludge)	Frequently used
	Mixed sludge (primary sludge and sludge from the aerobic biofilm reactor)	Frequently used
Dissolved air flotation	Mixed sludge (primary sludge and activated sludge)	Less frequent use, since results are similar to gravity thickeners
	Activated sludge	Frequently used, with much better results than gravity thickening
Centrifuge	Activated sludge	Increasing use
Belt press	Activated sludge	Increasing use

Source: adapted from Metcalf and Eddy (1991) and Jordão and Pessôa (1995)

5.5 SLUDGE THICKENING

The main processes used for sludge thickening are:

- gravity thickeners
- dissolved air flotation
- centrifuges
- belt presses

Other mechanical processes used for dewatering can also be adapted to sludge thickening. Table 5.4 presents the typical uses of these processes.

Table 5.5 presents typical thickened sludge concentrations, according to the thickening processes employed.

Gravity thickeners have a similar structure to settling tanks. The format is usually circular with central feeding, a bottom sludge exit and a supernatant side exit. The thickened sludge goes on to the next stage (normally digestion), while the supernatant is returned to the head of the works. Figure 5.2 presents a schematics of a gravity thickener.

In the process of *dissolved air flotation*, air is introduced in a solution maintained at high pressure. In these conditions, the air is dissolved. When there is a depressurisation, the dissolved air is released in the form of small bubbles. These bubbles, with an upward movement, tend to carry the sludge particles to the surface, from where they are removed. Thickening by flotation has a good applicability for activated sludge, which does not thicken well in gravity thickeners. Dissolved air flotation also has good applicability in WWTPs with biological phosphorus

Table 5.5. Dry solids levels in thickened sludges, according to process

Sewage treatment process	Process	Dry solids level (%)
Primary treatment (conventional)	Gravity	4–8
Conventional activated sludge		
• Primary sludge	Gravity	4–8
• Secondary sludge	Gravity	2–3
	Flotation	2–5
	Centrifuge	3–7
• Mixed sludge	Gravity	3–7
	Centrifuge	4–8
Activated sludge – extended aeration	Gravity	2–3
	Flotation	3–6
	Centrifuge	3–6
High rate trickling filter		
• Primary sludge	Gravity	4–8
• Secondary sludge	Gravity	1–3
• Mixed sludge	Gravity	3–7
Submerged aerated biofilter		
• Primary sludge	Gravity	4–8
• Secondary sludge	Gravity	2–3
	Flotation	2–5
	Centrifuge	3–7
• Mixed sludge	Gravity	3–7
	Centrifuge	4–8

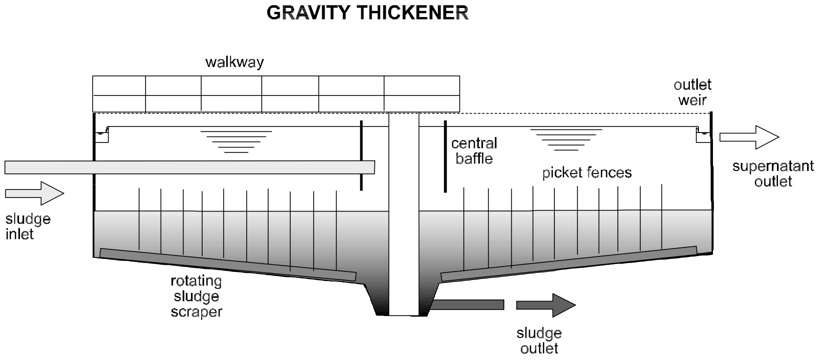


Figure 5.2. Schematics of a gravity thickener

removal, in which the sludge needs to remain in aerobic conditions in order not to release the phosphorus into the liquid mass.

The other mechanised thickening processes are described in Section 5.7.

5.6 SLUDGE STABILISATION

Raw sewage sludge is rich in microorganisms, decomposes easily and quickly releases offensive odours. The stabilisation processes were developed with the

Table 5.6. Comparison between raw and anaerobically digested sludge

Raw sludge	Digested sludge
Unstable organic matter	Stabilised organic matter
High biodegradable fraction in the organic matter	Low proportion of the biodegradable fraction
High potential in the generation of odours	Low potential in the generation of odours
High concentration of pathogens	Concentration of pathogens lower than in raw sludge

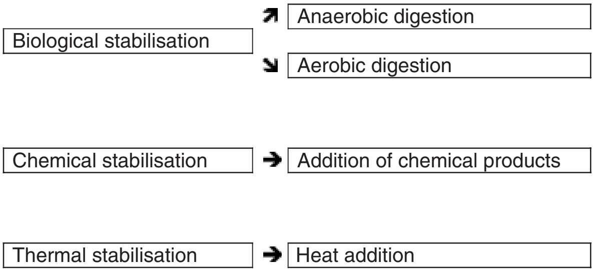


Figure 5.3. Main sludge stabilisation processes

objective of stabilising (digesting) the biodegradable fraction of the organic matter present in the sludge, thus decreasing the risk of putrefaction, as well as reducing the concentration of pathogens. Table 5.6 shows the main differences between raw and anaerobically digested sludge.

The stabilisation processes can be divided into (see Figure 5.3):

- **biological stabilisation**: use of specific bacteria to promote the stabilisation of the biodegradable fraction of the organic matter
- **chemical stabilisation**: the stabilisation is achieved by the chemical oxidation of the organic matter
- **thermal stabilisation**: obtained from the action of heat on the volatile fraction in hermetically closed recipients.

Anaerobic digestion is the most frequently used sludge stabilisation process. Aerobic digestion is less diffused, but has a good applicability in the stabilisation of the excess activated sludge originating from WWTPs with biological nutrient removal. Composting processes are common in urban solid waste treatment works, but only in a limited number of small-scale WWTPs. Alkaline treatment and thermal drying are other processes used in sludge stabilisation.

The main methods of final sludge disposal associated with different stabilisation processes can be found in Table 5.7. The stabilisation of the sludge facilitates its final disposal and opens alternatives for its reuse as an agricultural soil conditioner.

From the various sewage treatment systems listed in Table 5.2, it is seen that the degree of stabilisation of the sludge produced varies according to the treatment process used.

Table 5.7. Sludge stabilisation technologies and final disposal methods

Treatment process	Use or final disposal method
Anaerobic/aerobic digestion	Produces a biosolid that is suitable to be used with restrictions in agriculture, such as a soil conditioner and organic fertiliser. Usually followed by dewatering. Needs disinfection post-treatment for unrestricted use in agriculture.
Chemical treatment (lime stabilisation)	Agricultural use or in the daily cover of a sanitary landfill.
Composting	Agricultural humus-like product, appropriate for use in nurseries, horticulture and landscaping. Usually adopted after sludge dewatering.
Thermal drying	Product with a high level of solids, significant nitrogen concentration and free from pathogens. Indicated for unrestricted agricultural use.

The process of anaerobic digestion has been known by sanitary engineers since the end of the 19th century and is characterised by the stabilisation of the organic matter in an environment free from molecular oxygen. Owing to its robustness and high efficiency, anaerobic digestion is present from simple domestic septic tanks acting as an individual residential solution, up to completely automated plants, serving large metropolitan regions.

In a conventional activated sludge or trickling filter plant, the mixture between primary sludge and excess biological sludge is stabilised biologically under anaerobic conditions and converted into methane (CH_4) and carbon dioxide (CO_2). The process is done in closed biological reactors known as anaerobic digesters. The digester is fed in a continuous or batch form and the sludge is maintained inside it for a certain detention time.

The anaerobic digesters are constructed of concrete or steel. The raw sludge is mixed – and heated in temperate climate countries – with the gas produced, and the gas is stored in floating gasholders for processing or burning. The configuration of the digesters varies according to land availability, the need for maintaining a completely-mixed regime and the removal of grit and scum. Figure 5.4 illustrates cylindrical and oval anaerobic digesters.

5.7 SLUDGE DEWATERING

Dewatering is done with digested sludges and has an important impact on the sludge transportation and final disposal costs. The main reasons for sludge dewatering are:

- reduction of the transportation costs to the final disposal site;
- improvement of the handling conditions of the sludge, since dewatered sludge is easier to be handled and transported;
- increase of the calorific value of the sludge, through the reduction of the water aiming at preparing it for incineration;
- reduction in the volume for disposal in a landfill or for agricultural use;

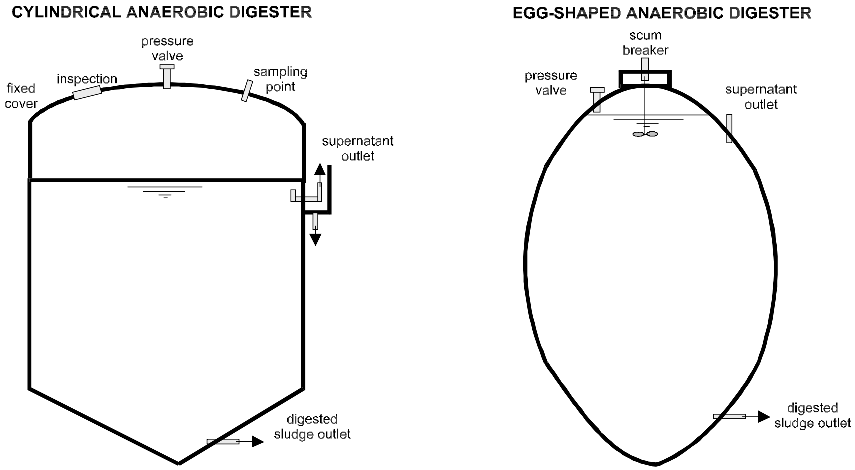


Figure 5.4. Typical shapes of anaerobic digesters (adapted from WEF, 1996)

- reduction in the production of leachate when the sludge is disposed of in landfills.

Sludge dewatering can be done by *natural* or *mechanised* processes. Natural processes use evaporation and percolation as the main water removal mechanisms, thus requiring more time for dewatering. Although simpler and cheaper to operate, they need larger areas and volumes for installation. In contrast, the mechanised processes are based on mechanisms such as filtration, compaction, or centrifugation to accelerate dewatering, resulting in compact and sophisticated units, from an operational and maintenance point of view.

Many variables influence the selection of the dewatering process, but the sludge type and land availability are the most important ones. Natural processes such as drying beds are considered the best alternative for small scale WWTPs located in regions where there are no area restrictions. In the same way, medium and large scale WWTPs located in metropolitan areas tend to use mechanical dewatering.

The main sludge dewatering processes are listed below:

Natural

- Drying beds
- Sludge lagoons

Mechanised

- Centrifuges
- Vacuum filters
- Belt presses
- Filter presses

Typical dry solids levels obtained from dewatering processes applied to sludge originating from various wastewater treatment processes are presented in Table 5.8.

Table 5.8. Typical dry solids levels in dewatered sludges from various wastewater treatment processes

Sewage treatment system	Dewatering process	Dry solids level in the dewatered sludge (%)
Primary treatment (conventional)	Drying bed	35–45
	Filter press	30–40
	Centrifuge	25–35
	Belt press	25–40
Primary treatment (septic tank)	Drying bed	30–40
Facultative pond	Drying bed	30–40
Anaerobic pond – facultative pond	Drying bed	30–40
Facultative aerated lagoon	Drying bed	30–40
Completely-mixing aerated lagoon – sedimentation pond	Drying bed	30–40
Septic tank + anaerobic filter	Drying bed	30–40
Conventional activated sludge (mixed sludge)	Drying bed	30–40
	Filter press	25–35
	Centrifuge	20–30
	Belt press	20–25
Activated sludge – extended aeration	Drying bed	25–35
	Filter press	20–30
	Centrifuge	15–20
	Belt press	15–20
High rate trickling filter (mixed sludge)	Drying bed	30–40
	Filter press	25–35
	Centrifuge	20–30
	Belt press	20–25
Submerged aerated biofilter (mixed sludge)	Drying bed	30–40
	Filter press	25–35
	Centrifuge	20–30
	Belt press	20–25
UASB reactor	Drying bed	30–45
	Filter press	25–40
	Centrifuge	20–30
	Belt press	20–30
UASB reactor + activated sludge (combined sludge)	Drying bed	30–45
	Filter press	25–40
	Centrifuge	20–30
	Belt press	20–30
UASB reactor + aerobic biofilm reactor (combined sludge)	Drying bed	30–45
	Filter press	25–40
	Centrifuge	20–30
	Belt press	20–30

- Mixed sludge = primary sludge + secondary sludge
- Combined sludge = anaerobic sludge + aerobic sludge resulting from post treatment and returned to the anaerobic reactor
- The wide ranges of dry solids reflect distinct climatic conditions and operational modes

Table 5.9. Main characteristics of sludge dewatering processes

	Natural processes		Mechanised processes			
Characteristics	Drying beds*	Sludge lagoons	Centrifuges	Vacuum filters	Belt presses	Filter presses
Land requirements	+ + +	+ + +	+	+ +	+	+
Energy requirements	–	–	+ +	+ + +	+ +	+ + +
Implementation cost	+	+	+ + +	+ +	+ +	+ +
Operational complexity	+	+	+ +	+ +	+ +	+ + +
Maintenance requirements	+	+	+ +	+ +	+ + +	+ + +
Complexity of installation	+	+	+ +	+ +	+ +	+ +
Influence of climate	+ + +	+ + +	+	+	+	+
Sensitivity to the sludge quality	+	+	+ + +	+ +	+ +	+ +
Chemical products	+	–	+ + +	+ + +	+ + +	+ + +
Sludge removal complexity	+ +	+ + +	+	+	+	+
Level of dry solids in the cake	+ + +	+ +	+ +	+	+ +	+ + +
Odours and vectors	+ +	+ + +	+	+	+	+
Noise and vibration	–	–	+ + +	+ +	+ +	+ +
Groundwater contamination	+ +	+ + +	+	+	+	+

\+ Little, reduced + + + large, high, very
cake = dewatered sludge
(*) drying bed: a dewatering cycle of 30 days assumed

The main characteristics, advantages and disadvantages of the various dewatering methods are listed in Table 5.9.

To increase the dewatering capability and the solids capture (solids incorporated in the sludge), the sludge can be submitted to a conditioning stage before the dewatering stage itself. The conditioning can be accomplished using chemical products or physical processes; the most common of the latter is the heating of the sludge. The chemical products are applied to the sludge upstream of the dewatering unit, favouring the aggregation of the solids particles and the formation of flocs. The conditioning can be also employed upstream of the mechanised thickening units. The main coagulants used are metallic salts and polyelectrolytes (polymers). The most common metallic coagulants are:

- aluminium sulphate
- ferric chloride
- ferrous sulphate
- ferric sulphate
- quicklime/ hydrated lime

The polymers are organic compounds, usually synthetic, of high molecular weight that can be used as coagulants or flocculating aids. Depending on the

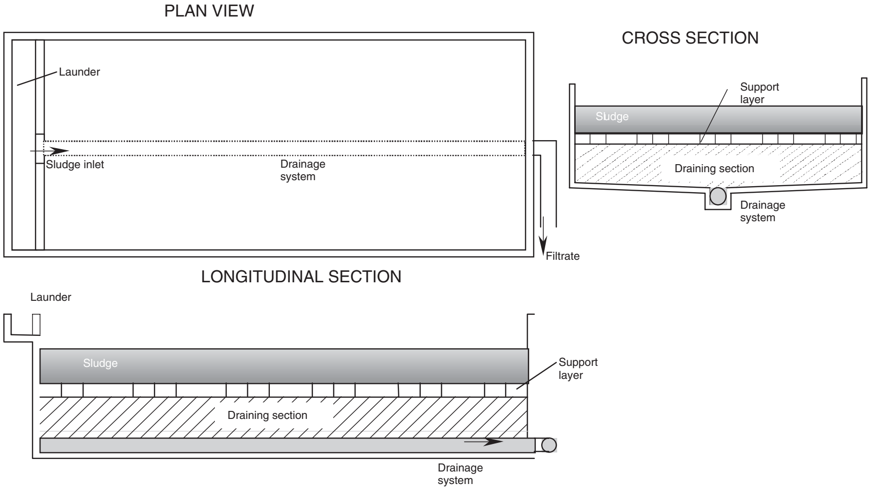


Figure 5.5. Schematics of a sludge drying bed (Gonçalves, 1999)

prevailing surface charge, the polymers are classified into cationic, anionic and non ionic.

A brief description of the main dewatering processes is presented below. A detailed description is included in Chapter 50.

a) Sludge drying beds

Drying beds are one of the oldest techniques and very much used for solids-liquid separation in sludge. The construction costs are generally low in comparison with mechanical dewatering options, especially for small-sized communities. The process generally has a rectangular tank with masonry or concrete walls and a concrete bottom. On the inside of the tank are the following devices to drain the water present in the sludge (Figure 5.5):

- support layer (bricks and coarse sand), on top of which the sludge is placed
- draining medium (fine to coarse sand followed by fine to coarse gravel)
- drainage system (open or perforated pipes)

Part of the liquid evaporates and part percolates through the sand and support layer. The dewatered sludge stays in the layer above the sand.

Drying beds are suggested for small and medium sized communities with the STWs treating a population equivalent of up to around 20,000 inhabitants. The main advantages and disadvantages of using drying beds are presented in Table 5.10.

b) Sludge drying lagoon

Sludge drying lagoons are used for thickening, complementary digestion, dewatering and even for the final disposal of sewage sludge. Drying lagoons are generally

Table 5.10. Advantages and disadvantages of sludge drying beds

Advantages	Disadvantages
• Low construction costs	• Large area required
• Operational simplicity	• Previous stabilisation of the sludge required
• Low level of attention required	• Significant climate influence on the operational performance of the process
• Operator with a low qualification level required	• Slow removal of the sludge cake
• Low or non existent electrical energy consumption	• Requires a high quantity of labour to remove the dry cake
• Low or non existent consumption of chemical products	• High risk of odour release and proliferation of flies
• Low sensitivity to variations of the sludge characteristics	• Contamination risk of the groundwater, in case the bottom and the drainage system of the beds are not well executed
• Cake with high solids level	

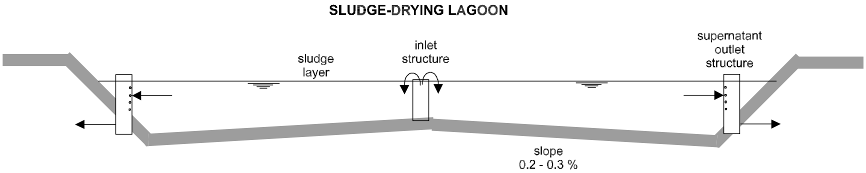


Figure 5.6. Schematics of a sludge drying lagoon (adapted from EPA, 1987)

excavated in the soil, located in natural depressions in the land, or put inside banks, where the discarded sludge from the WWTPs is accumulated for prolonged time periods (from 3 to 5 years). During this period, the sludge is thickened by the action of gravity, further digested by the microorganisms present in the sludge, and dewatered through drainage of the free water and evaporation. The process is only recommended for dewatering previously digested sludge by aerobic or anaerobic methods, and not for use in the dewatering of primary or mixed sludge. *Among the natural dewatering processes, the sludge lagoons are much less used than drying beds*.

The main difference between this process and the drying beds resides in the fact that evaporation is the principal mechanism of influence in the dewatering process. Percolation has a lesser effect than in the drying beds. The dewatering in the lagoon can be accelerated with the use of devices for the removal of the supernatant water at various levels after the loading of the sludge (Figure 5.6). The use of drains at the bottom is not common practice in drying lagoons, because the sewage sludge has reduced drainability and the risk of pipe blocking is very likely to occur.

When the lagoon is full, it can be put out of operation without the removal of the sludge, thus serving as a solution for final disposal. Another possibility is the removal of the sludge from the full lagoon, allowing its reuse and utilisation as a continuous dewatering unit.

Table 5.11. Advantages and disadvantages of sludge lagoons

Advantages	Disadvantages
• Very small energy consumption • Absence of chemical products • Little sensitivity to the variation of the sludge characteristics • Complementary stabilisation of the organic matter in the sludge • Low requirement of skilled labour • Spare unit in STWs with operational problems in sludge dewatering • Low implementation costs in cases where land is cheap	• Large land requirements • Possible generation of odours of difficult control • Possible pollution of ground and surface waters • Attraction of vectors, mainly mosquitoes and flies • Visual impact

The main advantages and disadvantages of sludge drying lagoons are listed in Table 5.11.

c) Centrifuge

Centrifugation is a solids/liquid separation operation forced by the action of a centrifugal force. In a first stage, the sludge particles settle at a velocity much higher than would occur under the action of gravity. In a second stage, compaction occurs when the sludge loses part of the capillary water under the prolonged action of centrifugation. The cake is removed from the process after this last dewatering stage.

Centrifuges are equipment that may be used indistinctly for sludge thickening and dewatering. The operating principle is the same, and it is possible to install the centrifuges in series, the first for the thickening of the sludge and the second for the dewatering. The main types of centrifuges used for sludge dewatering are vertical and horizontal-shaft centrifuges. The main differences are in the type of feeding of the sludge, the intensity of the centrifugal force and the manner in which the cake and the liquid are unloaded from the equipment. Currently, the majority of treatment plants that dewater sludge by centrifugation use horizontal-shaft centrifuges. The semi-continuous feeding of the sludge and relatively lower solids levels in the cake produced by the vertical-shaft centrifuges are some of the reasons for this preference.

Horizontal centrifuges can be classified according to the direction of the sludge feeding and the removal of the cake as *co-current* and *counter-current*. Their main differences reside in the sludge feeding points, in the removal of the centrate (liquid phase removal) and in the direction of the flow of the solid and liquid phases in its interior. In the *co-current* centrifuges, the solid and the liquid phases cross all the extension of the longitudinal shaft of the equipment, until they are unloaded. In the *counter-current models*, the feeding is done in the junction of the cylindrical section with the conical section of the equipment. The solid phase is transported

Table 5.12. Advantages and disadvantages of horizontal shaft centrifuges

Advantages	Disadvantages
• Can be used for sludge thickening and dewatering	• Noise and vibration
• Low land requirements	• Wearing of some components
• Ease of installation	• High energy consumption at the engine start
• Operation under high loading rates	
• Requirement of small quantities of polymers for conditioning	• Complex and slow adjustments during the start-up
• Requirement of low attention from operators	• Requirement of careful maintenance
	• High costs in many places (especially when imported)

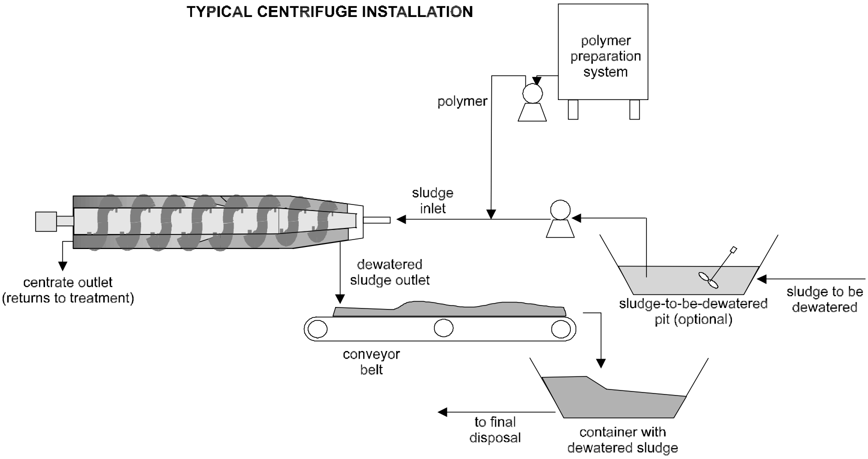


Figure 5.7. Typical set up of a decanter-type centrifuge

by an screw conveyor to the end of the conical section, while the liquid moves to the opposite direction.

The main advantages and disadvantages of the horizontal centrifuges are summarised in Table 5.12. Figure 5.7 illustrates a typical set up of a centrifuge.

d) Vacuum filter

Vacuum filters were highly used in industrialised countries for sludge dewatering up until the 1970s. When compared with more modern sludge dewatering processes, *their use entered into decline* due to the high-energy consumption and lower efficiency.

A vacuum filter consists of a rotating cylindrical drum installed with partial submergence in a tank with conditioned sludge. Around 10 to 40% of the drum

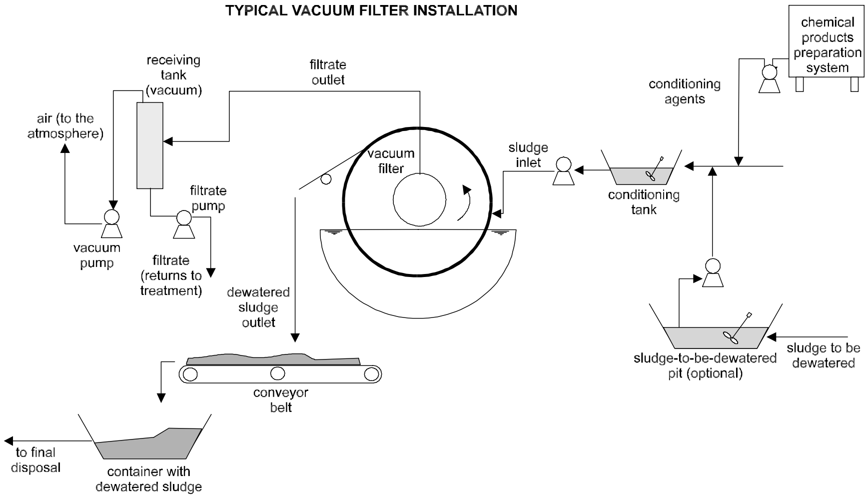


Figure 5.8. Typical set up of a vacuum filter system for sludge dewatering

surface is submerged in the tank; this part forms the filtration or the cake formation zone. The cake is formed in the outer part of the cylinder, while the filtered liquid migrates to the interior, where there is a vacuum. Next, in the direction of the rotation, there is a dewatering region that occupies between 40 to 60% of the cylinder surface. In the final region of the cylinder that is almost completing the rotation cycle there is the unloading zone. A valve brings the surface of the cylinder to the atmospheric pressure in this region, and the sludge cake is separated from the filtering medium. Figure 5.8 shows a typical set up of a vacuum filter system.

e) Filter press

A filter press operates in an intermittent mode, with cycles consisting of sludge loading, filtration, and cake unloading stages. The liquid sludge is pumped into plates surrounded by filter cloths. The pumping of the sludge increases the pressure in the space between the plates and forces the sludge to pass through the filter cloth. The solids are then retained on the filtering medium, forming the cake. Next, a hydraulic piston pushes a steel plate against the other polyethylene plates, making up the pressing. The filtrate (liquid) goes through the filter cloths and is collected by the plate outlet ports. The cake is easily removed from the filter when the pneumatic piston is retreated and the plates are separated. At this moment, the dry cake falls from the plate and can be taken to storage or final destination. Figure 5.9 presents a typical set up of a filter press.

Filter presses were developed for industrial uses and then underwent subsequent adaptations to be used for dewatering sludge. The equipment operates in

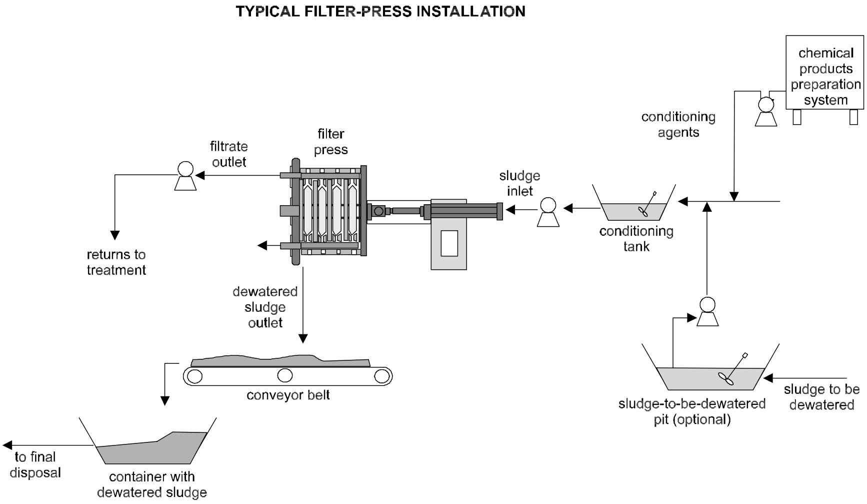


Figure 5.9. Typical set up of a filter press for sludge dewatering

batch mode, what requires the intervention of trained operators, and has as a main characteristic a high reliability. The principal advantages of filter presses are:

- high concentration of solids in the cake (higher than from other mechanised dewatering processes);
- high solids capture;
- clarified liquid effluent;
- low consumption of chemical products for sludge conditioning;
- reliability.

Currently filter presses are automated, which reduces labour needs. The weight of the equipment, its purchasing costs, and the need for the regular substitution of the filter cloths usually make the utilisation of filter presses limited to medium and large size WWTPs.

f) Belt press

Belt presses operate on a continuous mode. The operation process of a belt press can be divided into three distinct stages and zones: (a) zone of gravity drainage, (b) low-pressure zone and (c) high-pressure zone.

The zone of gravity drainage is located at the entrance of the press, where the sludge is applied onto an upper screen and the free water percolates under the action of gravity through the opening pores in the screen. After this, the sludge is directed to the low-pressure zone, where the rest of the free water is removed and the sludge is gently compressed between the upper and lower screens. In the high-rate pressure zone, which is formed by various rollers with different diameters

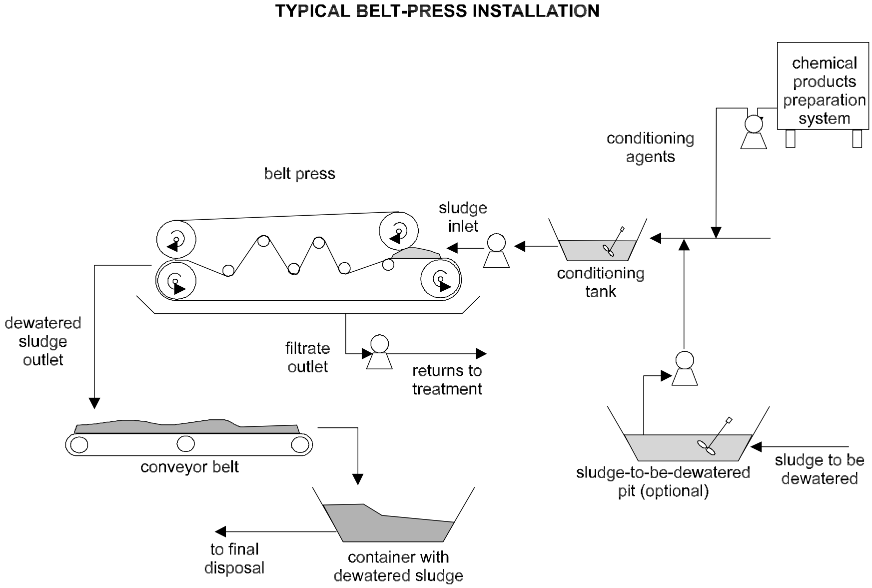


Figure 5.10. Typical set up of a belt press system for sludge dewatering

in series, the sludge is progressively compressed between two screens with the objective of releasing the interstitial water. Finally, scrapers remove the sludge and high-pressure water jets wash the screens. Figure 5.10 presents a typical set up of a belt press system.

Low acquisition costs and reduced energy consumption are the main advantages. However, since the equipment is open, the belt press may have the following disadvantages: aerosol emissions, high level of noise and eventual unpleasant odours (depending on the sludge type). Another large disadvantage of the belt press is the high number of rollers (40–50), which require operational attention and regular substitution.

5.8 SLUDGE DISINFECTION

The objective of introducing a sludge disinfection stage in the sewage treatment works is to guarantee a low level of pathogens in the sludge, such that, when it is disposed of on land, will not cause health risks to the population and to the workers that will handle it and also negative impacts to the environment. However, the need to include a complementary sludge disinfection system will depend on the final disposal alternative to be adopted.

The application of sludge in public parks and gardens or its recycling in agriculture implies a higher sanitary level than other disposal alternatives, such as landfills. These requirements can be met by a sludge disinfection process or by temporary restrictions to public use and access.

Some stabilisation processes of the organic matter in the sludge also lead to a reduction of pathogenic microorganisms, producing a sanitarily safe sludge. Others reduce the pathogenic microorganisms to levels lower than the detection limits, after the stabilisation of the organic matter, in a complementary sludge treatment stage. The most important processes are described below.

a) Composting

Composting is an aerobic organic matter decomposition process that is achieved through controlled conditions of temperature, water content, oxygen and nutrients. The resultant product (compost) has a high agricultural value as a soil conditioner. The inactivation of the pathogenic microorganisms occurs mainly by the increase of temperature during the highest activity phase of the process. The temperature reaches values between 55 and 65 °C by means of biochemical reactions.

Raw sludge and digested sludge can be composted. Materials such as wood chips, leaves, green waste, rice straw, sawdust, or other structuring agents need to be added to the sludge to improve the water retention, increase the porosity, and balance the ratio between carbon and nitrogen.

Figure 5.11 shows a typical flowsheet of the composting process.

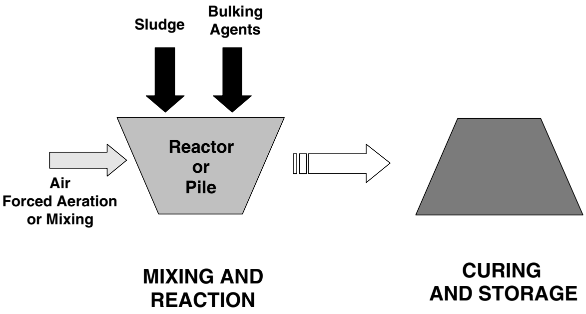


Figure 5.11. Flowsheet of the composting process

The composting process can be carried out according to the three following main methods (see Figure 5.12 and Table 5.13):

- **Windrows**. Periodical turning, in order to allow aeration and mixture. Detention time between 50 and 90 days.
- **Aerated static pile**. Aeration by perforated pipes from air blowers or exhausting systems. Detention time between 30 and 60 days.
- **In-vessel biological reactors**. Closed systems, with a greater control and lower detention time. Detention time of at least 14 days in the reactor and 14 to 21 days for cooling.

Table 5.13. Comparison between the composting methods

Composting method	Advantages	Disadvantages
Windrows	• Low investment cost • Low operation and maintenance costs	• Requirement of large areas • Possible odour problem • Difficulty in reaching the required temperature • Possible mixing problem • High composting period
Aerated static piles	• Better odour control • Better conditions for temperature maintenance • Lower reaction time	• Investment required for the aeration system • Moderate operation and maintenance costs
Biological reactors	• Small land requirement • High degree of process control • Easiness in controlling temperature and odours	• Higher investment and operation and maintenance costs • Economically feasible only for large scales

Source: Teixeira Pinto (2001)

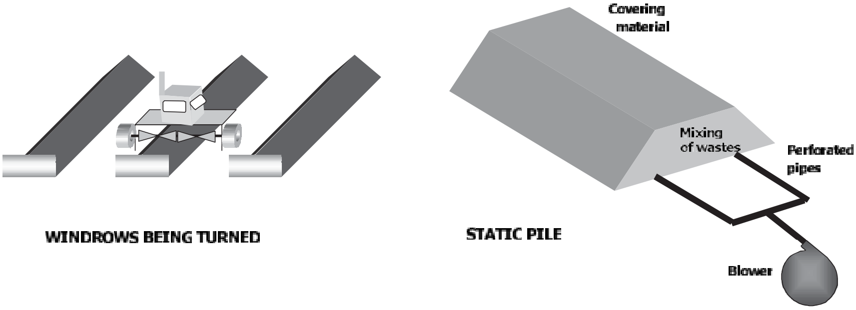


Figure 5.12. Windrows and aerated static piles (source: Teixeira Pinto, 2001)

b) Thermophilic aerobic digestion

The process of thermophilic aerobic digestion (also called autothermal digestion) follows the same principles as the conventional aerobic digestion system. The difference is that it operates in a thermophilic phase due to some alterations in the concept and operation of the system.

In this process, the sludge is generally pre-thickened and operates with two aerobic stages, without the need of introducing energy to raise the temperature. Since the reaction volume is small, the system is closed and the concentration of solids in the sludge is higher, the heat released from the aerobic reactions heats the sludge to temperatures higher than 50 °C in the first stage and 60 °C in the second. Due to the temperature increase the pathogenic microorganisms are reduced to

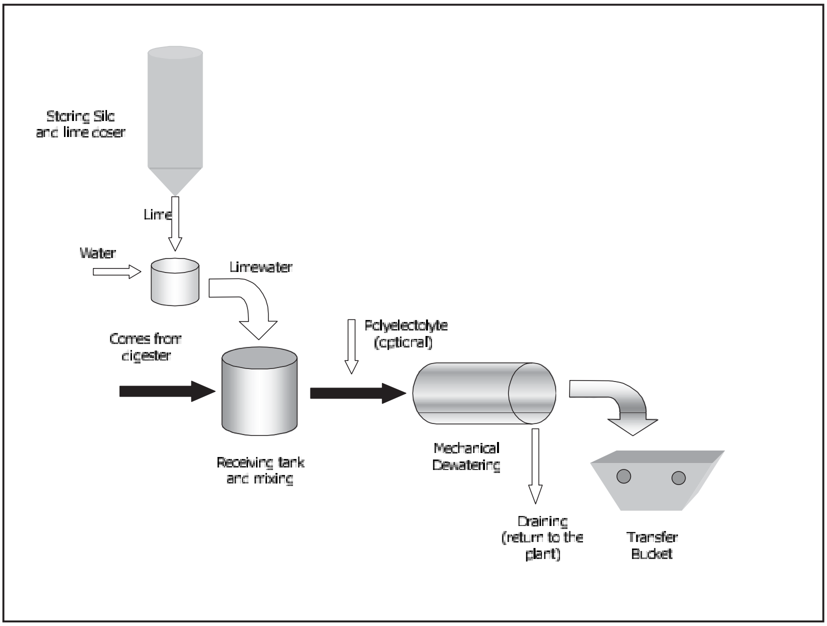


Figure 5.13. Addition of lime to liquid sludge (lime pre-treatment)

values lower than the detection limits if the sludge is maintained at a temperature between 55–60 °C for 10 days (or 5 to 6 days for reactors in series). The mixing and aeration efficiency are the two most important factors for the operational success of the system.

c) Lime stabilisation

Lime stabilisation is used to treat primary, secondary, or digested sludge. When a sufficient quantity of lime is added to the sludge to increase the pH to 12, a reduction of the population of microorganisms (including pathogens) and the potential occurrence of odours takes place. Lime can be added to liquid or dewatered sludges (see Figures 5.13 and 5.14). Owing to the addition of lime, the quantity of sludge to be disposed of increases.

d) Pasteurisation

Pasteurisation involves the heating of the sludge to 70 °C for 30 minutes, followed by a rapid cooling to 4 °C. The sludge can be heated by heat exchangers or by hot steam injection. The steam injection process is more commonly used and the sludge is pasteurised in batch to decrease the recontamination risks.

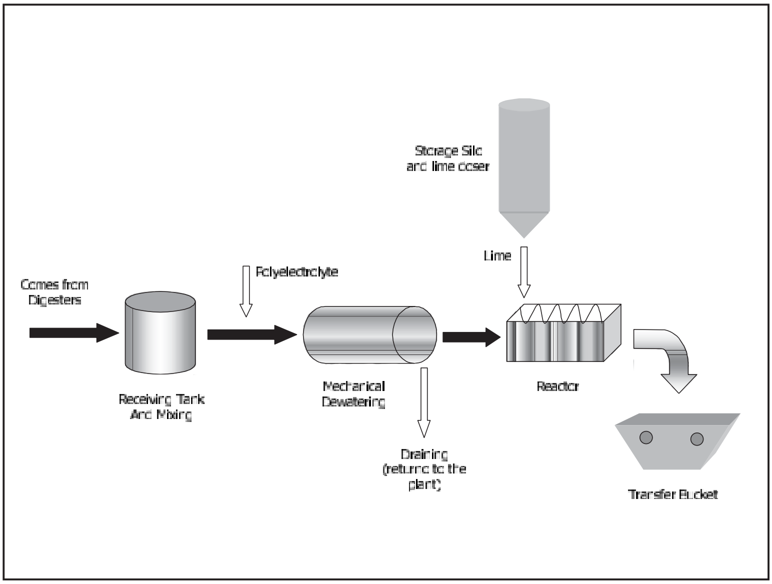


Figure 5.14. Addition of lime to dewatered sludge (lime post-treatment)

e) Thermal treatment

Thermal treatment consists of passing the sludge through a heat source that causes the evaporation of the existing moisture in the sludge and consequently the thermal inactivation of the microorganisms. To be economically feasible, the sludge needs to be previously digested and dewatered to a solids concentration in the order of 20–35%, before being thermally treated. The dried sludge has a granular aspect and presents a very high level of solids, in the region of 90–95%.

f) Comparison between the disinfection processes

Tables 5.14 and 5.15 present a comparison of the characteristics of the various sludge disinfection processes.

5.9 FINAL DISPOSAL OF THE SLUDGE

The most commonly applied alternatives for the final disposal of the sludge are summarised in Table 5.16. Table 5.17 lists the main positive and negative aspects associated with each disposal route. Whenever possible, alternatives that bring beneficial uses must be associated with the final disposal.

Table 5.14. Comparison between sludge disinfection technologies – Implementation

Process	Area	Skilled personnel	External energy	Chemical products	External biomass	Construction cost	O&M cost
Composting (windrows/ piles)	+++	+	+/++	+	+++	+	+
Composting (reactors)	++	++	++	+	+++	++	++
Thermophilic aerobic digestion	++	++	++	+	+	++	++
Pasteurisation	++	++	+++	+	+	++	++
Lime stabilisation	++	+/++	+	+++	+	+	++
Thermal drying	+	+++	+++	+	+	+++	+++

Source: Teixeira Pinto (2001)
+++: Significant importance ++: Moderate importance +: Little or no importance

Table 5.15. Comparison between sludge disinfection technologies – Operation

	Effect against pathogens						
Process	Bacteria	Viruses	Eggs	Product stability	Volume reduction	Odour potential	Observations
Composting (windrows/ piles)	+++/++	++/+	+++/++	+++	↑	+++	Effect depends on mixing
Composting (reactors)	+++	+++/++	+++	+++	↑	++	Effect depends on mixing
Thermophilic aerobic digestion	+++/++	+++/++	+++	++	++	++	Effect depends on the operational regime
Pasteurisation	+++	+++	+++	++	+	++	Must be previously stabilised
Lime stabilisation	+++/++	+++	+++/++	++/+	↑	+++/++	Effect depends on pH maintenance
Thermal drying	+++	+++	+++	+++	+++	+	Total stabilisation and inactivation

Source: Teixeira Pinto (2001)
+ + + : Significant importance ++ : Moderate importance + : Little or no importance
↑ : Increase in volume

Table 5.16. Main final disposal alternatives for the sludge

Alternative	Comment
Ocean disposal	After pre-conditioning, the sewage is disposed in the sea, through ocean outfalls or barges. Disposal without beneficial uses.
Incineration	Thermal decomposition process by oxidation, in which the volatile solids of the sludge are burnt in the presence of oxygen and are converted into carbon dioxide and water. The fixed solids are transformed into ashes. Disposal without beneficial uses.
Sanitary landfill	Disposal of the sludges in ditches or trenches, with compaction and covering with soil, until they are totally filled, after which they are sealed. The sewage sludge can be disposed of in dedicated landfills or co-disposed with urban solid wastes. Disposal without beneficial uses.
Landfarming	Land disposal process, in which the organic substrate is biologically degraded in the upper layer of the soil and the inorganic fraction is transformed or fixed into this layer. Disposal without beneficial uses.
Land reclamation	Disposal of sludge in areas that have been drastically altered, such as mining areas, where the soil does not offer conditions for development and fixation of vegetation, as a result of the lack of organic matter and nutrients.
Agricultural reuse	Disposal of the sludge in agricultural soils, in association with the development of crops. Beneficial use of the sludge (which, in this case, is named as a biosolid).

Source: adapted from de Lara et al (2001)

The potential environmental risks or impacts that are related to the sludge disposal alternatives are presented in Table 5.18. The environmental impacts can be more or less complex depending, amongst others, on: (a) quantity of the sludge disposed; (b) physical, chemical and biological characteristics of the sludge and (c) frequency, duration and extent of disposal.

Table 5.17. Advantages and disadvantages of the main sludge disposal alternatives

Disposal alternative	Advantages	Disadvantages
Ocean disposal	• Low cost	• Ocean water, flora and fauna pollution
Incineration	• Drastic volume reduction • Sterilisation	• High costs • Ash disposal • Atmospheric pollution
Sanitary landfill	• Low cost	• Requirement of large areas • Problems with locations near urban centres • Requirement of special soil characteristics • Gas and leachate production • Difficulty in reintegrating the area after decommissioning
Landfarming	• Low cost • Disposal of large volumes per unit area	• Accumulation of metals and hardly decaying constituents in the soil • Possible groundwater contamination • Odour release and vector attraction • Difficulty in reintegrating the area after decommissioning
Land reclamation	• High application rates • Positive results for the recovery of the soil and flora	• Odours • Composition and use limitations • Contamination of the groundwater, fauna and flora
Agricultural reuse	• Large area availability • Positive effects for the soil • Long term solution • Potential as a fertiliser • Positive outcome for the crops	• Limitations regarding composition and application rates • Contamination of the soil by metals • Food contamination with toxic elements and pathogenic organisms • Odours

Source: Lara et al (2001)

Table 5.18. Environmental impacts related to the different sludge disposal alternatives

Sludge disposal alternative	Potential negative environmental impacts
Ocean disposal	• Water and sediment pollution • Alteration of the marine fauna communities • Disease transmission • Contamination of elements of the food web
Incineration	• Air pollution • Impacts associated with the ash disposal locations
Sanitary landfill • Dedicated • Co-disposal with urban wastes	• Surface and groundwater pollution • Air pollution • Soil pollution • Disease transmission • Aesthetic and social impacts
Landfarming	• Surface and groundwater pollution • Soil pollution • Air pollution • Disease transmission
Land reclamation	• Surface and groundwater pollution • Soil pollution • Odour • Contamination of elements of the food web • Disease transmission
Agricultural reuse	• Surface and groundwater pollution • Soil pollution • Contamination of elements of the food web • Disease transmission • Aesthetic and social impacts

Source: Lara et al (2001)

6

Complementary items in planning studies

6.1 PRELIMINARY STUDIES

The initial phase of a planning or a design corresponds to the *preliminary studies*. These comprise the overall characterisation of the system to be designed, including a quantitative and qualitative evaluation of the wastewater to be treated, the definition of the treatment objectives and a simple technical–economical screening of the various wastewater treatment processes potentially applicable. This stage is highly important, since the selected alternative will be a result of all the considerations and studies completed in this phase. Consequently, efforts should be directed to obtaining data and subsequently drawing the conclusions, always aiming at the highest possible accuracy and reliability, since the technical success and the economic feasibility of the chosen alternative depend largely on this initial analysis.

Preliminary studies are an integral part of the *planning stage* of the design, which comprise the following fundamental elements:

- *Quantitative characterisation of the influent wastewater* (domestic flow, infiltration flow, industrial flow)
- *Qualitative characterisation of the influent wastewater* (domestic sewage, industrial wastewater)
- *Population forecast studies*

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- *Determination of the design horizon and implementation stages*
- *Wastewater treatment objectives and effluent quality requirements*
- *Site selection for the wastewater treatment plant*
- *Initial screening of the treatment alternatives potentially applicable in the situation under analysis*
- *Preliminary design of the most technically promising alternatives* (dimensions of tanks; requirements of power, equipment, resources; flowsheet and plant layout; sludge management)
- *Economical evaluation of the alternatives preliminarily designed*
- *Environmental impact assessment*
- *Selection of the alternative to be adopted and to be subjected to a detailed design, based on the technical and economical analysis*

The population studies and the quantitative and qualitative characterisation of the influent are covered in Chapter 2, while the effluent requirements are discussed in Chapter 3. The selection criteria for the initial screening of alternatives are described in Chapters 4 and 5. Some of the other topics are commented individually in the present chapter. An in depth analysis of these items is not the objective of the chapter, but only to emphasise their importance within the conception and design of the sewage treatment works.

Presented below are short comments on the integration of the points listed above within the preliminary studies phase.

- *Quantification of the polluting loads*. Initially a quantification of the polluting loads needs to be made, based on the quantity and quality of the wastewater. The design population, flows and polluting loads need to be estimated on a yearly (or almost) basis until the end of the design horizon, in order to allow the definition of the staging periods (design stages). (See Chapter 2 for polluting loads and forecasts and current chapter for staging periods studies.)
- *Treatment objectives*. The treatment level and the required removal efficiencies and effluent quality need to be well defined, based on the interaction between the predicted impacts on the water body from the discharge of the effluent and the intended uses for this water body. (See Chapter 3.)
- *Site selection*. Possible sites for the implementation of the treatment plant need to be selected, based on a number of considerations, such as size, geology, topography, ground-water level, flooding level, distance of the intercepting lines, accessibility, neighbouring houses, environmental impact, economics etc. Different sites may be selected, depending on the treatment system to be employed (processes with small or large land requirements). (See relevant sections in this book, related to each treatment process.)
- *Treatment alternatives*. An initial screening of the potentially applicable treatment processes must be undertaken. Based on a global technical analysis, intimately linked to the specificities of the system under analysis, the most promising alternatives are selected for further studies.

- *Process flowsheet*. The flowsheet of each screened alternative to be further analysed should be structured in such a way as to orient the preliminary design stage. The flowsheets should present the main units and flow lines (liquid, sludge, supernatants and recirculations). (See relevant sections in this book, related to each treatment process.)
- *Preliminary design*. The preliminary engineering design is undertaken for the selected alternatives in such a way as to produce data and information to support the subsequent economical analysis. Sub-alternatives may be analysed, such as tank formats, aeration system, sludge treatment options, etc., which may be defined by separate in-parallel comparative technical-economic studies. (See relevant sections in this book, related to each treatment process.)
- *Layout and design of the main units*. Plant layouts, showing in scale the physical arrangement of the units on site should be drawn for the selected alternatives. In order to support the subsequent costs estimates, drawings of the main units should be made, showing the main details that could influence the costs. The preliminary design and the corresponding layouts should be made based on site-specific data, such as topography and geology. (See relevant sections in this book, related to each treatment process.)
- *Economical and financial study*. Based on the characteristics of the main alternatives, a cost estimate for each alternative is undertaken. In many cases it is sufficient to compare only those items which are not common to the options. The economic analysis should take into consideration the construction as well as the operating costs. All of the costs should be brought to the present value, allowing a comparison according to a common basis. (See current chapter.)
- *Environmental impact assessment.* The impacts of the implementation and running of the plant must be taken into account, including the positive impacts resulting from the improvement in the quality of the receiving water body, but also occasional negative impacts associated with the construction, day-to-day operation and occasionally occurring operational problems. Environmental impact assessment (EIA) techniques, such as evaluation matrices, may be employed, and the pertinent legislation must be followed.
- *Selection of the proposed alternative*. The proposed alternative should be the one that offers the greatest advantages from a technical and economical point of view.

6.2 DESIGN HORIZON AND STAGING PERIODS

The selection of an appropriate design horizon and its subdivision into staging periods is an item that affects, not only the economy of the plant in terms of construction and operation, but also its performance. These two concepts may be understood as (Qasim, 1985):

- *Design horizon* or *planning period*: period between the initial year and the final year of the plant operation
- S*taging periods:* the time intervals when plant expansions are made

The design horizon of a sewage treatment works should be relatively short, preferably 20 years or less. The design horizon should still be divided into staging periods, in the order of 7 to 10 years. The larger the population growth rate, the more important is the subdivision into stages, the greater should be the number of staging periods and the lower the duration of each stage. High population growth rates are observed in many urban areas of developing countries. On the other hand, very short stages should be avoided in view of the disturbances associated with the almost continuous coexistence with construction works in the plant.

The preliminary studies should be done considering the whole design horizon, in order to allow the estimation of the full land requirements for the plant. However, the detailed design and the construction of the units should be confined to each implementation stage. Some reasons for this are:

- The division in stages is an economically positive factor, which postpones a considerable part of the investments to the future, thus reducing the present value of the construction costs. The higher the interest rates, the greater are the savings.
- For each new stage the design parameters can be reviewed, especially the flows and the incoming loads, as well as the data obtained with the operational experience of the plant itself.
- Over dimensioned units can generate problems, such as septicity in settling tanks (higher than desired detention time), excessive aeration, etc.
- The staging allows the continuous follow-up of the development of the wastewater treatment technology, allowing more modern solutions to be adopted, which may be, in many cases, the most efficient and economical ones.

Therefore, the design of the plant should foresee flexibility for the integration of the existing or first-stage units with future units.

Example 6.1

Carry out a simplified staging study, based on the forecasts of population, influent flow and influent BOD load from the example in Section 2.2.7, which are presented below:

Year	Served population (inhab)	Average flow (m^3/d)	Average BOD load (kg/d)
0	24,000	3,888	1,325
5	47,000	7,477	2,475
10	53,000	8,409	2,900
15	58,000	9,179	3,150
20	62,000	9,820	3,350

Example 6.1 (*Continued*)

Solution:

The following table of percentage population, flow, and load values can be composed, having as a basis the end of the planning period (considered as 100%).

Year	% of final population	% of final flow	% of final load
0	39	40	40
5	76	76	74
10	85	86	87
15	94	93	94
20	100	100	100

The staging of this plant does not allow good combinations, because the population growth is not so significant during the design years of the second stage. A possible alternative could be:

Item	1st stage	2nd stage
Year of stage implementation	*Year 0*	*Year 5*
Years covered	*Years 0 to 5*	*Years 6 to20*
Duration	*5 years*	*15 years*
% of the implementation	*75%*	*25%*
Number of modules implemented in the stage	3	1
Total number of modules in the plant	3	4

Because the population, flow and load reach in year 5 around 75% of the total final value, the alternative is for the implementation of the first stage comprising 75% of the plant (3 modules, of a total of 4 modules). However, the reach of this first stage is very small, up to year 5 only, when the second stage should enter into operation, until the end of the project (year 20), completing the remaining module (fourth module, in parallel with the others). Although staging is generally advantageous, the benefits in this case are relatively small, because of the need of implementing 75% of the works in the first stage, for only a 5-year period. It is probable that already in year 4 of operation the plant would be in construction works again, associated with the implementation of the units of Stage 2. Therefore, it is unlikely that staging in this plant will be advantageous.

6.3 PRELIMINARY DESIGN OF THE ALTERNATIVES

There is no need for the elaboration of a detailed design for the technical and economical study of alternatives.

The main objective of the preliminary design is to obtain information to support the technical and economical comparison of the alternatives. The drawing of the plant layout, including the main units, is necessary. In addition, the main dimensions of the units, the area occupied, the earth cut and fill volumes, the concrete volumes, the energy to be consumed, the required equipment and other items judged of relevance in the works in question should be known. Such knowledge serves as the basis for the preliminary cost estimates, which can support the economical study.

For the preliminary design, focus is given to *process* calculations, without the need to deepen in detailing the units and in the hydraulic calculations of pipes and interconnections. *The various design examples presented in this book are at the preliminary-design level.*

6.4 ECONOMICAL STUDY OF ALTERNATIVES

The costs for wastewater treatment vary widely with the characteristics of the wastewater, treatment process, climate, design criteria, local conditions and unit local costs for labour, materials, land, and energy.

The cost estimate should comprise the implementation costs (concentrated in time) and the annual operation costs (distributed in time). These costs include (Arceivala, 1981):

- *Implementation costs*
    - construction costs (including equipment and installation)
    - buying or expropriation of the land
    - project costs and supervision, legal taxes
    - interests on the loans during the construction period

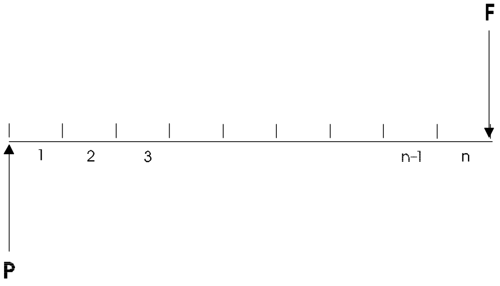
- *Annual costs*
    - interests on the loans
    - annual payment for capital recovery
    - depreciation of the works
    - insurance of the works
    - operation and maintenance (O&M) costs of the works

Of the annual costs, the first four items can be considered as fixed since they have to be included if the plant is working or not. In the preliminary economic studies, the costs for construction and land acquisition (implementation) and operation and maintenance (annual) are usually considered.

The present section does not intend to cover the criteria for the making of cost surveys and economical studies. Some simple Economical Engineering methods are presented, which allow the conversion of implementation and annual costs to a common basis that can be used for the comparison between alternatives.

- Present value of a future investment

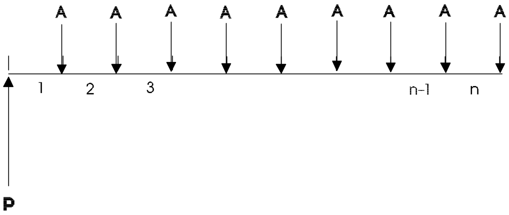
$$\boxed{P = \frac{F}{(i+1)^n}} \tag{6.1}$$



where:
P = present value ($)
F = future value ($)
i = annual interest rate
n = number of years

- Present value of constant annual expenditures

$$\boxed{P = A\frac{(1+i)^n - 1}{i.(i+1)^n}} \tag{6.2}$$



where:
A = annual expenditure ($/year)

Example 6.2

There are two alternatives for the wastewater treatment in a community, each one with different implementation and operation/maintenance costs. The basic characteristics are:

- Planning period: 20 years
- Interest rate: 11% per year

- Alternative A
    - implementation cost (year 0): US$ 3×10^6
    - operation/maintenance cost: US$ 0.5×10^6/year

- Alternative B
    - implementation cost (first stage, year 0): US$ 5×10^6
    - operation/maintenance cost (first stage): US$ 0.2×10^6

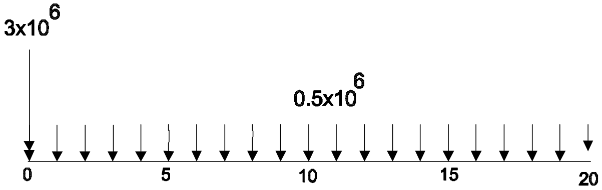
***Example 6.2* (*Continued*)**

- implementation cost (second stage, year 10): US$ 3×10^6
- operation/maintenance cost (second stage): US$ 0.3×10^6

Based on the lowest present value, indicate the best alternative in economical terms.

Solution:

a) Present value of alternative A



- Present value of the annual expenses (Equation 6.2)

$$P = A\frac{(1+i)^n - 1}{i.(i+1)^n} = 0.5 \times 10^6 \cdot \frac{(1+0.11)^{20} - 1}{0.11 \times (1+0.11)^{20}} = 4.0 \times 10^6$$

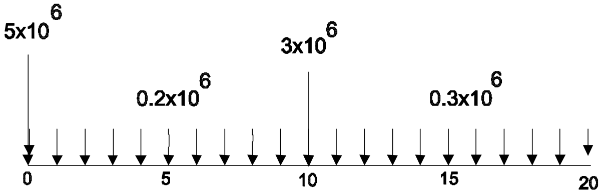
- Total present value

Total present value = Implementation cost + Present value of the annual expenses

Total present value = $3.0 \times 10^6 + 4.0 \times 10^6$

Total present value = US$ 7.0×10^6

b) Present value of alternative B



- Present value of the implementation costs of the second stage (Equation 6.1)

$$P = \frac{F}{(i+1)^n} = \frac{3 \times 10^6}{(1+0.11)^{10}} = 1.2 \times 10^6$$

***Example 6.2* (*Continued*)**

- Present value of the annual expenses of the second stage

Transport the values for year 10 (first year of the second stage) (Equation 6.2):

$$P = A\frac{(1+i)^n - 1}{i.(i+1)^n} = 0.3 \times 10^6.\frac{(1+0.11)^{10} - 1}{0.11 \times (1+0.11)^{10}} = 1.8 \times 10^6$$

Transport the concentrated value of year 10 to the present value (Equation 6.1):

$$P = \frac{F}{(i+1)^n} = \frac{1.8 \times 10^6}{(1+0.11)^{10}} = 0.6 \times 10^6$$

- Present value of the annual expenses of the first stage (Equation 6.2)

$$P = A\frac{(1+i)^n - 1}{i.(i+1)^n} = 0.2 \times 10^6 \cdot \frac{(1+0.11)^{10} - 1}{0.11 \times (1+0.11)^{10}} = 1.2 \times 10^6$$

- Total present value

Total present value = Implementation cost of the 1st stage + Present value of the implementation of the 2nd stage + Present value of the annual expenses of the 1st stage + Present value of the annual expenses of the 2nd stage

Total present value = $5 \times 10^6 + 1.2 \times 10^6 + 1.2 \times 10^6 + 0.6 \times 10^6$
Total present value = US\$ 8.0 $\times$ 10^6

c) Summary of the results

Alternative	Present value
A	US\$ 7.0 $\times 10^6$
B	US\$ 8.0 $\times 10^6$

Based on the lowest present value, alternative A is recommended. In this case, it is more advantageous to have a plant with a smaller implementation cost, even though presenting higher operation and maintenance costs.

The present analysis was made in a simplified manner. Other economical and financial considerations can be included, in order to give support to the study of alternatives.

Part One

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PART TWO

Basic principles of wastewater treatment

7

Microbiology and ecology of wastewater treatment

7.1 INTRODUCTION

Biological wastewater treatment, as the name suggests, occurs entirely by biological mechanisms. These biological processes reproduce, in a certain way, the natural processes that take place in a water body after a wastewater discharge. In a water body, organic matter is converted into inert mineralised products by purely natural mechanisms, characterising the *self-purification* phenomenon (see Chapter 3). In a wastewater treatment plant the same basic phenomena occur, but the difference is that there is the introduction of technology. This technology has the objective of making the *purification* process develop under **controlled conditions** (operational control) and at **higher rates** (more compact solution).

The understanding of the microbiology of sewage treatment is therefore essential for the optimisation of the design and operation of biological treatment systems. In the past, engineers designed the treatment works based essentially on empirical criteria. In the last few decades, the multidisciplinary character of Sanitary and Environmental Engineering has been recognised, and the biologists have brought fundamental contributions for the understanding of the process. The rational knowledge has expanded, together with the decrease in the level of empiricism, allowing the systems to be designed and operated with a more solid base. The result has brought an increase in the efficiency and a reduction in the costs.

The main organisms involved in sewage treatment are bacteria, protozoa, fungi, algae and worms. Their characterisation is presented in Section 7.2. This chapter

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also covers sewage treatment from a biological and ecological (study of the communities involved) points of view. Recognising the great importance of the bacteria in the conversion process of the organic matter, a more detailed description of them is given in the present chapter.

7.2 MICROORGANISMS PRESENT IN WATER AND WASTEWATER

Microbiology is the branch of biology that deals with microorganisms. In terms of water quality, the microorganisms play an essential role, due to their large predominance in certain environments, their action in wastewater purification processes and their association with water borne diseases. Microorganisms can only be observed microscopically.

Some microorganism groups have properties in common with plants whilst others have some animal characteristics. In the past, the classification of living creatures used to be according to the two main kingdoms, **Plants** and **Animals**, and the microorganisms were present in each of these two large subdivisions.

Subsequently, however, biologists have adopted a more practical division, placing microorganisms in the separate kingdoms of the **Monera** (simpler creatures, without a separate nucleus, such as bacteria, cyanobacteria and archaea) and the **Protists** (simple creatures, but with a separate nucleus, such as algae, fungi and protozoa). There are still other possible subdivisions into other kingdoms, but these are not important for the objectives of this book.

The basic difference between the monera/protists and the other organisms (plants and animals) is the high level of cellular differentiation found in the plants and animals. This means that, in monera and protist organisms, the cells of a single individual are morphologically and functionally similar, which reduces its adaptation and development capacity. However, in organisms with cellular differentiation, a functional division occurs. In the higher organisms, the differentiated cells (generally of the same type) combine into larger or smaller groups, called tissue. The tissues constitute the organs (e.g. lungs), and these form the systems (e.g. respiratory system). The level of cellular differentiation is therefore an indication of the developmental level of a species.

Table 7.1 presents the basic characteristics of the kingdoms in the living world, while Table 7.2 lists the main characteristics of the various groups that comprise the monera and protist kingdoms. A short description of the main microorganisms of interest in wastewater was presented in Table 2.15 (Chapter 2).

Protists have the nucleus of the cell confined by a nuclear membrane (algae, protozoa and fungi), being characterised **eukaryotes**. Monera have the nucleus disseminated in the protoplasm (bacteria, cyanobacteria and archaea), being characterised as **prokaryotes**. In general, the eukaryotes present a higher level of internal differentiation and may be unicellular or multicellular. The viruses were not included in the above classification because of their totally specific characteristics. Cyanobacteria were previously called blue-green algae. Archaea are similar to bacteria in size and basic cell components. However, their cell wall, cell material

Table 7.1. Basic characteristics of the kingdoms of the living world

Characteristics	Monera and Protists	Plants	Animals
Cell	Unicellular or multicellular	Multicellular	Multicellular
Cellular differentiation	Non-existent	High	High
Energy source	Light, organic matter or inorganic matter	Light	Organic matter
Chlorophyll	Absent or present	Present	Absent
Movement	Immobile or mobile	Immobile	Mobile
Cell wall	Absent or present	Present	Absent

Table 7.2. Basic characteristics of the main groups of microorganisms (monera and protists)

	Monera (prokaryotes)		Protists (eukaryotes)		
Characteristic	Bacteria	Cyanobacteria	Algae	Protozoa	Fungi
Nuclear membrane	Absent	Absent	Present	Present	Present
Photosynthesis	Minority	Majority	Yes	No	No
Movement	Some	Some	Some	Mobile	Immobile

Source: adapted from La Riviére (1980)

and RNA composition are different. Archaea are important in anaerobic processes (mainly methanogenesis).

7.3 BIOLOGICAL CELLS

Generally, the majority of living cells are very similar. A short description of their main components is presented below (La Riviére, 1980; Tchobanoglous & Schroeder, 1985).

The cells generally have as an external boundary a *cell membrane*. This membrane is flexible and functions as a selective barrier between what is contained inside the cell and the external environment. The membrane is semi-permeable and therefore exerts an important role in selecting the substances that can leave or enter the cell. However, bacteria, algae, fungi, and plants have yet another external layer called *cell wall*. This is generally composed of a rigid material that gives structural form to the cell, even offering protection against mechanical impacts and osmotic alterations. It is believed that this wall is not semi-permeable and therefore does not exert a role in the regulation of the consumption of dissolved substances in the surrounding medium. In some bacteria the cell wall can even be involved by another external layer, generally of a gelatinous material, called *capsule* (with defined limits) or *gelatinous layer* (when diffused). In the case that the individual cells present motility, they usually have *flagella* or *cilia*.

The interior of the cell contains *organelles* and a colloidal suspension of proteins, carbohydrates and other complex forms of organic matter, constituting the *cytoplasm*.

Each cell contains *nucleic acids*, a genetic material vital for reproduction. Ribonucleic acid (RNA) is important for the synthesis of proteins and is found in

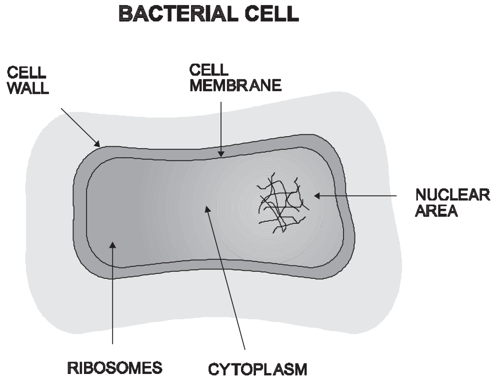


Figure 7.1. Simplified schematic representation of a bacterial cell

the ribosome present in the cytoplasm. The prokaryotic cells, such as those from bacteria, contain only a *nuclear area*, while the eukaryotic cells have a *nucleus* enclosed within a membrane. The nucleus (or nuclear area) is rich in deoxyribonucleic acids (DNA), which contain the genetic information necessary for the reproduction of all the cell components. The cytoplasm of the prokaryotic cells frequently contains DNA in small structures called plasmids.

A simplified schematic representation of a bacterial cell is presented in Figure 7.1.

7.4 ENERGY AND CARBON SOURCES FOR MICROBIAL CELLS

All living creatures need (a) energy (b) carbon and (c) nutrients (nitrogen, phosphorus, sulphur, potassium, calcium, magnesium, etc.) for functions of growth, locomotion, reproduction and others.

In terms of the *carbon source*, there are two fundamental organism types:

- **Autotrophic organisms**. Carbon source: *carbon dioxide* (CO_2);
- **Heterotrophic organisms**. Carbon source: *organic matter.*

In terms of the *energy source*, there are two basic organism types:

- **Phototrophic organisms**. Energy source: *light*;
- **Chemotrophic organisms**. Energy source: *energy from chemical reactions.*

The combinations between these four types are shown in Table 7.3.

In most of the sewage treatment processes (with the exception of facultative and maturation ponds), light does not penetrate significantly in the liquid contained in

Table 7.3. General classification of microorganisms based on sources of energy and carbon

Classification	Energy source	Carbon source	Representative organisms
Photoautotrophs	Light	CO_2	Higher plants, algae, photosynthetic bacteria
Photoheterotrophs	Light	Organic matter	Photosynthetic bacteria
Chemoautotrophs	Inorganic matter	CO_2	Bacteria
Chemoheterotrophs	Organic matter	Organic matter	Bacteria, fungi, protozoa, animals

Source: Tchobanoglous and Schroeder (1985); Metcalf & Eddy (1991)

the biological reactors, due to the high turbidity of the liquid. Because of this, the presence of microorganisms that have light as energy sources (photoautotrophs and photoheterotrophs) is extremely limited. Therefore, the organisms of real importance in this case are the *chemoautotrophs* (responsible, for example, for nitrification) and the *chemoheterotrophs* (responsible for most of the reactions that occur in biological treatment). For simplicity, the latter ones will be subsequently named only *heterotrophs*.

7.5 METABOLISM OF MICROORGANISMS

The chemical processes that simultaneously take place in the cell are jointly called *metabolism*, and can be divided into two categories (La Riviére, 1980):

- *Dissimilation* or *catabolism*: reactions of energy production, in which substrate decomposition occurs;
- *Assimilation* or *anabolism*: reactions that lead to the formation of cellular material (growth), using the energy released in the dissimilation.

In a simplified way, the organisms grow and reproduce themselves at the expense of the energy released in the dissimilation. In dissimilation, the energy stored in chemical form in the organic compounds (substrate) is released and converted in the assimilation in cellular material. The net growth is the result of the balance between the anabolism (positive) and the catabolism (negative).

In both categories, the chemical transformations occur in a sequence of diverse and intricate intermediate reactions, each catalysed by a specific type of enzyme. Most of the enzymes are located inside the cell: these are called intracellular enzymes or *endoenzymes*. However, some enzymes are released in the external medium and are designated as extracellular enzymes or *exoenzymes*. Their importance is associated with the fact that they lead to hydrolysis reactions outside the cell, in the liquid medium, converting large and complex substrate molecules into smaller and simpler molecules, which can then pass through the cell membrane to become available for consumption by the cell.

Table 7.4. Main characteristics of the oxidative and fermentative catabolism

Characteristic	Oxidative catabolism (respiration)	Fermentative catabolism (fermentation)
Electron donor	Organic matter	Oxidised organic matter
Electron acceptor	External: inorganic compound (oxygen, nitrate or sulphate)	Internal: reduced organic matter
Number of final products resulting from the organic matter	One (CO_2)	At least two (CO_2 and CH_4)
Form of carbon in the final product	Oxidised inorganic carbon (CO_2)	Oxidised inorganic carbon (CO_2) + Reduced organic carbon (CH_4)
Oxidation state of carbon in the final product	4+ (CO_2)	4+ (CO_2) and 4− (CH_4)

The removal of the organic matter from sewage occurs through the process of dissimilation or catabolism. The two types of catabolism of interest in sewage treatment are *oxidative catabolism* (oxidation of the organic matter) and *fermentative catabolism* (fermentation of the organic matter) (van Haandel and Lettinga, 1994):

- *Oxidative catabolism*: redox reaction in which an oxidising agent present in the medium (oxygen, nitrate or sulphate) oxidises the organic matter.
- *Fermentative catabolism*: there is no oxidant. The process occurs due to the rearrangement of the electrons in the fermented molecule in such a way that at least two products are formed. Generally, there is the need of various fermentation sequences for the products to be stabilised, that is, be no longer susceptible to fermentation.

The main characteristics of the oxidative and fermentative catabolism are presented in Table 7.4. The concept of electron acceptor is explained in Section 7.6.

7.6 ENERGY GENERATION IN MICROBIAL CELLS

As seen in Section 7.5, the generation of energy in the microbial cells can be accomplished, depending on the microorganism, by means of respiration (oxidative catabolism) or fermentation (fermentative catabolism).

The name respiration is not restricted to the processes that involve oxygen consumption. In general, oxidation implies the loss of one or more electrons from the oxidised substance (in oxidation, the substance gives in negative charges in the form of electrons when passing to a higher oxidation state). The oxidised substance can be the organic matter, as well as reduced inorganic compounds – both are therefore **electron donors**. The electrons taken from the oxidised molecule are transferred through complicated biochemical reactions with the help of enzymes to another inorganic compound (oxidising agent), which receives the generic denomination

Table 7.5. Typical electron acceptors in the oxidation reactions in sewage treatment (listed in decreasing order of energy release)

Conditions	Electron acceptor	Form of the acceptor after the reaction	Process
Aerobic	*Oxygen* (O_2)	H_2O	Aerobic metabolism
Anoxic	*Nitrate* (NO_3^-)	Nitrogen gas (N_2)	Nitrate reduction (denitrification)
Anaerobic	*Sulphate* (SO_4^{2-})	Sulphide (H_2S)	Sulphate reduction
	Carbon dioxide (CO_2)	Methane (CH_4)	Methanogenesis

of **electron acceptor**. As a result, the electron acceptor has its oxidation state reduced.

The main electron acceptors used in respiration are listed in Table 7.5 in a decreasing order of energy release.

When various electron acceptors are available in the medium, the microorganisms use the one that produces the highest quantity of energy. For this reason, the *dissolved oxygen* is used first and, after it is exhausted, the system stops being **aerobic**. If there is *nitrate* available in the liquid medium (which is not always the case), the organisms that are capable of using nitrate in their respiration start to do it, converting the nitrate to nitrogen gas (denitrification). These conditions receive an specific name, being designated as **anoxic** (absence of dissolved oxygen but presence of nitrates). When the nitrates are finished, strict **anaerobic** conditions occur. In these, *sulphates* are used and reduced to sulphides, and *carbon dioxide* is converted into methane. While there are substances with greater energy release, the other ones are not used (Arceivala, 1981).

The methanogenesis stage can occur in two pathways. The first is the oxidative process of the *hydrogenotrophic methanogenesis* (production of methane from hydrogen), in which carbon dioxide acts as an electron acceptor and is reduced to methane. This pathway is less important in terms of the global conversion, but can be made by practically all methanogenic organisms. The second pathway is *acetotrophic methanogenesis* (production of methane from acetate), in which the organic carbon, in the form of acetate (acetic acid) is converted into methane. This pathway is responsible for most of the conversions, although it is accomplished by few bacteria species (Lubberding, 1995).

There are organisms that are functionally adapted to the various respiration conditions. The main ones are:

- **Strict aerobic organisms**: use only *free oxygen* in their respiration
- **Facultative organisms**: use *free oxygen* (preferably) or *nitrate* as electron acceptors
- **Strict anaerobic organisms**: use *sulphate* or *carbon dioxide* as electron acceptors and cannot obtain energy through aerobic respiration

Owing to the release of more energy through the aerobic reactions than through the anaerobic reactions, the aerobic organisms reproduce themselves and stabilise the organic matter faster than the anaerobes. Because of the reproduction rate of the aerobic organisms being greater, the sludge generation is also greater.

The main reactions for the generation of energy that occur in aerobic, anoxic and anaerobic conditions are:

- Aerobic conditions:

$$\boxed{C_6H_{12}O_6 + 6\,O_2 \text{ ------------->} 6\,CO_2 + 6\,H_2O} \tag{7.1}$$

- Anoxic conditions: *nitrate reduction (denitrification)*

$$\boxed{2\,NO_3^{\,-} + 2\,H^+ \text{ ------------->} \ N_2 + 2{,}5\,O_2 + H_2O} \tag{7.2}$$

- Anaerobic conditions: *sulphate reduction*

$$\boxed{CH_3COOH + SO_4^{\,2-} + 2\,H^+ \text{ ------------->} \ H_2S + 2\,H_2O + 2\,CO_2} \tag{7.3}$$

- Anaerobic conditions: *CO_2 reduction (hydrogenotrophic methanogenesis)*

$$\boxed{4\,H_2 + CO_2 \text{ ------------->} \ CH_4 + 2\,H_2O} \tag{7.4}$$

- Anaerobic conditions: *acetotrophic methanogenesis*

$$\boxed{CH_3COOH \text{ ------------->} \ CH_4 + CO_2} \tag{7.5}$$

Figure 7.2 illustrates the main routes of organic matter decomposition in the presence of different electron acceptors.

The sequence of transformations that occurs in sewage treatment is a function of the electron acceptor and the oxidation state of the compounds, measured by its oxidation–reduction potential (expressed in millivolts). Figure 7.3 illustrates these reactions.

The oxidation state of the compound determines the maximum quantity of energy available through it. The more reduced the compound, the more energy it contains. The objective of the energetic metabolism is to conserve as much energy as possible in a form available for a cell. The maximum energy available from the oxidation of a substrate is the difference between its energetic content (given by the oxidation state) and the energetic content of the final products of the reaction (also given by their oxidation state at the end of the reaction) (Grady and Lim, 1980).

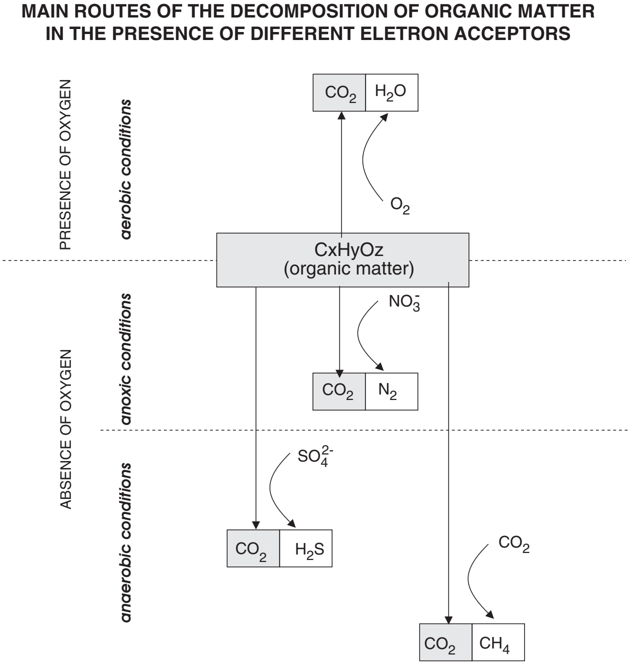


Figure 7.2. Main routes of organic matter decomposition in the presence of different electron acceptors (modified from Lubberding, 1995)

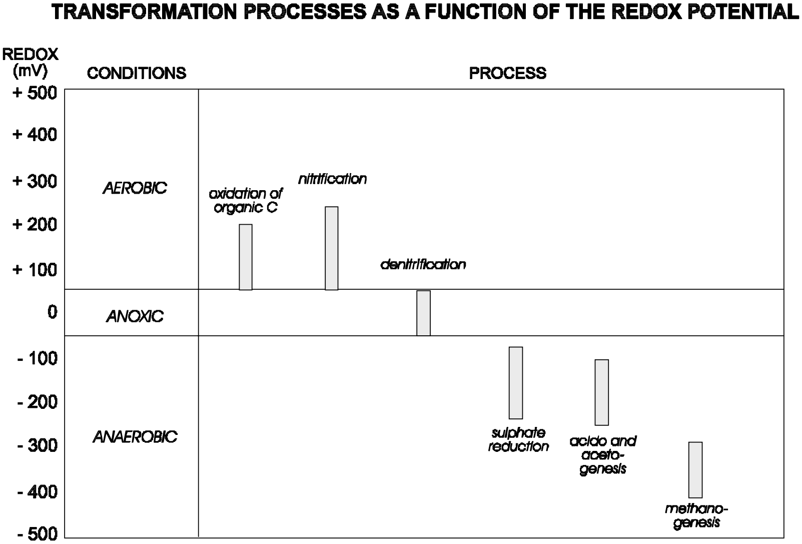


Figure 7.3. Transformation sequence in sewage treatment, as a function of the electron acceptor and the redox potential (adapted from Eckenfelder and Grau, 1992)

The following points apply:

- The greater the oxidation state of the final product, the greater the energy release. The carbon in CO_2 is at its higher state of oxidation. Therefore oxidation reactions that oxidise the carbon in the substrate completely to CO_2 (aerobic respiration) release more energy than the reactions that produce, for example, ethanol (fermentation).
- The lower the oxidation state of the substrate, the greater the energy release. For instance, the oxidation of acetic acid to CO_2 releases less energy than the oxidation of ethanol to CO_2, because the carbon in the acetic acid is at a higher oxidation state than in ethanol.
- CO_2 can never act as an energy source, because its carbon is at the highest possible state of oxidation (CO_2 cannot be oxidised).

7.7 ECOLOGY OF BIOLOGICAL WASTEWATER TREATMENT

7.7.1 Introduction

The role played by microorganisms in sewage treatment depends on the process being used. In facultative ponds, algae have a fundamental function related to the production of oxygen by photosynthesis. The design of ponds is done in such a way as to optimise the presence of algae in the liquid medium and to obtain an adequate balance between bacteria and algae. In anaerobic treatment systems, the conditions are favourable or even exclusive for the development of microorganisms functionally adapted to the absence of oxygen. In this case, the acidogenic and methanogenic organisms are essential.

The microbial mass involved in the aerobic processes consists mainly of bacteria and protozoa. Other organisms, such as fungi and rotifers, can also be found, but their importance is lower. The capacity of fungi to survive in reduced pH ranges and with little nitrogen makes them important in the treatment of certain industrial wastewaters. However, fungi with a filamentous structure can deteriorate the sludge settleability, thus reducing the efficiency of the process. Rotifers are efficient in the consumption of dispersed bacteria and small particles of organic matter. Their presence in the effluent indicates an efficient biological purification process (Metcalf & Eddy, 1991). Generally, it can be said that the species diversity of the various microorganisms in the biomass is low. Table 2.15, in Chapter 2, presents a summary of the main characteristics of the bacteria, protozoa and fungi.

Figure 7.4 presents a sequence of the relative predominance of the main microorganisms involved in aerobic sewage treatment. The ecological interactions in the microbial community cause the increase in the population of a group of microorganisms to be followed by the decline of another group, in view of the selective characteristics of the medium in transformation. Immediately after the introduction of sewage into the biological reactor, the remaining BOD (organic

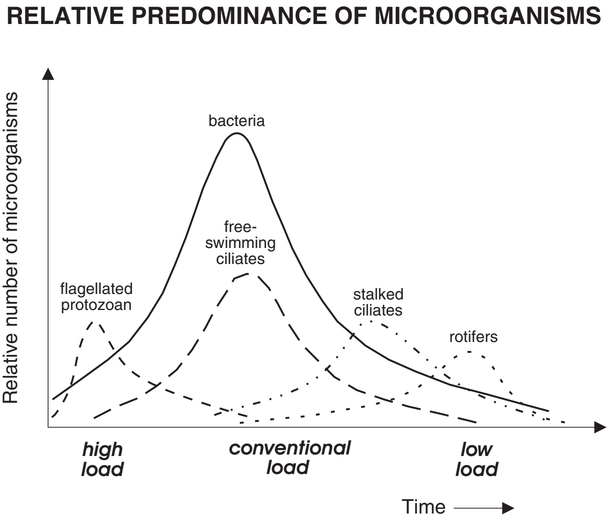


Figure 7.4. Sequence of the relative predominance of the microorganisms in sewage treatment (adapted from König, 1990; Metcalf & Eddy, 1991)

matter) is at its maximum level. The number of bacteria is still reduced, and protozoa of the amoeba type can be found. These are inefficient in the competition for the available food, being found mainly at the start-up of reactors. Due to the great availability of the substrate, the bacterial population grows. The amoebas are then substituted by flagellated protozoa that, due to their motility, are more efficient in the competition for the available food. These flagellated protozoa are characteristic of *high load* systems. With the passing of time and the decrease of the available organic matter, ciliate protozoa substitute the flagellated ones, since the former are capable of surviving with lower food concentrations. This point characterises the operation of *conventional load* systems, where a large number of free-living ciliates are present together with the maximum number of bacteria and a low concentration of organic matter (remaining BOD). In long retention periods, which are characteristic of *low load* systems, the available organic matter is at a minimum and the bacteria are consumed by ciliates and rotifers (König, 1990).

The sections below describe the two main groups involved in the conversion of organic matter: bacteria and protozoa, with greater emphasis being given to the first group.

7.7.2 Bacteria

Bacteria are unicellular prokaryotic (absence of a defined nucleus) microorganisms that live isolated or in colonies. The classification of bacteria according to the shape includes the categories listed in Table 7.6.

Table 7.6. Categories of bacteria according to shape

Name	Shape	Size
Cocci (singular, coccus)	spheroid	0.5 to 3.0 μm in diameter
Bacilli (singular, bacillus)	rod	0.3 to 1.5 μm in width (or diameter) 1.0 to 10.0 μm in length
Spirilla (singular, spirillum)	spiral	<50 μm in length
Vibrios	curved rod	0.6 to 1.0 μm in width (or diameter) 2.0 to 6.0 μm in length
Various	filamentous	>100 μm in length

The bacteria have a more or less rigid cell wall and may or may not present flagella for locomotion. Their reproduction is principally by binary fission, besides the formation of spores and sexual reproduction (minority) (Branco, 1976; Metcalf & Eddy, 1991).

Bacteria constitute the largest and most important group in biological wastewater treatment systems. Considering that the main function of a treatment system is the removal of BOD, the heterotrophic bacteria are the main agents of this mechanism. In addition of playing the role of conversion of the organic matter, the bacteria have the property to agglomerate themselves in structural units such as flocs, biofilms or granules, which have important implications in wastewater treatment (see Section 7.7.4).

Besides the removal of the carbonaceous organic matter, sewage treatment can also incorporate other objectives, which depend on specific groups of bacteria. Thus, the following phenomena can take place:

- *Conversion of ammonia to nitrite (nitrification)*: chemoautotrophic bacteria
- *Conversion of nitrite to nitrate (nitrification)*: chemoautotrophic bacteria
- *Conversion of nitrate to nitrogen gas (denitrification)*: facultative chemoheterotrophic bacteria

The cellular structure of bacteria was presented in Section 7.3 and illustrated in Figure 7.1. Approximately 80% of the bacterial cell is composed of water and 20% of dry matter. Of this dry matter, around 90% is organic and 10% inorganic. Widely used formulas for the characterisation of the approximate cell composition are (Metcalf & Eddy, 1991):

$$\boxed{\begin{array}{ll} C_5H_7O_2N & \text{(without phosphorus in the formula)} \\ C_{60}H_{87}O_{23}N_{12}P & \text{(with phosphorus in the formula)} \end{array}}$$

In any of the two formulations, the C:H:O:N ratio is the same. An important aspect is that all of these components should be obtained from the medium, and the absence of any of them could limit the growth of the bacterial population.

The utilisation by the bacteria of the substrate available in the medium depends on the relative size of the particle. The two main fractions of the organic matter in the wastewater are (a) *easily biodegradable fraction* and (b) *slowly biodegradable fraction*. In a typical domestic sewage, most of the organic matter in soluble form

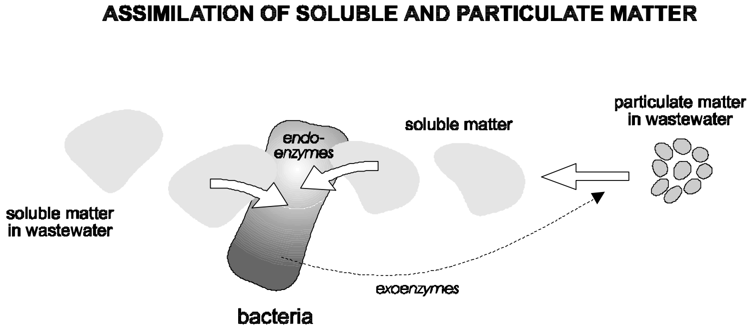


Figure 7.5. Mechanisms of assimilation of soluble matter and conversion of particulate matter into soluble matter

is easily degradable. Owing to their small dimensions, soluble compounds can penetrate the bacterial cell through their cellular membranes. Inside the cell, the soluble organic matter is consumed with the aid of *endoenzymes*. Organic compounds of larger dimensions and more complex formulas (particulate or suspended organic matter), should undergo a transformation process outside the cells, leading to smaller molecules, which can be assimilated by the bacteria. This action is accomplished with the aid of *exoenzymes* in a reaction of *hydrolysis*. In the hydrolysis, energy consumption is not taken into account, and there is no use of electron acceptors. The final product of hydrolysis is present in an easily biodegradable form, penetrating through the cellular membrane inside the cell, where it is consumed in a similar way as the soluble matter (IAWPRC, 1987) (see Figure 7.5).

The environmental requirements for the bacteria vary with the species. For example, bacteria involved in the nitrification process (chemoautotrophic bacteria) are much more sensitive to environmental conditions than the usual heterotrophic bacteria involved in the stabilisation of the carbonaceous organic matter. In general, the optimal growth rate for bacterial growth occurs within relatively limited ranges of temperature and pH, even though their survival can occur within much wider ranges.

Temperatures below the optimal level have a greater influence in the growth rate compared to temperatures above the optimal level. Depending on temperature range, bacteria can be classified as *psycrophilic, mesophilic* or *thermophilic*. The typical temperature ranges for each of these categories are presented in Table 7.7.

pH is also an important factor in bacterial growth. Most bacteria do not support pH values above 9.5 or below 4.0, and the optimal value is around neutrality (6.5 to 7.5) (Metcalf & Eddy, 1991).

7.7.3 Protozoa

Most of the protozoa group comprises unicellular eukaryotic microorganisms without a cell wall. Although they have no cellular differentiation, some have a relatively complex structure with some differentiated regions within the cell for the

Table 7.4. Temperature ranges for optimal bacterial growth

	Temperature (°C)	
Type	Range	Ideal
Psycrophilic	−10 to 30	12 to 18
Mesophilic	20 to 50	25 to 40
Thermophilic	35 to 75	55 to 65

Source: Metcalf & Eddy, 1991

undertaking of different functions. The majority are represented by strictly aerobic or facultative heterotrophic organisms. Their reproduction occurs by binary fission. Protozoa are usually larger than bacteria and can feed on them. This makes the protozoa group an important level in the food web, allowing that larger organisms feed indirectly on the bacteria, which would otherwise be an inaccessible form of food. Depending on some structural characteristics and on the mode of motility, the protozoa can be divided into various groups. Those of principal interest are the following: amoebas, flagellates and free-swimming and stalked ciliates (Branco, 1976; La Riviére, 1980). Some species are pathogenic, as shown on Table 2.15 (Chapter 2).

In terms of the role of protozoa in biological wastewater treatment, the following are of importance:

- Consumption of organic matter;
- Consumption of free bacteria;
- Participation in floc formation.

The last aspect, related to the contribution to the formation of flocs, seems to be a mechanism of lower importance (La Riviére, 1977). The first two aspects (consumption of organic matter and of free bacteria) depend on the feeding mode of the protozoa, which varies with its type, as seen below (Horan, 1990):

- *Flagellates*. Use of soluble organic matter by diffusion or active transport. In this feeding mode, bacteria are more efficient in the competition.
- *Amoebas and ciliates*. Formation of a vacuole around the solid particle (that can include bacteria), through a process called phagocytosis. The organic fraction of the particle is then utilised after an enzymatic action inside the vacuole (inside the cell).
- *Ciliates* (principally). Predation of bacteria, algae and other ciliated and flagellated protozoa.

Although the protozoa contribute to the removal of the organic matter in sewage, their main role in treatment (by processes such as activated sludge) is by the predatory activity that they exert on bacteria freely suspended in the liquid medium (La Riviére, 1977). Hence, bacteria that are not part of the floc, but are dispersed in the medium are not normally removed in the final sedimentation. As a result, they contribute to the deterioration of the final effluent in terms of suspended solids, organic matter (from the bacteria themselves) and even pathogens. Therefore, the

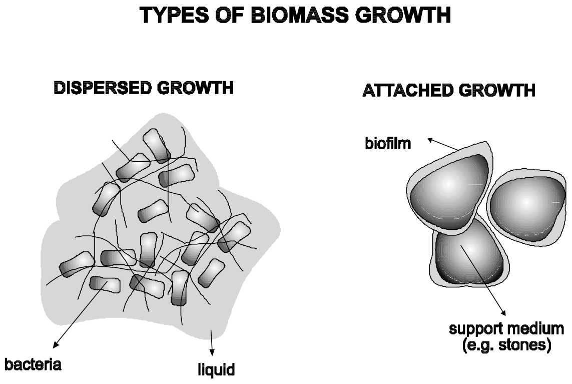


Figure 7.6. Typical examples of biomass growth

action of protozoa on bacteria contributes to improving the quality of the final effluent (Horan, 1990).

The free-swimming ciliates have greater food requirements than the stalked ciliates, because a large part of their energy is spent in locomotion. The predominance of the stalked ciliates occurs after the decline of the free-swimming ciliates population, when they can feed on the bacteria available in the floc.

7.7.4 Suspended and attached biomass growth

With relation to the structural formation of the biomass, biological sewage treatment processes can be divided into the basic configurations listed below (see Figure 7.6). The list is organised according to the prevailing mechanism, although mechanisms of attached and dispersed growth can occur simultaneously.

- **Dispersed growth**: the biomass grows in a dispersed form in the liquid medium, without any supporting structure
  Systems:
  - stabilisation ponds and variants
  - activated sludge and variants
  - upflow anaerobic sludge blanket reactors (receiving wastewaters containing suspended solids)
- **Attached growth**: the biomass grows attached to a support medium, forming a biofilm. The support medium can be immersed in the liquid medium or receive continuous or intermittent liquid discharges. The support medium can be a solid natural (stones, sand, soil) or artificial (plastic) material or consist of an agglomerate of the biomass itself (granules).
  Systems with a solid support for attachment:
  - trickling filters
  - rotating biological contactors

- submerged aerated biofilters
- anaerobic filters
- land disposal systems

Systems with the support for attachment consisting of the agglomerated biomass.

- upflow anaerobic sludge blanket reactors (receiving predominantly soluble sewage)

Even though the principles of biological treatment are the same for both biomass support systems, the treatment kinetics are influenced by the intervenience of specific aspects. The greater theoretical development in terms of modelling is with respect to aerobic treatment with dispersed growth. This results from the facts that there have been for many years a larger number of researches directed to the activated sludge process and that the formulation of dispersed-growth models is, in a way, simpler than for attached-growth systems.

7.7.5 The biological floc in dispersed-growth systems

In some treatment processes, such as activated sludge, the organisms concentrate and form a broader structural unit that is called a *floc*. Although microorganisms are the agents in BOD removal, the floc in activated sludge plays an essential role in the organic matter removal process. It is not only the property of the heterotrophic organisms in stabilising organic matter that makes the activated sludge process efficient. Also of fundamental importance is the property shown by the main microorganisms to organise themselves in the structural unit of a floc, which is capable of being separated from the liquid by the simple physical mechanism of sedimentation, in separate settling units. This separation allows the final effluent to be clarified (with reduced concentrations of suspended organic matter). The quality of the final effluent is therefore characterised by low values of soluble BOD (removed in the reactor) and of suspended BOD (flocs removed in the final settling unit). The flocculation mechanism, which in water treatment is reached at the expense of adding chemical products, occurs by entirely natural mechanisms in biological sewage treatment.

The floc represents a heterogeneous structure that contains the adsorbed organic matter, inert material from the sewage, microbial material produced, and alive and dead cells. The size of the floc is regulated by the balance between the forces of cohesion and shear stress caused by the artificial aeration and agitation (La Riviére, 1977). Among the microorganisms that constitute the floc, besides the bacteria and protozoa, fungi, rotifers, nematodes and occasionally even insect larvae can be found (Branco, 1978).

The floc matrix can rapidly absorb up to 40% of the soluble and particulate BOD entering the biological reactor through ionic interactions. The particulate material is hydrolysed by exoenzymes before it is absorbed and metabolised by the bacteria. Considering that the size of a floc varies between 50 and 500 μm, there will be a marked gradient of BOD and oxygen concentrations from the external

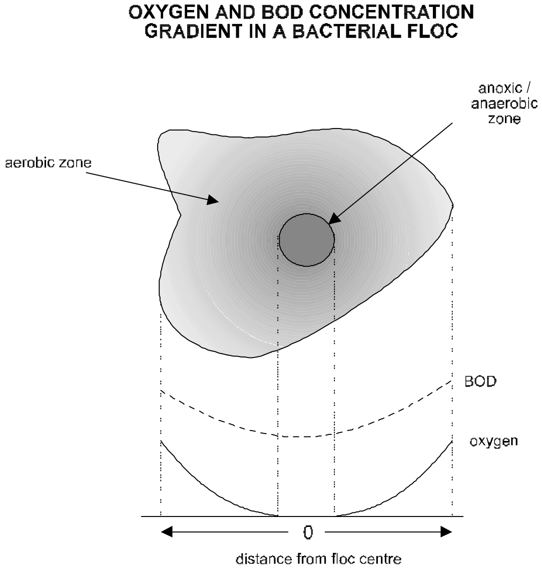


Figure 7.7. BOD and oxygen gradients along a typical floc (adapted from Horan, 1990).

border of the floc (larger values) to the centre (where very low BOD values and zero DO values can be found). Consequently, in the direction of the centre of the floc, the bacteria become deprived of nutrient sources, what reduces their viability (Horan, 1990). When analysing the availability of oxygen or nutrients in the liquid medium, their possible absence inside the floc must be taken into consideration. This supports the fact that, for instance, in many cases anoxic conditions may be assumed, even though a small concentration of DO (0.5 mg/L) in the liquid medium may still be found. Although the liquid medium is not deprived of oxygen, a large part of the floc is. In this situation, the anoxic conditions are prevalent in the interior of the floc, and the biochemical reactions take place as if in the absence of oxygen. Figure 7.7 illustrates the BOD and oxygen gradients along the floc.

The conditions that lead to the microbial growth in the form of flocs instead of cells freely suspended in the liquid medium are not fully known. A plausible hypothesis for the structure of the floc is that the filamentous bacteria exert the function of a structural matrix, to which the floc-forming bacteria adhere. It is believed that the attachment occurs through exopolysaccharides, present in the form of a capsule or gelatinous layer. In the past this phenomena was attributed only to *Zoogloea ramigera,* but there are indications that the production of the gelatinous layer occurs through various genera, including *Pseudomonas*. The continuous production of these exopolymers results in the adherence of other microorganisms and colloidal particles and, as a consequence, the floc size increases. Finally, the protozoa adhere and colonise the floc, and there is some evidence that they also

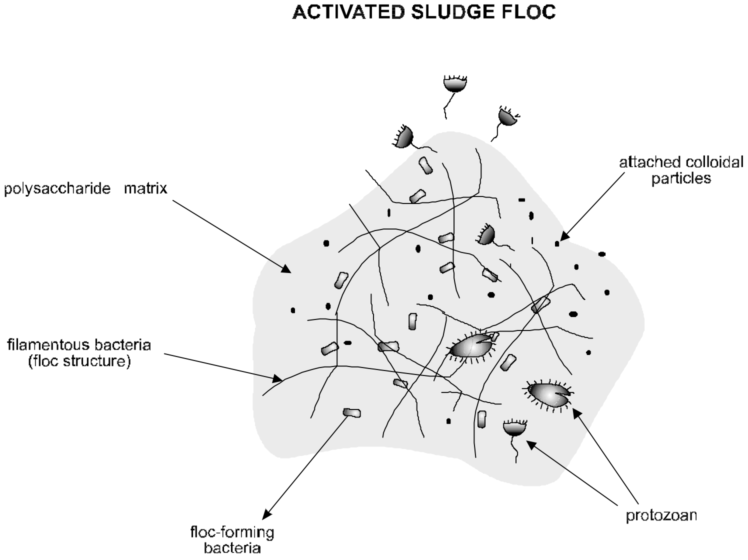


Figure 7.8. Typical structure of an activated sludge floc (adapted from Horan, 1990).

excrete a viscous mucus that contributes to the cohesion of the floc (Horan, 1990). Figure 7.8 shows an schematics of a typical structure of an activated sludge floc.

The balance between the filamentous and floc-forming organisms is delicate, and a good part of the operational success of an activated sludge plant depends on it (see also Chapter 39). Three basic conditions can occur (Horan, 1990):

- *Equilibrium between filamentous and floc-forming organisms*. Good settleability and thickening properties of the sludge.
- *Predominance of floc-forming organisms.* There is insufficient rigidity in the floc, generating a small and weak floc, with poor settleability. This results in the so-called pin-point floc.
- *Predominance of filamentous organisms.* The filaments extend themselves outside the floc, impeding the adherence of other flocs. Therefore, after sedimentation, the flocs occupy an excessively large volume, which can bring problems in the operation of the secondary sedimentation tank, causing a deterioration in the quality of the final effluent. Such a condition is called sludge bulking.

7.7.6 Biofilm in attached growth systems

Immobilisation is the attachment of the microorganisms to a solid or suspended supporting medium. The immobilisation has the advantage of enabling a high biomass concentration to be retained in the reactor for long time periods. Although practically all the microorganisms have the potential to adhere to a supporting

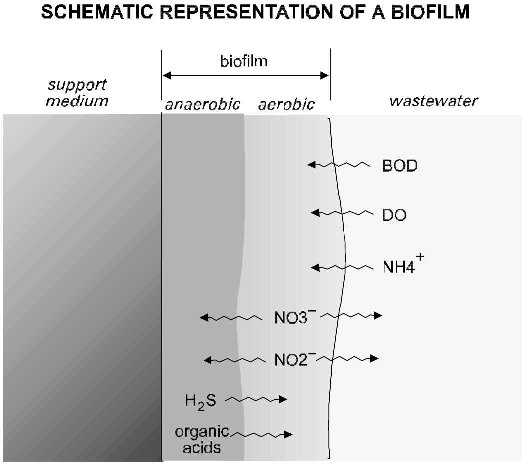


Figure 7.9. Schematic representation of a biofilm (adapted from Iwai and Kitao, 1994)

medium through the production of extracellular polymers that allow physical–chemical attachment, it is only recently that the technological application of cellular sorption processes is being employed in a wider and optimised scale in various biotechnological processes and in sewage treatment (Lubberding, 1995).

The attachment is influenced by cell-to-cell interactions, by the presence of polymer molecules on the surface and by the composition of the medium (Rouxhet and Mozes, 1990).

In the biofilm, the compounds necessary for bacterial development, such as organic matter, oxygen and micronutrients, are adsorbed onto the surface. After adhering, they are transported through the biofilm through diffusion mechanisms, where they are metabolised by the microorganisms. Colloidal or suspended solids cannot diffuse through the biofilm and need to be hydrolysed to smaller molecules. The final metabolic products are transported in the opposite direction, to the liquid phase (Iwai and Kitao, 1994). Figure 7.9 illustrates the operating principle of a biofilm in sewage treatment.

In an aerobic reactor, oxygen is consumed as it penetrates the biofilm, until anoxic or anaerobic conditions are reached. Therefore, an external layer with oxygen and an internal layer deprived of oxygen may be found. DO is the determining factor in the establishment of the layers. Nitrate reduction will occur in the anoxic layer. In anaerobic conditions, there will be the formation of organic acids and a reduction of sulphates. This coexistence between aerobic, anoxic and anaerobic conditions is an important characteristic of biofilm systems (Iwai and Kitao, 1994).

The biofilm formation process can be understood as occurring at three stages (Iwai and Kitao, 1994). Table 7.5 and Figure 7.10 present the main characteristics of these three stages associated with the thickness of the biofilm.

Table 7.5. Stages in the formation of the biofilm

Biofilm thickness	Characteristics
Thin	• The film is thin and frequently does not cover all the surface of the support medium • The bacterial growth follows a logarithmic rate • All the microorganisms grow under the same conditions, and the growth is similar to that of the dispersed biomass
Intermediate	• The thickness of the film becomes greater • The bacterial growth rate becomes constant • The thickness of the active layer stays unaltered, independently of the increase in the total thickness of the biofilm • If the supply of organic matter is limited, the microorganisms adopt a metabolism sufficient only for maintenance, but without growth • If the supply of organic matter is lower that the requirements for maintenance, the film thickness decreases
High	• The thickness of the biofilm reaches a very high level • The microbial growth is counteracted by the decay of the organisms, by the uptake by other organisms and by shearing stress • Parts of the biofilm can be dislodged from the support medium • If the biofilm continues to grow without being dislodged from the support medium, clogging will take place

Source: based on Iwai and Kitao (1994)

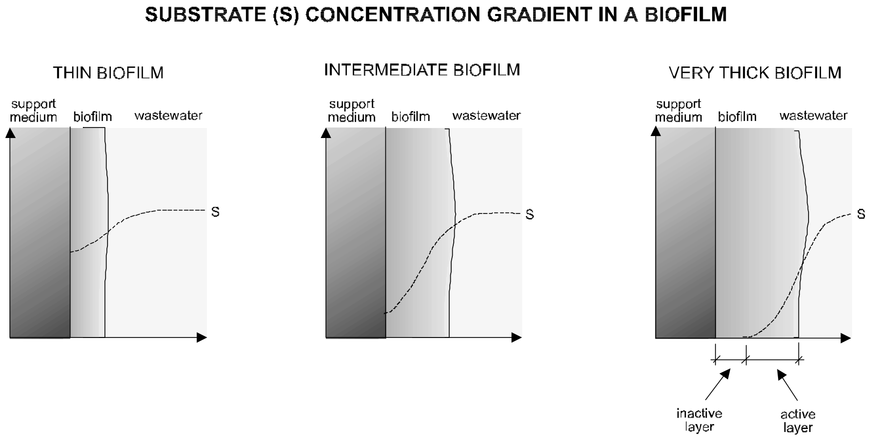


Figure 7.10. Concentration gradients of the substrate (S) in biofilms of different thicknesses (adapted from Lubberding, 1995)

When analysing *dispersed growth* and *attached growth* in sewage treatment, the comparison between the hydraulic detention time and the cell doubling time is an aspect of great importance. In dispersed growth systems, in order to have microbial population growth, the hydraulic detention time (average time that a water molecule stays in the system) has to be greater than the doubling time of the microorganisms, that is, the time necessary to generate new cells. If the hydraulic detention time is less than the cell doubling time, the bacteria are *"washed"* out of the system. This is a limiting factor for sizing the biological reactors, considering that reactor volume and detention time are directly related (detention time = volume/flow).

In the case of systems with biofilms, the hydraulic detention times can be less than the cell doubling time, without cell wash-out occurring, because of the fact that the bacteria are attached to a support medium. Consequently, it is possible to adopt lower volumes for the reactor.

In the comparison between dispersed-growth and attached-growth systems, there are the following aspects relative to attached-growth systems (Iwai and Kitao, 1994; Lubberding, 1995):

- The reactor can be operated with a hydraulic detention time lower than the cell doubling time.
- The concentration of *active* biomass can be higher than for dispersed-growth systems (see explanation below).
- The substrate removal rate can be higher than for dispersed-growth systems (see explanation below).
- The coexistence between aerobic and anaerobic microorganisms is greater than in the dispersed-growth systems because the thickness of the biofilm is usually greater than the diameter of the biological floc.
- The cells are fixed in the solid phase, while the substrate is in the liquid phase. This separation reduces the need or the requirements for the subsequent clarification stage.
- The microorganisms are continually reused. In the dispersed-growth system, reutilisation can only be implemented through recirculation of the biomass.
- If the biofilm thickness is high, there can be limitations for the diffusion of the substrate into the biofilm.

The potential difference between the activity of the dispersed and attached biomass and the consequent substrate removal rate can be explained as follows (Lubberding, 1995). The **dispersed biomass** has a density close to the sewage and moves itself in practically the same direction and velocity of the sewage inside the reactor. As a result, the biomass stays exposed to the same fraction of liquid for a larger period, leading to a low substrate concentration in the neighbourhood of the cell. With the low substrate concentrations, the bacterial activity and the substrate removal rate are also lower. Only at a certain distance from the cell is the substrate concentration higher. Considering the dependence between the substrate concentration and the microbial activity, the importance represented by the mixing level in the reactor becomes evident.

In the **attached biomass** systems, the density of the support medium together with the biomass is very different from the density of the liquid in the reactor, allowing the occurrence of velocity gradients between the liquid and the external layer of the biofilm. As a result, the cells are continually exposed to new substrates, potentially increasing their activity. However, if the biofilm thickness is very high, the substrate consumption along the biofilm could be such, that the internal layers have substrate deficiencies, which reduce their activity. In these conditions, the attachment with the support medium reduces and the biomass can be dislodged from the support medium.

8

Reaction kinetics and reactor hydraulics

8.1 INTRODUCTION

All biological wastewater treatment processes occur in a volume defined by specific physical boundaries. This volume is commonly called a *reactor*. The modifications in the composition and concentration of the constituents during the residence time of the wastewater in the reactor are essential items in sewage treatment. These changes are caused by:

- *hydraulic transportation of the materials in the reactor (input and output);*
- *reactions that occur in the reactor (production and consumption).*

The knowledge of the two components, which characterise the so-called *mass balance* around the reactor, is fundamental in the design and operation of wastewater treatment plants. Finally, the manner and efficiency in which these changes take place depend on the type and configuration of the reactor, which is dealt with in the study called *reactor hydraulics.*

The present chapter covers the following main topics:

- reaction kinetics
- mass balance
- reactor hydraulics

Biological Wastewater Treatment in Warm Climate Regions by Marcos von Sperling and Carlos Augusto de Lemos Chernicharo. ISBN: 1 84339 002 7. Published by IWA Publishing, London, UK.

8.2 REACTION KINETICS

8.2.1 Reaction types

Most reactions that take place in sewage treatment are slow and the consideration of their kinetics is important. The *reaction rate* **r** is the term used to represent the disappearance or formation of a constituent or chemical species. The relation between the *reaction rate*, *concentration of the reagent* and the *order of reaction* is given by the expression:

$$r = kC^n \tag{8.1}$$

where:

r = reaction rate ($ML^{-3}T^{-1}$)
k = reaction constant (T^{-1})
C = reagent concentration (ML^{-3})
n = reaction order

For different values of n, there are the following types of reactions:

- $n = 0$ *zero-order reaction*
- $n = 1$ *first-order reaction*
- $n = 2$ *second-order reaction*

When there is more than one reagent involved, the calculation of the reaction rate must take into consideration their concentrations. If there are two chemicals with concentrations A and B, the rate is:

$$r = kA^n B^m \tag{8.2}$$

The global reaction rate is defined as $(m + n)$. For example, if a global reaction rate was determined as being $r = kA^2B$, the reaction is considered second order with relation to reagent A and first order with relation to reagent B. The global reaction rate is of third order (Tchobanoglous and Schroeder, 1985).

If the logarithm is applied on both sides of Equation 8.1 for a reaction with only one reagent, the following equation is obtained:

$$\log r = \log k + n \log C \tag{8.3}$$

The visualisation of the above relation for different values of n is presented in Figure 8.1. The interpretation of Figure 8.1 is:

- The *zero-order reaction* results in a horizontal line. The reaction rate is independent of the reagent concentration, that is, it is the same independently of the reagent concentration.
- The *first-order reaction* has a reaction rate directly proportional to the reagent concentration.

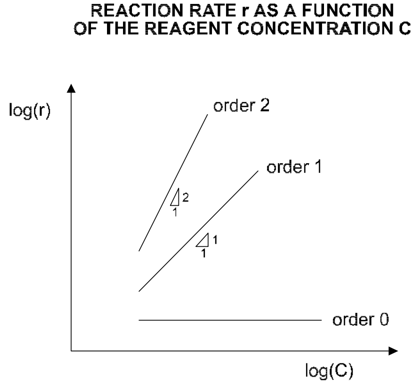


Figure 8.1. Determination of the reaction order on a logarithmic scale (adapted from Benefield and Randall, 1980)

- The *second-order reaction* has a reaction rate proportional to the square of the reagent concentration.

The most frequent reaction orders found in sewage treatment are zero order and first order. Second order reactions may occur with some specific industrial wastewaters. The reaction order does not necessarily need to be an integer, and the laboratory determination of the decomposition rates of certain industrial constituents can lead to intermediate orders. Besides these reactions with constant order, there is another type of reaction, which is widely used in the area of wastewater treatment, called *saturation reaction*. In summary, the following reactions are going to be analysed in detail:

- *zero-order reaction*
- *first-order reaction*
- *saturation reaction.*

8.2.2 Zero-order reactions

Zero-order reactions are those in which the *reaction rate is independent of the reagent concentration.* In these conditions, the *rate of change of the reagent concentration (C) is **constant***. This comment assumes that the reaction occurs in a batch reactor (see Item 8.4), in which there is no addition or withdrawal of the reagent during the reaction. In the case of a reagent that is disappearing in the reactor (for example, through decomposition mechanisms), the rate of change is given by Equation 8.4. The *minus* sign in the term on the right-hand side of the equation indicates *removal* of the reagent, whereas a *plus* sign would indicate *production* of the constituent.

$$\frac{dC}{dt} = -K.C^0 \tag{8.4}$$

or

$$\boxed{\frac{dC}{dt} = -K} \tag{8.5}$$

The development of the rate of change (dC/dt) with time according to Equation 8.5 can be seen in Figure 8.2.a. It is seen that the rate is constant with time.

The integration of Equation 8.5 with $C = C_o$ at $t = 0$ leads to:

$$\boxed{C = C_o - K.t} \tag{8.6}$$

This equation can be visualised in Figure 8.2.b.

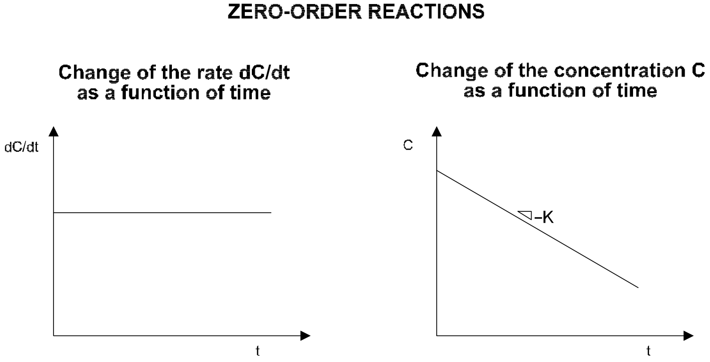


Figure 8.2. Zero-order reactions. (a) Change of the reaction rate dC/dt with time. (b) Change of the concentration C with time.

8.2.3 First-order reactions

First-order reactions are those in which the *reaction rate is proportional to the concentration of the reagent.* Therefore, in a batch reactor, the rate of change of the reagent concentration C is proportional to the reagent concentration at a given time. Assuming a reaction in which the constituent is being removed, the associated equation is:

$$\frac{dC}{dt} = -K.C^1 \tag{8.7}$$

or

$$\boxed{\frac{dC}{dt} = -K.C} \tag{8.8}$$

The development of the rate of change (dC/dt) with time according to Equation 8.8 is presented in Figure 8.3.a. It is noted that the rate decreases linearly with time.

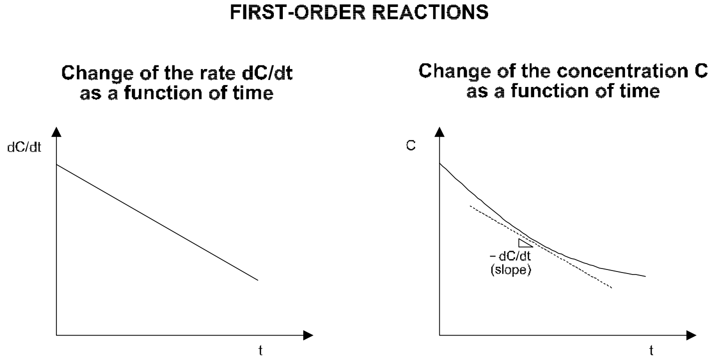
Integrating Equation 8.8 with $C = C_o$ at $t = 0$ leads to:

$$\boxed{\mathrm{In}\ C = \mathrm{In}\ C_o - K.t} \tag{8.9}$$

or

$$\boxed{C = C_o.e^{-Kt}} \tag{8.10}$$

Equation 8.10 is plotted in Figure 8.3.b.


FIRST-ORDER REACTIONS

Change of the rate dC/dt as a function of time

Change of the concentration C as a function of time

Figure 8.3. First-order reactions. (a) Change of the reaction rate dC/dt with time. (b) Change of the concentration C with time.

Various reactions in sewage treatment follow first-order kinetics. The introduction of oxygen by artificial aeration is an example. Other examples are the removal of organic matter in some systems and the decay of pathogenic organisms. The biological stabilisation of the organic matter may be represented by pseudo-first-order reaction, as covered in Section 8.2.4. Although various components are involved, such as oxygen concentration, number of microorganisms and concentration of the organic matter, the rate can be proportional to the concentration of one constituent (organic matter, in this case), provided the others are in relative abundance (Arceivala, 1981). However, if the organic matter is available in low concentrations, the reaction occurs as a first-order reaction. This aspect is discussed in Section 8.2.4.

The global rate follows first-order kinetics in various complex processes. Many substances can individually show zero-order kinetics, but the complex substrates in which many of these substances are aggregated (e.g. domestic and industrial wastewater) can suffer a decay rate that follows a first-order reaction. Initially, when most of the components are being simultaneously removed (consumed), the global removal rate is high. After a certain time, however, the rate can be slower, when only the most hardly degradable constituents are still present. In this case, the global reaction rate may resemble a typical first-order reaction (Arceivala, 1981).

8.2.4 Saturation reactions

Another expression used to describe the rates involved in biological sewage treatment is based on enzymatic reactions, whose kinetics were proposed by Michaelis and Menten. Since bacterial decomposition involves a series of reactions catalysed by enzymes, the structure of the Michaelis–Menten expression can be used to describe the kinetics of bacterial growth and the decomposition reactions in sewage (Sawyer and McCarty, 1978) (see Chapter 9).

The reaction rate follows a hyperbolic form, in which the rate tends to a saturation value (Equation 8.11):

$$\boxed{r = r_{max} \cdot \frac{S}{K_s + S}} \tag{8.11}$$

where

r = reaction rate ($ML^{-3}T^{-1}$)
r_{max} = maximum reaction rate ($ML^{-3}T^{-1}$)
S = concentration of the limiting substrate (ML^{-3})
K_s = half-saturation constant (ML^{-3})

Through Equation 8.11, it is seen that K_s is the concentration of the substrate in which the reaction rate r is equal to $r_{max}/2$. Equation 8.11 is illustrated in Figure 8.4.

Equation 8.11 is widely used in biological sewage treatment. Its large importance resides in its form that can approximately represent zero-order and first-order

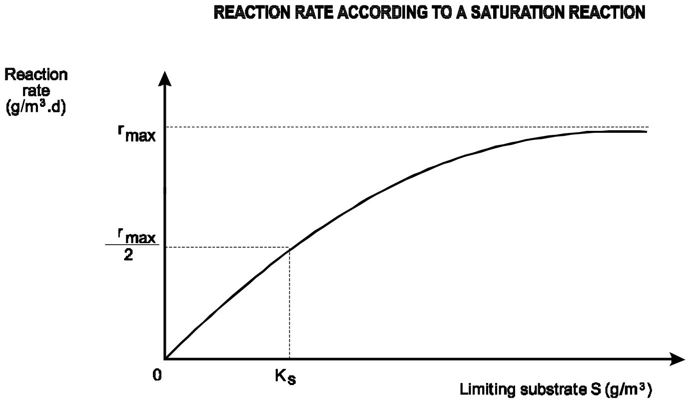


Figure 8.4. Graphical representation of the saturation reaction, according to Michaelis–Menten

kinetics (see Figure 8.5), as well as the transition between them. As mentioned before, at the start of a reaction of decomposition of substrate (organic matter), when the concentration is still high, there is no substrate limitation in the medium and the global removal rate approaches zero-order kinetic. When the substrate starts to be consumed, the reaction rate starts to decrease, characterising a transition region or mixed order. When the substrate concentration is very low, the reaction rate starts to be limited by the low availability of organic matter in the medium. In these conditions, the kinetics develops as in a first-order reaction. These situations occur as a function of the relative values of S and K_s, as described below.

- *Relative substrate concentration:* ***high***

$$\boxed{S >> K_s \text{ : approximately zero-order reaction}}$$

When the substrate concentration is much higher than the value of K_s, K_s can be neglected in the denominator of Equation 8.11. The equation is thus reduced to:

$$r = r_{max} \tag{8.12}$$

In these conditions, the reaction rate r is constant and equal to the maximum rate r_{max}. The reaction follows zero-order kinetics, in which the reaction rate is independent of the substrate concentration. In domestic sewage treatment, such a situation tends to occur, for instance, at the head of a plug-flow reactor, where the substrate concentration is still high.

- *Relative substrate concentration:* ***low***

$$\boxed{S << K_s \text{ : approximately low-order reaction}}$$

When the substrate concentration is much lower than the value of K_s, S can be ignored in the denominator of Equation 8.11. Consequently, the equation is reduced to:

$$r = r_{max} \cdot \frac{S}{K_S} \tag{8.13}$$

Since r_{max} and K_s are two constants, the term (r_{max}/K_s) is also a constant, and can be substituted by a new constant K. Thus, Equation 8.13 is reduced to:

$$r = K.S \tag{8.14}$$

In this situation, the reaction rate is proportional to the substrate concentration. The reaction follows *first-order kinetics*. Such a situation is typical in domestic sewage treatment, in complete-mix reactors, where the substrate

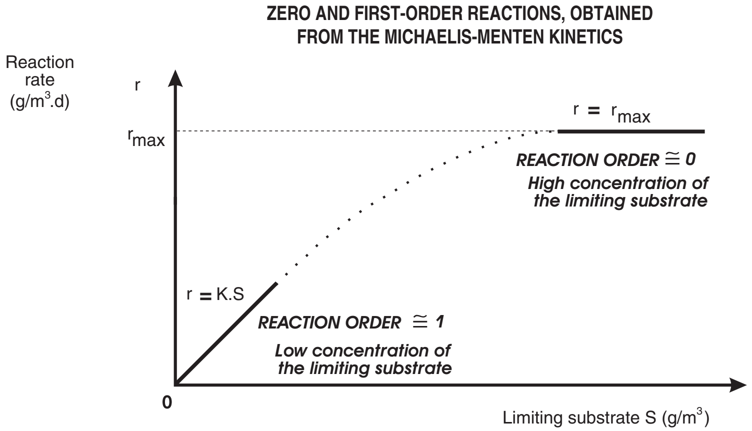


Figure 8.5. Michaelis–Menten kinetics. Two extreme conditions in the saturation reaction

concentration in the medium is low, due to the requirements of having low substrate levels in the effluent.

Figure 8.5 presents the two extreme situations, representatives of zero and first order-kinetics.

The form of the Michaelis–Menten equations is reanalysed in detail in Chapter 9 that treats the kinetics of bacterial growth and substrate removal. In these conditions, one has the so-called Monod kinetics. In Chapter 9, typical values of K_s for the treatment of domestic sewage are presented, and the conditions that lead to the predominance of the zero or first-order kinetics are re-evaluated.

8.2.5 Influence of the temperature

The rate of any chemical reaction increases with temperature, provided that this increase in temperature does not produce alterations in the reagents or in the catalyst. The biological reactions, within certain ranges, also present the same tendency to increase with temperature. However, there is an ideal temperature for the biological reactions, above which the rate decreases, possibly due to the destruction of enzymes at the higher temperatures (Sawyer and McCarty, 1978; Benefield and Randall, 1980).

A usual form to estimate the variation of the reaction rate as a function of temperature is through the formulation based on the van't Hoff-Arrhenius theory, which can be expressed as:

$$\boxed{\frac{K_{T_2}}{K_{T_1}} = \theta^{T_2 - T_1}} \tag{8.15}$$

where:

K_{T_2} = reaction coefficient for temperature 2
K_{T_1} = reaction coefficient for temperature 1
θ = temperature coefficient

Even thought θ is frequently treated as a constant, it can vary substantially, even inside a reduced temperature range (Sawyer and McCarty, 1978). The values of θ usually adopted for the corrections of various reactions involved in sewage treatment are presented in the relevant chapters of this book.

8.3 MASS BALANCE

8.3.1 Representative equations

Once the reaction rates of interest are known, their influence on the general mass balance of the constituent under analysis must be quantitatively evaluated. This is because the concentration of a certain constituent in a reactor (or in any place inside it) is a function, not only of the biochemical reactions, but also of the transport mechanisms (input and output) of the constituent. *Reactor* is the name given to tanks or generic volumes in which the chemical or biological reactions occur.

The mass balance is a quantitative description of all the materials that enter, leave and accumulate in a system with defined physical boundaries. The mass balance is based on the law of conservation of mass, that is, mass is neither created nor destroyed. The basic mass balance expression should be derived in a chosen volume, which can be either a tank or a reactor as a whole, or any volume element of them. In the mass balance, there are terms for (Tchobanoglous and Schroeder, 1985):

- *materials that enter*
- *materials that leave*
- *materials that are generated*
- *materials that are consumed*
- *materials that are accumulated in the selected volume*

In any selected volume (see Figure 8.6), the quantity of accumulated material must be equal to the quantity of material that enters, minus the quantity that leaves, plus the quantity that is generated, minus the quantity that is consumed. In linguistic terms, the mass balance can be expressed in the following general form.

$$\boxed{\text{Accumulation} = \text{Input} - \text{Output} + \text{Production} - \text{Consumption}} \qquad (8.16)$$

Some authors prefer not to include in an explicit form the term relative to consumption, which must be expressed as a produced material, with a minus sign in the reaction rate. The convention adopted in this text is the one of Equation 8.16, which leads to a clearer understanding of the four main components involved in

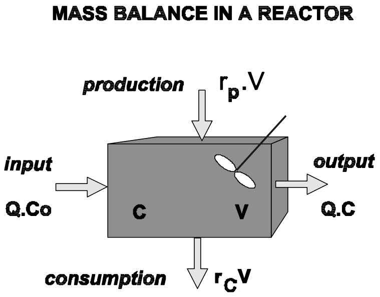


Figure 8.6. Mass balance in a reactor

the mass balance. Therefore, care and coherence must be exercised with the signs of each term when adopting one convention or another.

Mathematically the relation of Equation 8.16 can be expressed as:

$$\frac{d\,(C.V)}{dt} = Q.C_0 - Q.C + r_p.V - r_c.V \qquad (8.17)$$

where

C = concentration of the constituent at a time t (ML^{-3})
C_o = influent concentration of the constituent (ML^{-3})
V = volume of the reactor (completely mixed) or volume element of any reactor (L^3)
Q = flow (L^3T^{-1})
t = time (T)
r_p = reaction rate of production of the constituent ($ML^{-3}T^{-1}$)
r_c = reaction rate of consumption of the constituent consumed ($ML^{-3}T^{-1}$)

Equation 8.17 can be expressed in the following alternative form, in which the left-hand term has been expanded:

$$C.\frac{dV}{dt} + V.\frac{dC}{dt} = Q.C_0 - Q.C + r_p.V - r_c.V \qquad (8.18)$$

The volume in biological reactors can usually be considered as fixed (dV/dt=0), making the first term on the left-hand side disappear. This leads to the simplified and more usual form of the mass balance, presented in Equation 8.19. Since in this equation the only dimension is time, the formulation is of a *ordinary differential equation*, in which the analytical solution (or numeric computation) is much simpler. However, it must be emphasised that the mass balance in other systems, such as, for instance, the sludge volume in secondary sedimentation tanks in activated sludge systems, also implies variations in volume (besides concentration variations). In this particular case, there are two dimensions (time and space), which lead to a *partial differential equation*. The solution of these equations demands a greater mathematical sophistication. However, for completely-mixed biological reactors (fixed volume), the more usual mass balance, expressed in Equation 8.19, is used.

$$\boxed{V.\frac{dC}{dt} = Q.C_0 - Q.C + r_p.V - r_c.V} \tag{8.19}$$

In the preparation of a mass balance, the following steps must be followed (Tchobanoglous and Schroeder, 1985):

- Prepare a simplified schematic or flowsheet of the system or process for which the mass balance will be prepared.
- Draw the system boundaries, to define where the mass balance will be applied.
- List all the relevant data that will be used in the preparation of the mass balance in the schematic or flowsheet.
- List all the chemical or biological reaction equations that are judged to represent the process.
- Select a convenient basis on which the numerical calculations will be done.

8.3.2 Steady state and dynamic state

The mathematical model of the system can be structured for two distinct conditions:

- *Steady state*
- *Dynamic state*

The **steady state** is the one in which there are no accumulations of the constituent in the system (or in the volume being analysed). Thus, dC/dt = 0, that is*, the concentration of the constituent is constant*. In the steady state, the input and output flows and concentrations are constant. There is a perfect equilibrium between the positive and the negative terms in the mass balance, which, when summed, lead to a zero value. In the *design* of wastewater treatment plants, it is more usual to use the simplified steady state equations. Under these conditions, in which $dC/dt = 0$, the mass balance is given by Equation 8.20.

$$0 = Q.C_0 - Q.C + r_p.V - r_c.V \tag{8.20}$$

The **dynamic state** is the one in which there are mass accumulations of the constituent in the system. Hence, $dC/dt \neq 0$. *The concentration of the constituent in the system is therefore variable with time* and can increase or decrease, depending on the balance between the positive and negative terms. Usually in a treatment plant, the input flow and/or the input concentration are variable, besides the possibility

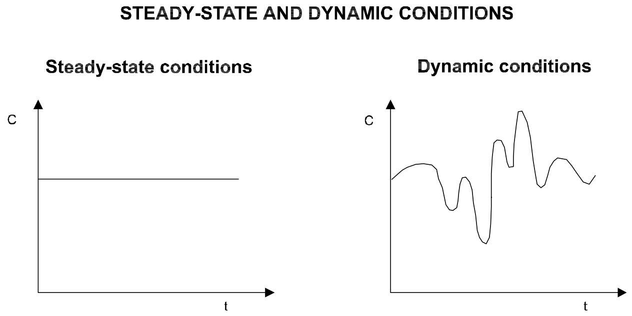


Figure 8.7. Steady and dynamic conditions. Profile of the concentration of the constituent with time.

of having other external stimulus to the system (temperature changes) that cause a *transient* in the concentration of the constituent. For this reason, dynamic conditions are the ones really prevailing in actual sewage treatment plants. The dynamic models are based on the generalised mass balance equation (Equation 8.19). For the *operational control* of a treatment plant, dynamic models are more adequate, due to the frequent variation of the external and internal conditions of the system. The dynamic models can be also used for design, principally for evaluating the impact of variable influent loads on the performance of the plant. The dynamic models have been less used due to the larger complexity involved in the solution of the equations and the greater requirements of values for model coefficients and variables. However, the trend of using more computers and numerical integration routines commercially available has contributed to a greater use of dynamic models. It must be emphasised that the steady state is only a particular case of the dynamic state.

Figure 8.7 illustrates the concept of the steady and dynamic states, through the representation of the variation of the concentration of the constituent with time.

8.4 REACTOR HYDRAULICS

8.4.1 Introduction

After the reaction rates are known (Section 8.2) and the mass balances have been established (Section 8.3), in order to calculate the concentration of the constituent in the reactor it is necessary to define the hydraulic model to be attributed to it.

The hydraulic model of the reactor is a function of the type of flow and the mixing pattern in the unit. The mixing pattern depends on the physical geometry of the reactor, the quantity of energy introduced per unit volume, the size or scale of the unit and other factors.

In terms of flow, there are the following two conditions:

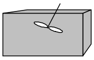
- *Intermittent flow* (batch): discontinuous input and/or output
- *Continuous flow*: continuous input and output

In terms of the mixing pattern, there are two basic idealised hydraulic models, which define an envelope inside which the other patterns are found. These are the **plug-flow** and the **complete-mix** reactors, which lead to the following main alternatives:

- *plug flow reactor*
- *complete-mix reactor*
- *dispersed flow*
- *reactors in series and/or in parallel.*

The main types of reactors used in sewage treatment are presented in Table 8.1 (Tchobanoglous and Schroeder, 1985; Metcalf & Eddy, 1991), and their operational characteristics are summarised in Table 8.2.

Table 8.1. Characteristics of the most frequently used reactors and hydraulic models in sewage treatment

Hydraulic model	Schematics	Characteristics
Batch reactors		A batch reactor is the one in which there is no flow entering or leaving. The reactor contents are completely mixed. All elements are exposed to the treatment for a time equal to the substrate residence time in the reactor. Consequently, the batch reactor behaves like a discrete volume of a plug-flow reactor. The BOD test bottle is an example of a batch reactor. One of the variants of the activated sludge process is the sequencing batch reactors.
Plug flow		The fluid particles enter the tank continuously in one extremity, pass through the reactor, and are then discharged at the other end, in the same sequence in which they entered the reactor. The fluid particles move as a piston, without any longitudinal mixing. The particles maintain their identity and stay in the tank for a period equal to the theoretical hydraulic detention time. This type of flow is reproduced in long tanks with a large length-to-breadth ratio, in which longitudinal dispersion is minimal. These reactors are also called tubular reactors. Plug-flow reactors are idealised reactors, since complete absence of longitudinal dispersion is difficult to obtain in practice.

(*Continued*)

Table 8.1 (*Continued*)

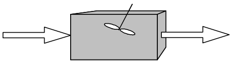
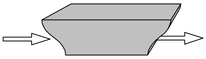
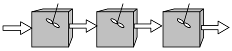
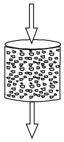
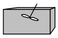
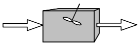
Hydraulic model	Schematics	Characteristics
Complete mix		The particles that enter the tank are immediately dispersed in all the reactor body. The input and output flows are continuous. The fluid particles leave the tank in proportion to their statistical population. Complete mix can be obtained in circular or square tanks in which the tank's contents are continuously and uniformly distributed. Complete-mix reactors are also known as CSTR or CFSTR (continuous-flow stirred tank reactors). Complete-mix reactors are idealised reactors, since total and identical dispersion is difficult to obtain in practice.
Dispersed flow		Dispersed or arbitrary flow is obtained in any reactor with an intermediate degree of mixing between the two idealised extremes of plug flow and complete mix. In reality, most reactors present dispersed-flow conditions. However, due to the greater difficulty in their modelling, the flow pattern is frequently represented by one of the two idealised hydraulic models. The input and output flows are continuous.
Complete-mix reactors in series		Complete-mix reactors in series are used to model the hydraulic regime that exists between the idealised plug flow and complete mix regimes. If the series is composed of only one reactor, the system reproduces a complete-mix reactor. If the system has an infinite number of reactors in series, plug flow is reproduced. Input and output flows are continuous. Reactors in series are also commonly found in maturation ponds.
Packed-bed reactors		These reactors are filled with some type of packing medium, such as stone, plastic, ceramic and others. Regarding the flow and saturation, these reactors can be submerged, with the pores saturated (anaerobic filters and submerged aerated filters) or with intermittent dosing, with non-saturated pores (trickling filters). The flow can be upward or downward.

Table 8.2. Operational characteristics of the main reactor systems (assuming steady-state conditions)

Reactor type	Schematics	Continuous flow	Variation of the composition with time (in a given position in the reactor)	Variation of the composition with the position in the reactor (at a given time)	Number of equivalent complete mix reactors	Typical length / breadth ratio
Batch reactor		No	Yes	No	–	≈1
Plug flow		Yes	No	Yes	∞	>>1
Complete mix		Yes	No	No	1	≈1

8.4.2 Ideal plug-flow reactor

The ideal plug flow is the one in which each fluid element leaves the tank in the same order of entrance. No element anticipates or delays another in the journey. The flow occurs as pistons moving from upstream to downstream, without mixing between the pistons and without dispersion. Consequently, each element is exposed to treatment for the same period of time (as in a batch reactor), which is equal to the theoretical hydraulic detention time (Arceivala, 1981).

Figure 8.8 presents a summary of the concentration profiles with time and position in an ideal plug-flow reactor submitted to constant influent flow and concentration (steady-state conditions). If the influent load is varied (dynamic conditions), the derivation of the formulas for the plug-flow reactor is more complicated than for complete mix. This is because the concentration in the plug flow varies with time and space in the reactor, while in complete mix the variation is only with time (same concentration in any position of the reactor). That is why complete-mix reactors in series are frequently used to simulate a plug-flow reactor under dynamic (time-varying) conditions.

If the influent (input) concentration is constant, the effluent concentration (output) also remains constant with time. The concentration profile in the tank and, therefore, the effluent concentration, depend on the type and reaction rate of the constituent. Table 8.3 summarises the main intervening equations.

The following generalisations can be made for an ideal plug-flow reactor under steady state conditions:

- *Conservative substances*: the effluent concentration is equal to the influent concentration.
- *Biodegradable substances with zero-order reaction*: the removal rate is constant from the inlet to the outlet end of the reactor.
- *Biodegradable substances with first-order reaction*: along the reactor, the substrate removal coefficient (K) is constant, but the concentration decreases gradually while the wastewater flows along the reactor. At the inlet end of the reactor, the concentration is high, which causes the removal rate to be also high (in first-order reactions the removal rate is proportional to

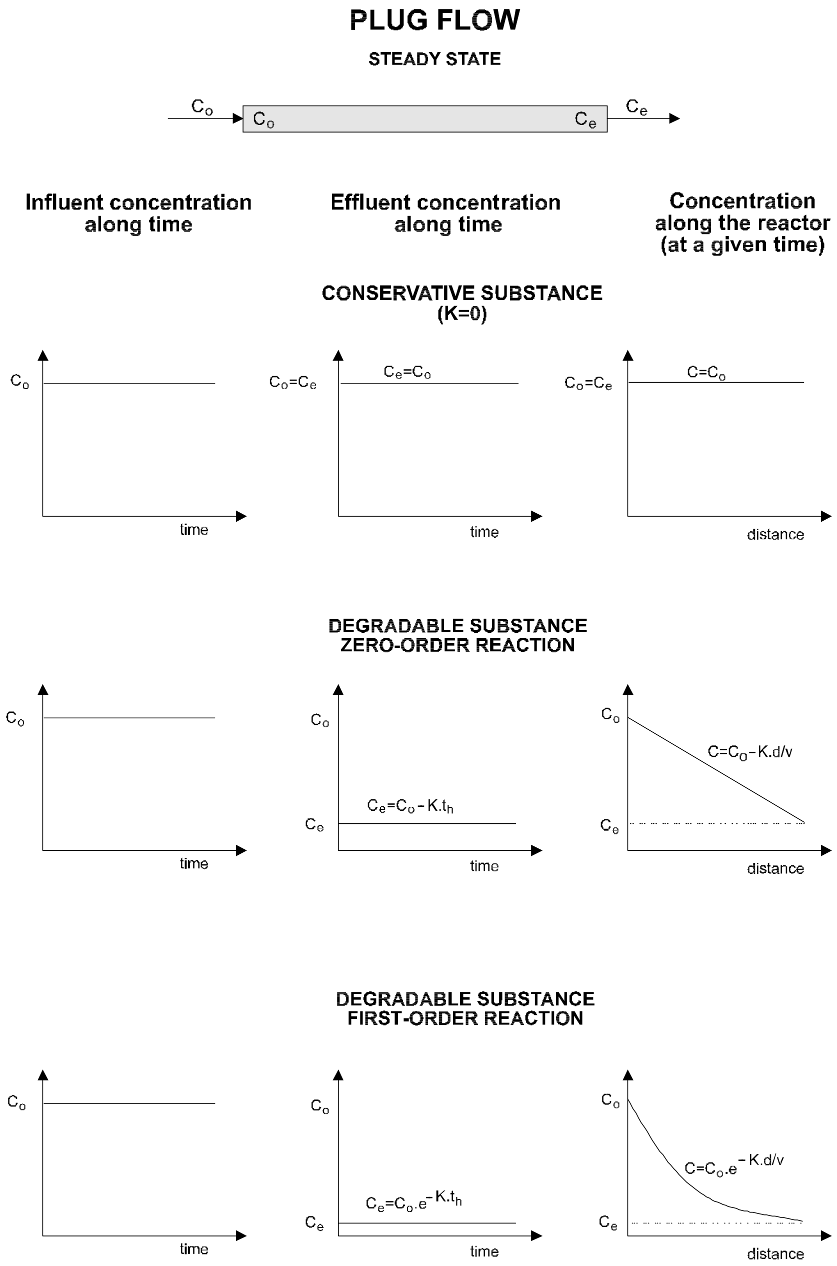


Figure 8.8. Concentration profiles – Ideal plug-flow reactor under steady-state conditions. Nomenclature: C = concentration at a given time; C_o = influent concentration; C_e = effluent concentration; K = reaction coefficient; t_h = hydraulic detention time; d = distance (length of the reactor); v = horizontal velocity. In this figure, time represents the operational time, and not the travel time along the reactor.

Table 8.3. Ideal plug-flow reactor. Steady-state conditions. Equations for the calculation of the concentration along the tank and the effluent concentration

Reaction	Concentration along the reactor (at a given time)	Effluent concentration
Conservative substance ($r_c = 0$)	$C = C_0$	$C_e = C_o$
Biodegradable substance (zero order reaction; $r_c = K$)	$C = C_o - K.d/v$	$C_e = C_o - K.t_h$
Biodegradable substance (first order reaction; $r_c = K.C$)	$C = C_o.e^{-K.d/v}$	$C_e = C_o.e^{-K.t_h}$

C = concentration at a given point in the reactor (g/m^3)
C_o = influent concentration (g/m^3)
K = reaction coefficient (d^{-1})
d = distance along the tank (m)
v = horizontal velocity (m/d)
t_h = hydraulic detention time (= volume/flow) (d)

the concentration). At the outlet end of the reactor, the concentration is reduced and, consequently, the removal rate is low, that is, more time is required to reduce a unit value of the concentration.

- *First-order or higher reactions*: the plug flow is more efficient than the complete-mix reactor.

Example 8.1

A reactor with predominantly longitudinal dimensions has a volume of 3,000 m^3. The influent has the following characteristics: flow = 600 m^3/d; substrate concentration = 200 g/m^3.

Calculate the concentration profile along the reactor (assuming an ideal plug-flow reactor under steady state) in the following conditions:

- conservative substance (K = 0)
- biodegradable substance with first-order removal ($K = 0.40\ d^{-1}$)

Solution:

a) Hydraulic detention time

The hydraulic detention time (t_h) is given by:

$$t_h = \frac{V}{Q} = \frac{3000\ m^3}{600\ m^3/d} = 5\ d$$

The travel distance is proportional to the time spent for the piston to flow. The total distance is covered when the hydraulic detention time is reached.

Example 8.1 (Continued)

b) Conservative substance

The application of the formula $C = C_o e^{-K.t}$ for steady state, with $K = 0$, for various values of t, leads to:

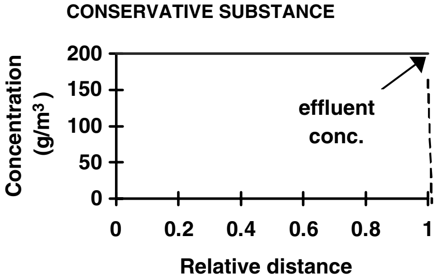
$$C = 200.e^{-0xt}$$

Travel time (d)	Distance / total length	Concentration along the tank (g/m^3)
0	0.0	200
1	0.2	200
2	0.4	200
3	0.6	200
4	0.8	200
5	1.0	200

The same values can be obtained through the direct application of the formula $C = C_o$ (Table 8.3) for the conservative substances.

The effluent concentration is the concentration at the end of the hydraulic detention time ($t_h = 5$ d), that is, 200 g/m^3. The same value can be obtained through the direct application of the formula $C_e = C_o$ (Table 8.3).

The profile of the concentration along the tank is plotted below.



c) Biodegradable substance (with a first-order reaction)

The application of the formula $C = C_o e^{-Kt}$ (steady state) for various values of t leads to:

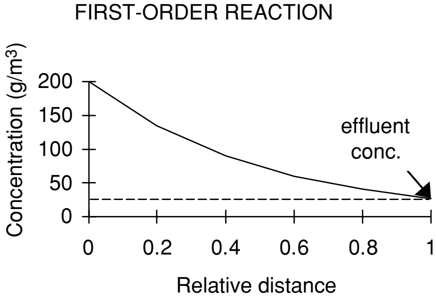
$$C = 200.e^{-0.40xt}$$

***Example 8.1* (*Continued*)**

Travel time (d)	Distance / total length	Concentration along the tank (g/m^3)
0	0.0	200
1	0.2	134
2	0.4	90
3	0.6	60
4	0.8	41
5	1.0	27

The effluent concentration is the concentration at the end of the hydraulic detention time ($t_h = 5$ d), that is, 27 g/m^3. The same value can be obtained through the direct application of the formula $C_e = C_0 e^{-K.th}$ (Table 8.3) for first-order reactions.

The concentration profile along the tank is plotted below.



8.4.3 Ideal complete-mix reactor

The reactor with continuous flow and ideal complete mixing conditions is the one in which all of the elements that enter the reactor are instantaneously and totally dispersed. Thus, the reactor contents are homogeneous, that is, the concentration of any component is the same at any point in the tank. As a result, the effluent concentration is the same as that at any point in the reactor.

The mass balance in the reactor is (see Equations 8.16 and 8.19):

$$\text{Accumulation} = \text{Input} - \text{Output} + \text{Production} - \text{Consumption} \tag{8.21}$$

$$\boxed{V.\frac{dC}{dt} = Q.C_0 - Q.C + r_{p.}V - r_{c.}V} \tag{8.22}$$

Table 8.4. Ideal complete-mix reactor. Steady-state conditions. Equations for the calculation of the concentration along the tank and the effluent concentration

Reaction	Concentration along the reactor (at a given time)	Effluent concentration
Conservative substance ($r_c = 0$)	$C = C_o$	$C_e = C_o$
Biodegradable substance (zero order reaction; $r_c = K$)	$C = C_o - K.t_h$	$C_e = C_o - K.t_h$
Biodegradable substance (first order reaction; $r_c = K.C$)	$C = C_o/(1 + K.t_h)$	$C_e = C_o/(1 + K.t_h)$

C = concentration at a given point in the reactor (g/m^3)
C_o = influent concentration (g/m^3)
K = reaction coefficient (d^{-1})
d = distance along the tank (m)
t_h = hydraulic detention time (= volume/flow) (d)

In the steady state there is no mass accumulation in the reactor, that is, $dC/dt = 0$. In this analysis there is no production of constituents, only consumption reactions. Therefore, $r_p = 0$. Dividing the remaining terms by Q, and knowing that $t = V/Q$, the following equation is obtained:

$$0 = C_0 - C - r_c.t \tag{8.23}$$

With the rearrangement of Equation 8.23, concentration profiles along the complete-mix reactor and the effluent concentration under steady-state conditions can be calculated (Figure 8.9).

If the influent (input) concentration is constant, the effluent (output) concentration also remains constant with time. The effluent concentration depends on the type and reaction rate of the constituent. However, the concentration profile along the reactor depicts a constant concentration, which is in agreement with the assumption that in a complete-mix reactor the concentrations are the same at any point in the tank. Table 8.4 summarises the main equations.

In comparison with the plug-flow reactor, the effluent concentration is only different for reactions of first order (or higher). For such reaction orders, the complete-mix reactor is less efficient than the plug-flow reactor.

The following generalisations can be made for an ideal complete-mix reactor under steady-state conditions:

- *Conservative and biodegradable substances*: the concentration and the removal rate are the same at any point in the reactor. The effluent concentration is equal to the concentration at any point in the reactor.
- *Conservative substances*: the effluent concentration is equal to the influent concentration.
- *Biodegradable substances with zero-order reaction*: the effluent concentration is equal to the effluent concentrations of a plug-flow reactor with the same detention time (the removal rate is independent of the local substrate concentration).

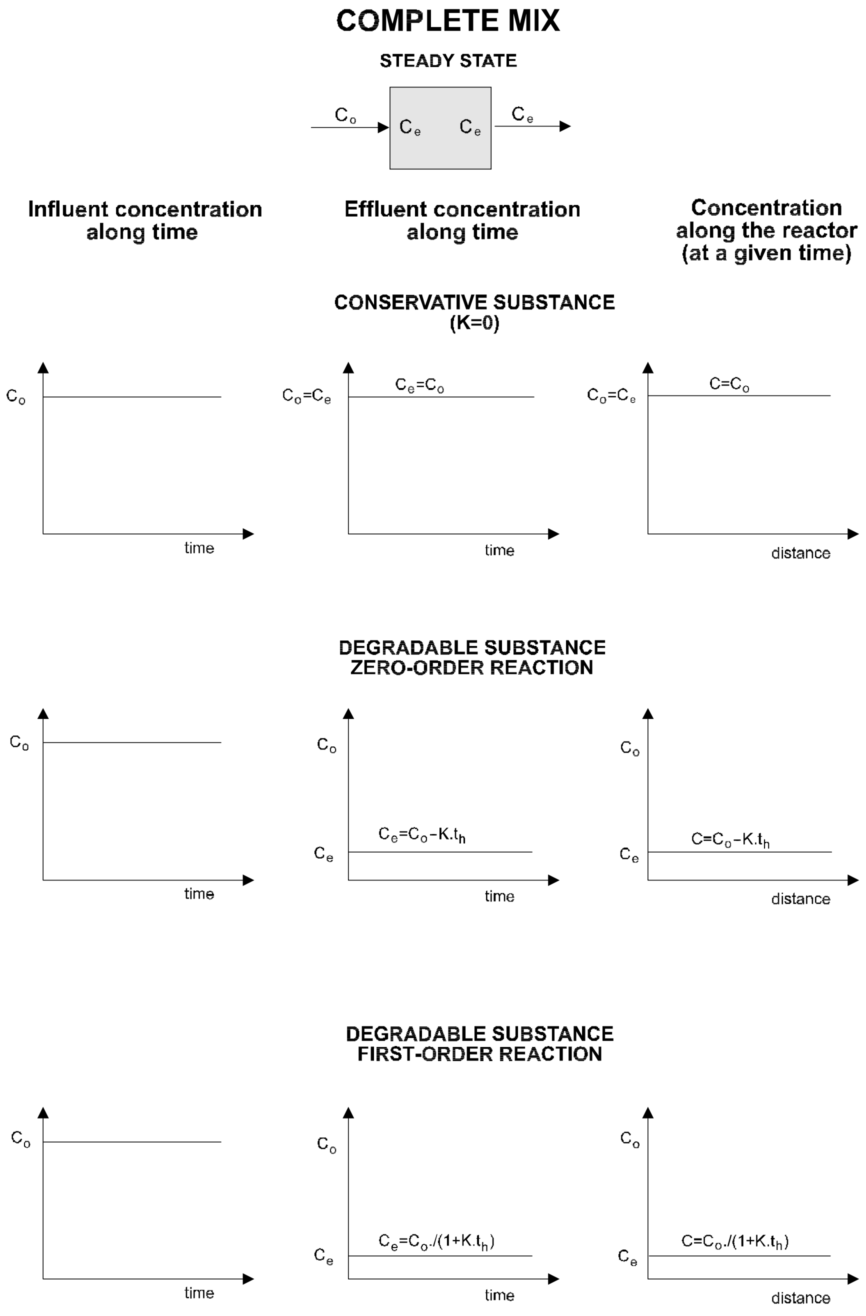


Figure 8.9. Concentration profiles – Ideal complete mix reactor under steady-state conditions. Nomenclature: C = concentration at a given time; C_o = influent concentration; C_e = effluent concentration; K = reaction coefficient; t_h = hydraulic detention time. In this figure, time represents the operational time, and not the travel time along the reactor.

- *Biodegradable substances with first-order reactions or higher*: the complete-mix reactor is less efficient than the plug-flow reactor. Considering (a) that the removal rate is a function of the local concentration in first or higher-order reactions and (b) that the concentration at a complete-mix reactor is lower than the *average* concentration along a plug-flow reactor, then the efficiency of the complete-mix reactor is lower than that of the plug-flow reactor.

Example 8.2

A reactor of an approximately square shape and good mixing conditions has the same volume as the reactor in Example 8.1 (3,000 m^3). The influent also has the same characteristics of the referred example (flow = 600 m^3/d; influent substrate concentration = 200 g/m^3).

Calculate the concentration profile along the reactor (assuming an ideal complete-mix reactor under steady state) in the following conditions:

- Conservative substance ($K = 0$)
- Biodegradable substance with first-order removal ($K = 0.40\ d^{-1}$)

Solution:

a) Hydraulic detention time

The hydraulic detention time is the same calculated in Example 8.1 that is, $t_h = 5$ days.

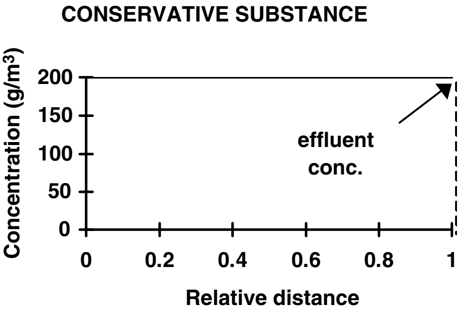
b) Conservative substance

In a complete-mix reactor, the concentration is the same at any point. For a conservative substance, $C = C_o$ (Table 8.4). Hence, for any distance, the concentration is:

$$C = 200\ g/m^3$$

The effluent concentration is also equal to 200 g/m^3. This value is equal to that calculated for the ideal plug-flow reactor.

The concentration profile along the tank is plotted below.



***Example 8.2* (*Continued*)**

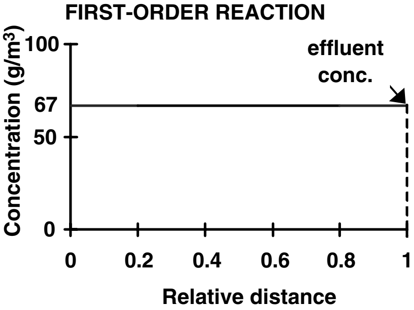
c) Biodegradable substance (with a first-order reaction)

At any point in the reactor, the concentration is given by:

$$C = \frac{C_o}{1 + K.t_h} = \frac{200}{1 + 0.40 \times 5} = 67\ g/m^3$$

The effluent concentration is also equal to 67 g/m^3. This value is higher than the value calculated for the plug-flow reactor in Example 8.1 (27 g/m^3), illustrating the fact that a complete-mix reactor is less efficient than a plug-flow reactor, for the same detention time.

The concentration profile along the tank is plotted below.



8.4.4 Cells in series

Another widely used hydraulic model is the complete-mix reactor in series, or cells in series. This system can occur in practice, such as in maturation ponds or activated sludge reactors with internal divisions, or it can be used as a theoretical model to represent intermediate hydraulic conditions between the complete-mix and the plug-flow reactor. When the total volume is distributed in *only one cell*, the system behaves like a conventional *complete-mix* reactor. Conversely, when the total volume is distributed in an *infinite number of cells*, the system reproduces *plug flow.* An *intermediate number of cells* simulates *dispersed flow,* with the system approaching the behaviour of complete mix or plug flow depending on the number of subdivisions adopted. When few cells are considered, the system tends to complete mix. On the other hand, when the system is subdivided into a larger number of cells it tends to plug flow.

Figure 8.10 presents the schematics of the two possible arrangements of cells in series, the first with cells of the same volume and the second with different volume cells.

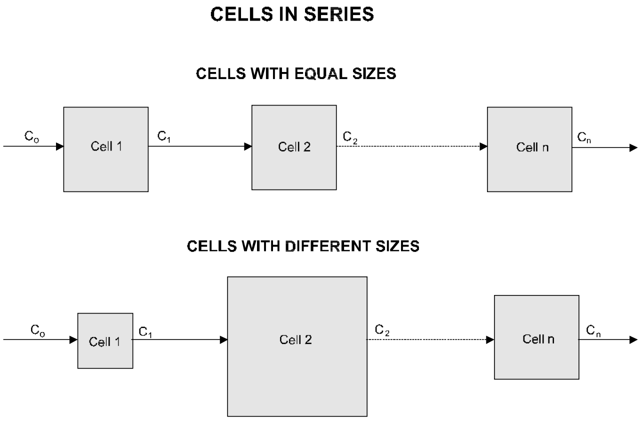


Figure 8.10. Schematic arrangement of cells in series. (a) Equal cells; (b) different cells.

The effluent concentration from each cell is given by the same formulas for complete mix. Thus, there are three possible cases, depending on the removal rate:

- *Conservative substances*

Since there is no removal of conservative substances, the effluent from each cell is equal to its influent, which is also equal to the overall influent (in the steady state). Thus, the final effluent is given by:

$$C_e = C_o \tag{8.24}$$

- *Biodegradable substances (zero-order removal)*

In zero-order reactions, the formula for a single cell is $C_e = C_o - K.t$. The effluent of the first cell is, therefore:

$$C_{e1} = C_o - K.t_1$$

where:

C_{e1} = effluent concentration from the first cell
t_1 = hydraulic detention time in the first cell

The effluent from the first cell is the influent to the second cell. Hence:

$$C_{e2} = C_{e1} - K.t_2 = C_o - K.t_1 - K.t_2$$

where:

C_{e2} = effluent concentration from the second cell
t_2 = hydraulic detention time in the second cell

For a system of n cells:

$$C_e = C_o - K.t_1 - K.t_2 - \cdots - K.t_n$$

$$C_e = C_o - K.(t_1 + t_2 + \cdots + t_n)$$

$$\boxed{C_e = C_o - K.t_h} \tag{8.25}$$

where:

C_e = final effluent concentration

t_h = hydraulic detention time in the system (summing up the volume of all cells) = $t_1 + t_2 + \cdots + t_n$

It can be observed from Equation 8.25 that the final effluent from a system of n cells in series with a *zero-order reaction* is equal to that from a one-cell complete-mix reactor (with a volume equal to the total volume of all the cells). Additionally, it must be noted that this final effluent is also equal to the effluent from a plug-flow reactor. This is as expected, considering that in zero-order reactions, the removal rate is independent of the concentration. Therefore, the three reactor systems behave in an identical manner.

- *Biodegradable substances (first-order removal)*

In the case of first-order reactions, the formula for a single cell is $C_e = C_o/(1 + Kt)$. Thus, the effluent from the first cell is:

$$C_{e1} = \frac{C_o}{(1 + K.t_1)}$$

The effluent from the first cell is the influent to the second cell. Hence:

$$C_{e2} = \frac{C_1}{(1 + K.t_2)} = \frac{C_0}{(1 + K.t_1).(1 + K.t_2)}$$

Generalising for n cells:

$$\boxed{C_{en} = \frac{C_0}{(1 + K.t_1).(1 + K.t_2)\ldots(1 + K.t_n)}} \tag{8.26}$$

If all the cells have the *same volume* (and, as a consequence, the same hydraulic detention time), Equation 8.26 is simplified to:

$$\boxed{C_e = \frac{C_o}{(1 + K.t_1)^n} = \frac{C_0}{\left(1 + K.\frac{t_h}{n}\right)^n}} \tag{8.27}$$

Table 8.5. Complete-mix cells in series. Steady-state conditions. Equations for the determination of the final effluent concentration

Reaction	Cells with different sizes	Cells with equal sizes
Conservative substance ($r_c = 0$)	$C_e = C_o$	$C_e = C_o$
Biodegradable substance (zero-order reaction; $r_c = K$)	$C_e = C_o - K.t_h$	$C_e = C_o - K.t_h$
Biodegradable substance (first-order reaction; $r_c = K.C$)	$C_e = C_o/[(1 + K.t_1)$ $\times(1 + K.t_2) \times \ldots \times (1 + K.t_n)]$	$C_e = C_o/(1 + K.t_1)^n$ $= 1/(1 + K.t_h/n)^n$

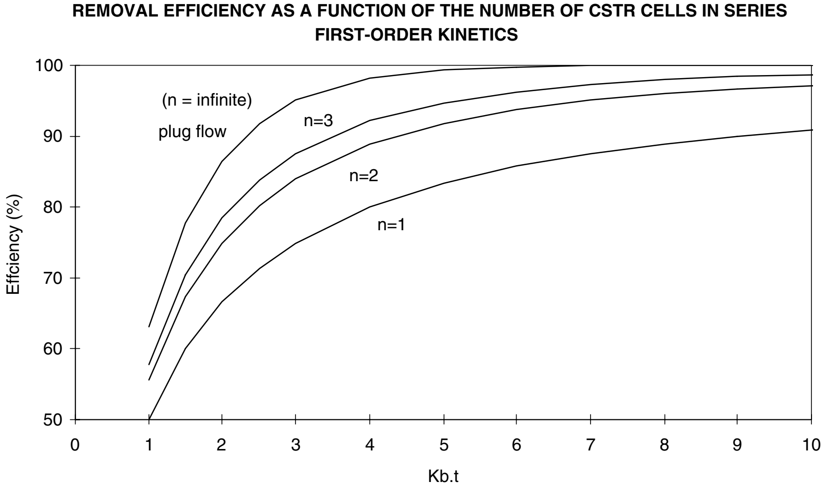


Figure 8.11. Removal efficiencies for first-order kinetics in a system composed of CSTR cells in series, as a function of the dimensionless product K.t

where:

C_e = final effluent concentration (g/m^3)
K = reaction coefficient (d^{-1})
t_1 = hydraulic detention time for only one cell (d)
t_h = total hydraulic detention time in the system (summing up the volume of all the cells) (d)
n = number of cells, all having the same volume

Table 8.5 presents a summary of the formulas for the calculation of the effluent concentration from a system composed of n cells in series.

Fig. 8.11 presents the removal efficiencies for first-order kinetics in a system composed of equal-sized CSTR cells in series, as a function of the dimensionless product K.t. The great influence of the number of cells is clearly seen.

In many practical applications, it should be taken into account that the reaction coefficient K may vary from cell to cell. For instance, the first cell, receiving more highly biodegradable substance may have a higher K value than the subsequent cells which receive a less biodegradable influent (because the compounds more easily biodegradable have been removed in the first cell).

Example 8.3

A system with three equal cells in series has the same *total* volume of the reactor in Example 8.1 (3,000 m^3). The influent also has the same characteristics from the referred example (flow = 600 m^3/d; influent substrate concentration = 200 g/m^3).

Calculate the concentration profile along the system. Assume that each cell is an ideal complete-mix reactor in a steady state and that the substance is biodegradable with first-order removal ($K = 0.40\ d^{-1}$).

Solution:

a) Hydraulic detention time in each cell

The hydraulic detention time in each cell is equal to the total detention time divided by the number of cells, that is:

$$t_1 = \frac{V}{n.Q} = \frac{3000\ m^3}{3 \times 600\ m^3/d} = 1,67\ d$$

b) Concentration in each cell

The concentration in each cell is given by (see Table 8.5):

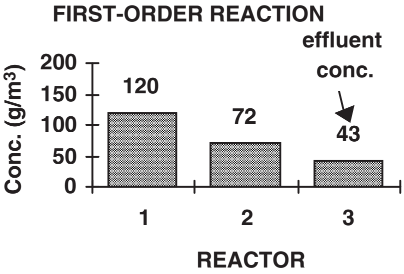
$$C = C_o/(1 + K.t_1)^n$$

For each of the 3 cells:

Formula	C (g/m^3)
$200/(1 + 0.40 \times 1.67)^1$	120
$200/(1 + 0.40 \times 1.67)^2$	72
$200/(1 + 0.40 \times 1.67)^3$	43

The final effluent concentration is equal to 43 g/m^3. As expected, this value is higher than that obtained in the most efficient system, represented by the plug flow ($C_e = 27\ g/m^3$; Example 8.1), but is lower than that from the less efficient system, represented by a single complete-mix reactor ($C_e = 67\ g/m^3$; Example 8.2).

The concentration profile along the three reactors is plotted below.



***Example 8.3* (*Continued*)**

c) Removal efficiency

The overall removal efficiency is:

$$E = (200 - 43)/200 = 0.785 = 78.5\%$$

The total hydraulic detention time is 5.0 days and the dimensionless product K.t is $0.4\ d^{-1} \times 5.0\ d = 2.0$. The efficiency of 78.5% can be also obtained from Figure 8.11, for K.t = 2.0 and n = 3 cells.

8.4.5 Dispersed flow

In real terms, the reactors that exist in practice do not behave exactly like the two idealised hydraulic models of plug flow and complete mix. However, these two ideal models configure an envelope, inside which the reactors can be found in practice. The reasons that cause real reactors not to follow the ideal models can be (Tchobanoglous & Schroeder, 1985):

- *Dispersion.* The dispersion is the longitudinal transportation of the material due to turbulence and molecular diffusion.
- *Hydraulic short circuits.* These take account of part of the flow and are the result of stratification, for instance due to a density difference, and not due to a physical characteristic of the system. The main effect is the reduction in the effective residence time.
- *Dead volumes.* The effect is similar to the short circuits (reduction of the effective residence time), but the causes are a function of the physical characteristics of the system. They occur in corners of tanks, underneath weirs and in the internal side of curves.

Consequently, the dispersed or arbitrary flow is a *non-idealised* case, and can be used in practice to describe flow conditions in most reactors. An approximation of the dispersed flow is represented by the system of cells in series, described in Item 8.4.4.

The mixing conditions in dispersed-flow reactors are characterised by a **Dispersion number**, defined as:

$$d = D/U.L \tag{8.28}$$

where:

d = dispersion number (–)

D = axial or longitudinal dispersion coefficient (L^2T^{-1})

U = mean horizontal velocity (LT^{-1})

L = reactor length (L)

Table 8.6. Typical values of d (= D/UL) for different treatment units

Treatment unit	Range of d (= D/UL)
Rectangular sedimentation tanks	0.2–2.0
Aeration tanks for activated sludge	
– plug-flow type	0.1–1.0
– complete mix type	3.0–4.0 or more
– oxidation ditches	3.0–4.0 or more
Aerated lagoons	
– long, rectangular	0.2–1.0
– square format	3.0–4.0 or more
Non-aerated stabilisation ponds	
– long, rectangular	0.1–0.3
– square format	0.8–1.2

Source: Arceivala (1981) and values from Chapter 13

In the two idealised reactors, there are the following limit conditions:

- *Plug flow*: *no dispersion* (D = 0 and d = 0)
- *Complete mixing*: *infinite dispersion* ($D = \infty$ and $d = \infty$)

The reactors found in practice have values of d situated between 0 and ∞. The value of d can be estimated by the use of tracers, a topic that is outside the scope of this text. The references (Grady & Lim, 1980; Arceivala, 1981; von Sperling, 1983b; Tchobanoglous & Schroeder, 1985; Viessman & Hammer, 1985) present the methodology and examples for this application. Table 8.6 presents ranges of d values for various treatment units. Chapter 13 presents typical values for d in stabilisation ponds, together with a simplified methodology for their estimation.

Treatment units that have d values around *0.2 or less* are closer to *plug flow*. Conversely, units with values of d around *3.0 or more* can be considered to approach *complete mix*. Among the factors that can affect the dispersion of the treatment units, the following can be listed (Arceivala, 1981):

- Scale of the mixing phenomenon;
- Geometry of the unit;
- Energy introduced per unit volume (mechanical or pneumatic);
- Type and arrangement of the inlets and outlets;
- Inflow velocity and its fluctuations;
- Density and temperature differences between inflow and reactor contents
- Reynolds number (which is a function of some of the factors listed above).

It is important to note that the characterisation between plug-flow and complete-mix conditions is also a function of the dynamics of the constituent being analysed. For example, oxidation ditches behave like complete-mix reactors for most of the variables, such as suspended solids and BOD. Samples collected along its length will give approximately the same concentrations. However, for constituents that exhibit fast dynamics, the situation is different. Dissolved oxygen (DO) in activated sludge reactors presents very rapid dynamics, with fast increases or decreases in its concentration. For this reason, DO concentrations are high in the vicinity of the aerators, decreasing due to the bacterial consumption as the liquid flows along the ditch, until it reaches the next aerator. Therefore, there is a gradient of the longitudinal DO concentration along the tank, what characterises a regime approaching plug flow.

The analytical solution of the equation for dispersed flow with first-order kinetics was proposed by Wehner and Wilhem in 1956. For other reactions different from first order, numerical solutions are necessary. The equation for first-order reactions is:

$$\boxed{\begin{aligned} & C = C_0 . \frac{4ae^{1/2d}}{(1+a)^2\, e^{a/2d} - (1-a)^2\, e^{-a/2d}} \\ & a = \sqrt{1 + 4K.t.d} \end{aligned}} \qquad (8.29)$$

where:

- d = dispersion number = $D/UL = D.t/L^2$ (–)
- D = coefficient of longitudinal dispersion (m^2/d)
- U = average flow velocity in the reactor (m/d)
- L = travel distance (m)
- t_h = hydraulic detention time (= V/Q) (d)
- K = removal coefficient (d^{-1})
- C = effluent concentration (g/m^3)
- C_o = influent concentration (g/m^3)

The advantage of this equation is that it allows a continuous solution between the limits of plug flow and complete mix. When d is small, Equation 8.29 gives results very close to the specific equation for plug flow. On the other hand, when d is very high, Equation 8.29 produces similar values to those obtained from the equation for complete mix.

The use of the Wehner–Wilhem equation can be facilitated through the employment of graphs. Figure 8.12 presents a graph of the dimensionless product $K.t_h$ versus the removal efficiency $[(C_o - C)/C_o]$, following first-order kinetics. Various curves are presented, all situated inside the envelope represented by d varying from 0 (plug flow) to ∞ (complete mix). In the design of a treatment unit, given the values of d and K and for a desired removal efficiency, the necessary hydraulic detention time t_h (and as a result the reactor volume) can be obtained. Similarly, if it is desired to estimate the efficiency of a reactor with a pre-defined volume, knowing K, t_h and d, the efficiency can be readily obtained from the graph.

Figure 8.13 presents the same family of curves, for a broader scale of $K.t_h$, and for a greater efficiency range (applicable, for instance, to coliform removal,

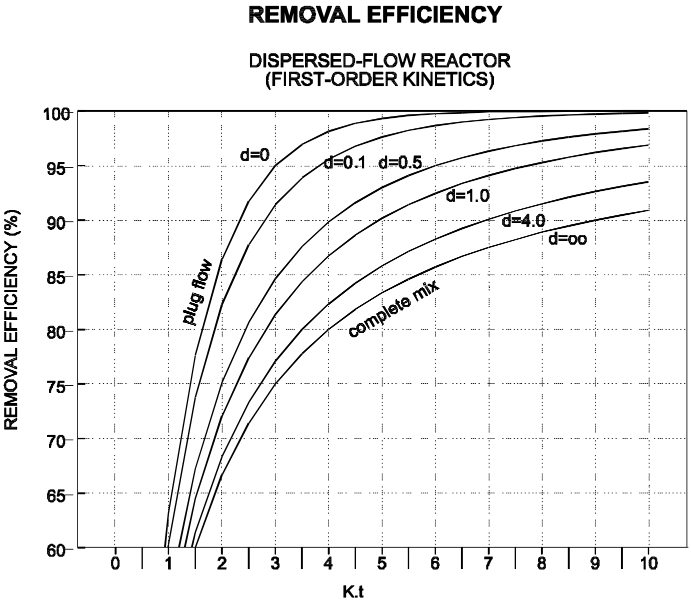


Figure 8.12. Removal efficiency (first order reaction) according to the Wehner–Wilhem equation for dispersed flow

in which high efficiencies are required). The removal efficiencies are presented in a logarithmic scale and also in terms of log units removed. An efficiency of E = 90% corresponds to a removal of 1 logarithmic unit; E = 99% → 2 log units; E = 99.9% → 3 log units; E = 99.99% → 4 log units; E = 99.999% → 5 log units, or:

$$\boxed{\text{Log units removed} = -\log_{10}[(100 - E)/100]} \qquad (8.30)$$

where:

E = removal efficiency, expressed in percentage (%)

The interpretation of Figures 8.12 and 8.13 for constituents that decay following first-order kinetics leads to the following points (Arceivala, 1981):

- For a given value of $K.t_h$, the reactors that approach plug flow always give higher efficiencies than the reactors that approach complete mix.
- A complete-mix reactor or even a relatively well-mixed reactor ($d > 4.0$) is incapable of giving a removal efficiency greater than 97% for values of $K.t_h$ less than 20.
- Very high efficiencies (greater than 99%), can only be reached if the system approaches plug-flow conditions (if the removal coefficient K is not especially high, or if the adoption of very high detention times is not desired).

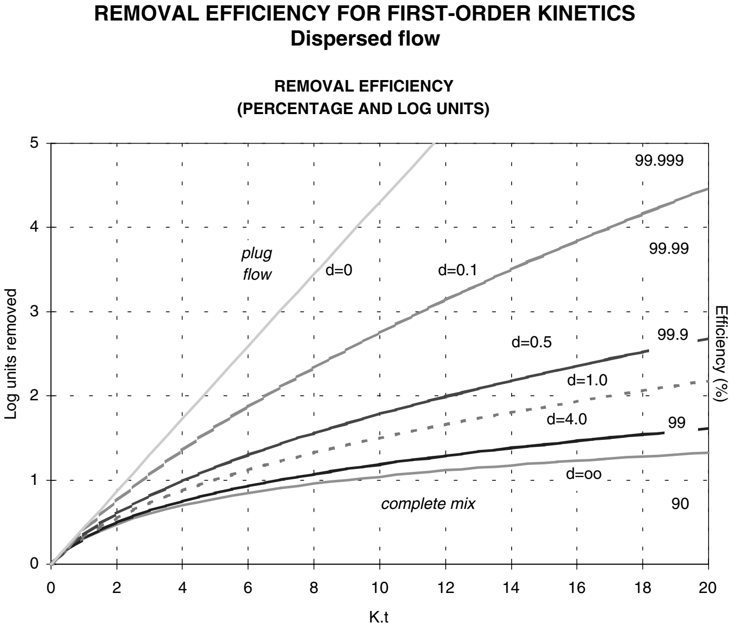


Figure 8.13. Removal efficiency following a first-order reaction, in a dispersed-flow reactor, for different values of $K.t_h$ and d

Example 8.4

A reactor has the same volume as the reactor of Example 8.1 (3,000 m^3). The influent also has the same characteristics as the referred example (flow = 600 m^3/d; influent substrate concentration = 200 g/m^3).

Calculate the effluent concentration from the reactor. Assume that the dispersion number is 1.0 and that the substance is biodegradable with first order removal ($K = 0.40\ d^{-1}$).

Solution:

a) Hydraulic detention time

The detention time is calculated in the same way as in Example 8.1, that is, $t_h = 5$ days.

b) Calculation of the parameter *a*

According to Equation 8.29:

$$a = (1 + 4.K.t_h.d)^{0.5} = (1 + 4 \times 0.4 \times 5 \times 1.0)^{0.5} = 3.0$$

***Example 8.4* (*Continued*)**

c) Calculation of the effluent concentration

According to Equation 8.29:

$$C = C_0.\frac{4ae^{1/2d}}{(1+a)^2e^{a/2d} - (1-a)^2e^{-a/2d}}$$

$$= 200.\frac{4 \times 3.0.e^{1/(2\times1.0)}}{(1+3.0)^2.e^{3.0/(2\times1.0)} - (1-3.0)^2.e^{-3./(2\times1.0)}}$$

$$= 200 \times 0.28 = 56\ g/m^3$$

This value is between the values obtained for a plug-flow reactor ($C_e = 27 g/m^3$; Example 8.1) and complete-mix reactor ($C_e = 67 g/m^3$; Example 8.2), although it is closer to a complete-mix reactor (because of the relatively high dispersion number).

The same value can be obtained from Figure 8.12. For $K.t_h = 0.4 \times 5 = 2.0$ and $d = 1.0$, a removal efficiency of 72% is obtained. With a removal of 72%, the remaining concentration is 28%, what corresponds to the value of 0.28 obtained in the second term on the right-hand-side of the Wehner–Wilhem equation above. Therefore, $C_e = 200 \times 0.28 = 56 g/m^3$.

8.4.6 Cells in parallel

A treatment system is frequently composed of cells in parallel. Figure 8.14 shows a possible arrangement of cells in parallel.

With cells in parallel, the following points should be noted (Arceivala, 1981):

- The cells can be equal or different in size, since they operate independently.
- Even if the cells are of different sizes, they can be operated with the same detention time through the individual adjustment of each inlet flow.

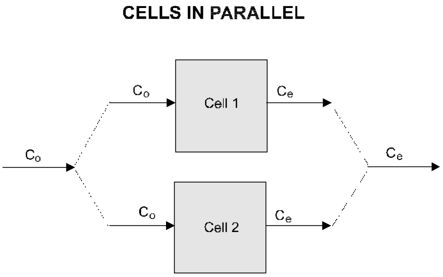


Figure 8.14. Schematic arrangement of cells in parallel

- Each cell can be designed individually using the dispersed-flow model and adequate values of d for each cell. The d values can vary from cell to cell.
- If each cell behaves as a complete-mix reactor, the final effluent will have the same concentration of that from a reactor with only one cell and a volume equal to the total volume of all cells. Therefore, the subdivision of a reactor (complete mix) into parallel cells (each one complete mix) does not affect the effluent concentration.
- However, the point above will not apply for reactors modelled according to dispersed flow, since the subdivision of a reactor into smaller reactors in parallel can lead to reactors with different geometries from the original large reactor. Therefore, the smaller reactors will have different dispersion numbers and, as a result, different effluent concentrations.
- For a given total volume, the substrate removal efficiency in first-order reactions is lower for cells in parallel than for cells in series. However, a parallel arrangement is frequently convenient due to reasons such as operational flexibility, operation continuity even with the closing of one unit, sludge removal etc.

8.4.7 Cells in series with incremental feeding

When there is an arrangement of cells *in series*, it is possible that the inflow distribution is split between the various cells. Hence, each cell is fed not only by the effluent from the upstream cell, but also by a fraction of the general influent. In activated sludge the denomination *step feed* has been employed to classify plug-flow reactors or cells in series that receive this type of incremental feed. Such an arrangement can also be used in stabilisation ponds and trickling filters. Figure 8.15 shows possible arrangements of cells in series with step feed.

In an arrangement in series with n cells of equal or different volumes with incremental feeding, all the liquid fractions do not receive the same treatment exposure. The first fraction receives treatment in all the cells; the second fraction is treated in n − 1 cells; the third in n − 2 cells and so on. When the cells have the same volume and receive the same fraction of the total flow, and decay follows a

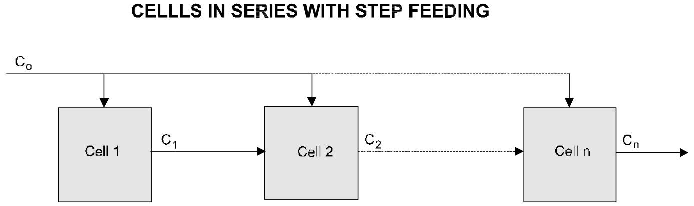


Figure 8.15. Schematic arrangement of cells in series with step feed

first-order reaction, the effluent concentration is given by (Arceivala, 1981):

$$\boxed{C_e = \frac{C_o}{1 + K.\left(\frac{n.V_1}{Q}\right)}} \tag{8.31}$$

where:

n = number of cells in series with incremental feeding (–)
V_1 = volume of each cell (assuming equal volumes) (m^3)
Q = total influent flow in the system (m^3/d)

It can be seen that such an arrangement has the same efficiency of a single complete-mix cell with an equivalent total volume. In other words, a reactor with incremental feeding behaves like a complete-mix reactor. In terms of efficiency, the incremental feeding loses the benefits from the arrangement in series. Obviously, other reasons of practical and operational order may justify the inclusion of this option, principally a greater operational flexibility. In the cases when the inlet end of a reactor or the first cell are overloaded, such flexibility can contribute to the control of this localised overload.

If the cells have different volumes and flows, the calculation can be done individually in each cell through the individual mass balances. If convenient, the dispersed flow model can be adopted for each cell with the corresponding d value.

8.4.8 Cells in series and in parallel

With the aim of having greater operational flexibility, the arrangement of cells in series and in parallel is frequently used. Thus, there are the benefits of efficiency with the arrangement in series and of flexibility with the arrangement in parallel. Figure 8.16 shows a typical series/parallel arrangement.

Since the units in parallel do not interfere in the efficiency, the calculations of the effluent concentration can be done using the formulas for cells in series (Table 8.5), adopting the complete-mix model and the corresponding value of

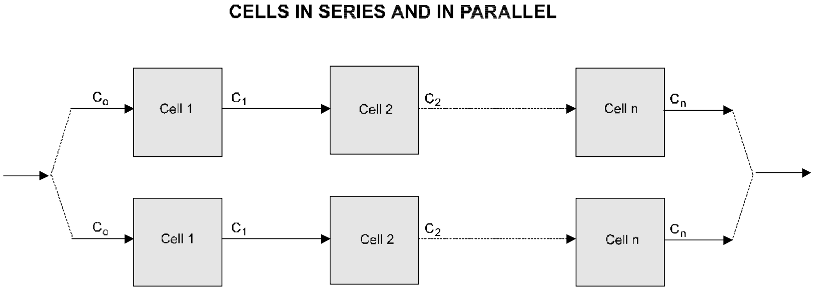


Figure 8.16. Schematic arrangement of cells in series and in parallel

n (number of cells in series in each line). Alternatively, the dispersed-flow model can be used with the appropriate value of d for each cell.

8.4.9 The influence of recirculation

In plug-flow or cells-in-series systems, a fraction of the effluent can be recirculated to the inlet of the reactor. Depending on the application, this fractions can be lower, equal or greater than the influent flow.

The recirculation is an inherent component of various treatment processes, such as activated sludge and high rate trickling filters. In the case of the activated sludge system, the recirculation is specific, since the recycled liquid has different characteristics (sludge removed from the bottom of the secondary settling tank). Therefore, the characteristics of the return (recycle) sludge are different from the effluent from the reactor, principally in terms of the concentration of the suspended solids. Given the importance of this phenomenon and the complexity of its interactions with the reactor, the sludge recirculation in activated sludge systems is not covered in the present section (see Part 5).

In any situation, when making a mass balance on the reactor, the following points must be taken into consideration:

- The incoming flow to the reactor is equal to the sum of the influent flow (Q_o) and the recycled flow (Q_r).
- The concentration in the incoming flow to the reactor is given by a weighted average between the influent and the recycled flows, according to:

$$\boxed{C'_0 = \frac{Q_0.C_0 + Q_r.C_r}{Q_0 + Q_r}} \tag{8.32}$$

where:

C'_o = incoming concentration in the reactor (mixture of the influent and the recycled flows) (g/m^3)
C_o = concentration in the influent to the system (g/m^3)
C_r = concentration of the recycled liquid (g/m^3)
Q_o = influent flow to the system (m^3/d)
Q_r = recycled flow (m^3/d)

- The outgoing flow from the reactor is equal to the sum of the influent flow (Q_o) and the recycled flow (Q_r).
- The concentration in the reactor outlet is obtained using the pertinent equations, as a function of the reaction order and the hydraulic model adopted.

In the case of activated sludge, in which the sludge comes from the bottom of the secondary settling tank, the concentrations of the suspended solids in the return sludge are much higher than in the influent concentration. In these conditions, the weighted average leads to an incoming value C'_o greater than in the

Table 8.7. Required hydraulic detention time to obtain an effluent concentration C_e (steady state)

	Hydraulic detention time	
Reaction order	Complete mix	Plug flow
0	$(C_e - C_o)/K$	$(C_e - C_o)/K$
1	$[(C_o/C_e) - 1]/K$	$[\ln(C_o/C_e)]/K$
Saturation	$(C_o - C_e).(K_s + C_e)/(K.C_e)$	$[(K_s.\ln(C_o/C_e) + C_o - C_e]/K$

Source: Benefield and Randall (1980), Tchobanoglous and Schroeder (1985)
K = reaction coefficient
K_s = half-saturation coefficient

influent. Conversely, the substrate concentration is lower than in the influent, and the recirculation contributes to the reduction of the influent concentration.

In a generic system, in which the recirculation is taken directly from the effluent from the reactor (with low substrate concentration), the recirculation becomes responsible for a dilution of the influent substrate concentration. Consequently, the efficiency of the system is reduced, in the case of first or higher order reactions.

8.4.10 Comparison between the reactor types

The comparison between the performances of the various types of reactor is an important topic in the analysis and design of a wastewater treatment plant. As previously mentioned, the following generalisations can be made, assuming steady-state conditions.

- *Conservative substances*: plug-flow reactors, cells in series and complete-mix reactors present the same performance.
- *Biodegradable substances with a zero-order reaction*: plug-flow reactors, cells in series and complete-mix reactors present the same performance.
- *Biodegradable substances with a first-order reaction*: the plug-flow reactor presents the highest efficiency, followed by the cells-in-series system. The single complete-mix reactor is the least efficient.

The statement that a system is more efficient than another implies that, if both present the same effluent concentration, the less efficient system requires a higher detention time. In other words, the less efficient system must have a larger reactor volume. This consideration is of great importance in the design of a treatment plant. Table 8.7 presents a summary of the equations used to calculate the detention time required to obtain a certain concentration C_e in the effluent.

The interpretation of Table 8.7 leads to the following points:

- *Zero-order reactions*. For zero-order reactions, the required hydraulic detention is the same.
- *First-order reactions.* For first-order reactions, the application of the pertinent formulas leads to the requirement of the greatest detention times for the complete-mix system.

Table 8.8. Relative volumes (expressed as $K.t_h$) required for various removal efficiencies. First-order reactions (steady state)

	Relative volume (dimensionless product $K.t_h$)					
Number of reactors in series	85% efficiency	90% efficiency	95% efficiency	99% efficiency	99.9% efficiency	99.99% efficiency
1	5.7	9.0	19	99	999	9999
2	3.2	4.3	7.0	18	61	198
4	2.5	3.1	4.5	8.6	18	36
∞ (plug flow)	1.9	2.3	3.0	4.6	6.9	9.2

Source: Arceivala (1981), Metcalf & Eddy (1991)

- *Saturation reactions.* For saturation reactions, the result depends on the relative value of K_s with respect to C_e. When $C_e >> K_s$, the reaction tends to zero order and the reactor volumes for complete mix and plug flow are approximately the same. When $C_e << K_s$, the reaction tends to first order and the complete-mix system requires larger volumes than the plug-flow system.

Table 8.8 presents the relative volumes necessary for obtaining different removal efficiency values (assuming ideal complete-mix reactors and a first-order reaction). The table gives values of the dimensionless product $K.t_h$. Based on the desired efficiency and on the product $K.t_h$ (after knowing or estimating K), the required detention time can be obtained. With the detention time and the flow, the volume can be determined ($V = t_h.Q$).

It is confirmed in this table the fact that, for first-order reactions, the plug flow requires the lowest volume for a given efficiency. The greater the required efficiency, the higher the ratio (complete mix volume) / (plug flow volume). For an efficiency of 85% this ratio is 3.0 (= 5.7/1.9), that is, the volume required by a single cell is three times higher than that of the plug flow. However, for an efficiency of 99% this ratio becomes 21.5 (= 99/4.6). The simple subdivision of the total volume in 2 cells changes these ratios to 1.7 (= 3.2/1.9) and 3.9 (= 18/4.6), respectively. However, it should be remembered that these considerations are based on the assumption of ideal complete-mix and plug-flow reactors, which is hardly achieved in practice.

8.4.11 Comparison between first-order reaction coefficients in different hydraulic models

8.4.11.1 Estimation of the reaction coefficients in existing reactors

Table 8.9 presents the formulae for the estimation of the effluent concentration of a first-order decay pollutant, as a function of the hydraulic regime assumed for the reactor. For an existing reactor, the coefficient K can be calculated by rearranging the equations in Tables 8.3 and 8.4, and making K explicit, provided the influent concentration C_0, the effluent concentration C and the detention

Table 8.9. Formulas for the estimation of the first-order decay coefficient K, for different hydraulic regimes

	Formula for the decay coefficient (K)	
Hydraulic regime	Equation	Equation number
Plug flow	$K = \frac{-\ln (C/C_0)}{t}$	(8.33)
Complete mix	$K = \frac{(C_0/C) - 1}{t}$	(8.34)
Dispersed flow	K value not explicit. Solve by iteration (trial-and-error or error function minimisation)	–

time t (also the dispersion number d, for dispersed-flow models) are known or have been determined. The analysis undertaken in the present section is based on von Sperling (2002).

For a given removal efficiency, the estimation of K based on the detention time and on the influent and effluent concentrations on an existing reactor leads to the two following divergent situations:

- adoption of the **complete-mix** (CSTR) model leads to K values which are **greater** than those found for dispersed flow
- adoption of the **plug-flow** model leads to K values which are **lower** than those found for dispersed flow

The following example will help to clarify the point. An existing reactor has the following average values of performance indicators: (a) influent coliform concentration: $C_0 = 1 \times 10^7$ FC/100 ml; (b) effluent coliform concentration: $C = 2.13 \times 10^5$ FC/100 ml; (c) detention time: $t = 30$ days; (d) dispersion number: $d = 0.5$. Use of Equations 8.33 and 8.34 will lead to the K coefficients for plug flow and complete mix, respectively. An iterative process of trial-and-error will lead to the K coefficient for dispersed flow. The following K values are obtained: (a) plug flow: $K = 0.13\ d^{-1}$; (b) CSTR: $K = 1.53\ d^{-1}$; (c) dispersed flow: $K = 0.30\ d^{-1}$. As can be seen, for the same reactor and the same kinetics, different K values are obtained in practice, depending on the hydraulic regime assumed.

In principle, there should be only one coefficient, representing the decay of the constituent, according to its kinetics. However, the inadequacy of idealised models in representing in a perfect manner the real hydraulic pattern in the reactor leads to the deviations that occur in practice. The reason for the differences observed in the example above is that, since complete-mix reactors are the least efficient for first-order removal kinetics, the lower efficiency is compensated by a higher K value. Conversely, since plug-flow reactors are the most efficient reactors, the K value is reduced to produce the same effluent quality. Depending on the length/breadth (L/B) ratio of the reactor (dispersion characteristics), the deviation can be very large, inducing considerable errors in the estimation. Naturally, K coefficients for dispersed flow are assumed to best represent reality and the true reaction kinetics. However, the confidence on K values for dispersed flow relies very much on the confidence on the assumed or determined values of the dispersion number d.

These divergences have been the subject of considerable confusion in the literature, when expressing K values. Reported K values usually show considerable variations, a large part of which can be attributed to inadequate consideration of the hydraulic regime of the reactor.

8.4.11.2 Relationship between K for idealised regimens (complete mix and plug flow) and K for dispersed flow

The present section, also based on von Sperling (2002), describes the relationship between first-order K values for the idealised flow patterns (CSTR – K_{CSTR} and plug flow – K_{plug}), and K for the general flow pattern, dispersed flow (K_{disp}).

The following explanation demonstrates the methodology applied for the CSTR regime. A similar methodology, using the appropriate equations, was also used for the plug-flow regime. Using the relevant equations for estimating the effluent concentrations under complete mix and dispersed flow, it was calculated, for different values of the dimensionless product $K_{disp}.t$ and dispersion number d, the corresponding K_{CSTR}, which yields the same efficiency of removal (first-order kinetics). The dispersion numbers d ranged from extremely high values (100,000, representing complete-mix conditions) to extremely low values (0.001, representing plug-flow conditions).

The results are presented in Table 8.10, showing the ratio between the K for CSTR and K for dispersed flow (K_{CSTR}/K_{disp}). The interpretation of the table is as follows. The same reactor analysed in the previous section, with $d = 0.5$, detention time $t = 30$ days and $K_{disp} = 0.3\ d^{-1}$ has the dimensionless product $K_{disp}.t = 0.3 \times 30 = 9.0$. For $d = 0.5$ and $K_{disp}.t = 9$, the table shows that the K_{CSTR} is equal to 5.144 times K_{disp}. In other words, K_{CSTR} is $5.144 \times 0.3 = 1.54\ d^{-1}$. This value is, apart from rounding values, the same obtained in the previous section ($1.53\ d^{-1}$), indicating the applicability of the table. The estimation of the removal efficiency using the dispersed-flow model (using K_b for dispersed flow) and the CSTR model (using K_b for CSTR) will lead to the same results.

Table 8.11 shows the corresponding values for the plug-flow model. In the same example, it is seen from Table 8.11 that, for $d = 0.5$ and $K_{disp}.\ t = 9$, K_{plug} is 0.430 times K_{disp}. Therefore, $K_{plug} = 0.430 \times 0.3 = 0.13\ d^{-1}$ (which is exactly the same value determined in the previous section).

Figure 8.17 illustrates the data from Tables 8.10 and 8.11. It can be clearly seen that, for the CSTR regime, the smaller the dispersion number d, the greater the departure between K_{CSTR} and K_{disp}. Conversely, for the plug-flow regime, the greater the Dispersion number d, the greater is the departure between K_{plug} and K_{disp}. The departure also increases with the detention time t. Another point to be observed is that the relative departures can be much larger for the CSTR regime

Table 8.10. Ratio between the K coefficients obtained for the complete-mix model and the dispersed-flow model, for different values of the dispersion number d and of the product K_{disp}.t

	Ratio K_{CSTR}/ K_{disp}							
K_{disp}.t	d = 100,000	d = 4	d = 1	d = 0.5	d = 0.2	d = 0.1	d = 0.02	d = 0.001
0	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
1	1.000	1.040	1.140	1.230	1.400	1.520	1.670	1.715
2	1.000	1.075	1.290	1.515	1.950	2.320	2.940	3.180
3	1.000	1.120	1.457	1.833	2.677	3.550	5.380	6.300
4	1.000	1.163	1.635	2.213	3.658	5.393	10.150	13.175
5	1.000	1.210	1.832	2.646	4.950	8.180	19.440	28.800
6	1.000	1.255	2.043	3.150	6.617	12.283	37.620	64.667
7	1.000	1.300	2.271	3.729	8.814	18.214	73.000	149.000
8	1.000	1.346	2.525	4.388	11.600	26.813	141.000	350.000
9	1.000	1.394	2.789	5.144	15.156	39.111	272.780	831.111
10	1.000	1.444	3.080	6.010	19.660	56.500	524.000	1995.000

Table 8.11. Ratio between the K coefficients obtained for the plug-flow model and the dispersed-flow model, for different values of the dispersion number d and of the product K_{disp}.t

	Ratio K_{plug}/K_{disp}							
K_{disp}.t	d = 100,000	d = 4	d = 1	d = 0.5	d = 0.2	d = 0.1	d = 0.02	d = 0.001
0	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
1	0.695	0.715	0.762	0.805	0.878	0.926	0.984	1.000
2	0.551	0.578	0.640	0.699	0.797	0.868	0.967	1.000
3	0.463	0.493	0.562	0.626	0.736	0.820	0.950	1.000
4	0.404	0.435	0.506	0.574	0.689	0.782	0.935	0.999
5	0.359	0.392	0.465	0.532	0.652	0.749	0.920	0.998
6	0.325	0.358	0.432	0.500	0.620	0.721	0.907	0.997
7	0.298	0.331	0.405	0.473	0.593	0.696	0.894	0.996
8	0.276	0.309	0.383	0.450	0.569	0.674	0.882	0.995
9	0.257	0.291	0.364	0.430	0.549	0.654	0.870	0.994
10	0.241	0.274	0.347	0.413	0.530	0.636	0.859	0.993

than for the plug-flow regime, indicating that an even greater caution needs to be exercised when applying the CSTR model.

In reactors without mechanical mixing, the lowering of the dispersion number d occurs with the increase in the length/breadth (L/B) ratio. In other words, a baffled reactor is likely to have a low value of d. Under these circumstances, utilisation of the CSTR model will be completely inadequate, due to the large difference between K_{CSTR} and $K_{dispersed}$, the latter being naturally expected to be a better predictor of the actual behaviour in the reactor. In this baffled reactor, use of the CSTR model for design purposes, adopting 'typical' values of K_{CSTR} from the literature will lead to an underestimation of the removal efficiency in the reactor. On the other hand, for an existing baffled reactor, the calculation of the coefficient

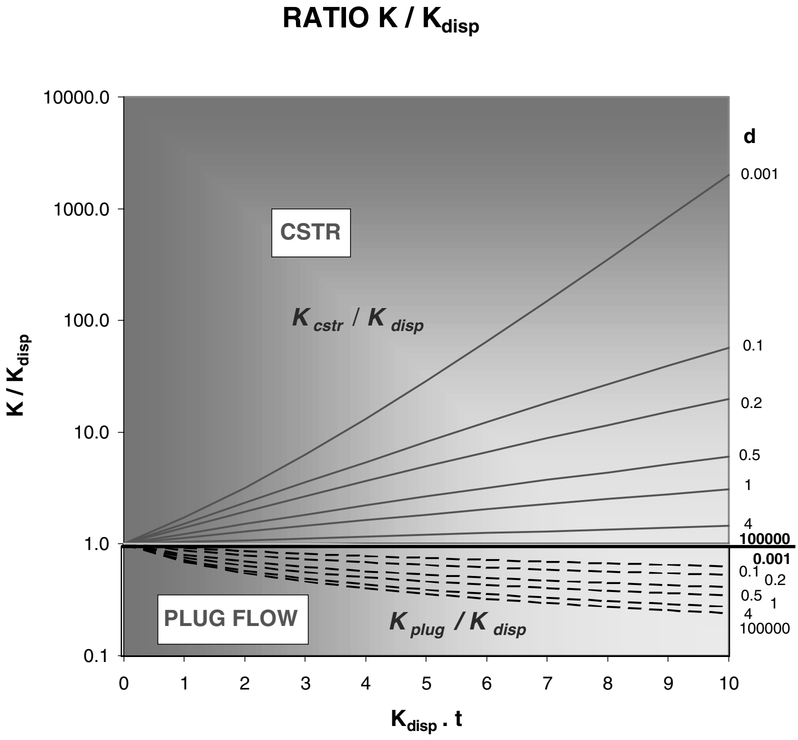


Figure 8.17. Relationship between coefficients K for CSTR and plug flow with the coefficient K for dispersed flow, as a function of the dispersion number d and the hydraulic detention time t.

K using the CSTR model will lead to an overestimation of the K coefficient, in order to compensate for the inherent lower efficiency associated with the CSTR model.

In order to extend the applicability of Tables 8.10 and 8.11, a regression analysis was done by von Sperling (2002), having as dependent variable the ratio between the K value for the idealised regime (CSTR or plug flow) and the K value for the general regime (dispersed flow). The dependent variables were then K_{CSTR} / K_{disp} and K_{plug} / K_{disp}. The independent variables were the dimensionless product $K_{disp}.t$ and the dispersion number d. Two regression analyses were done, each one having different applicability ranges. The equations of best fit obtained were:

Wider applicability range (d from 0.1 to 4.0; $K_{disp}.t$ from 0 to 10; n = 55 values from Tables 8.10 and 8.11):

- *For CSTR* ($R^2 = 0.994$):

$$\frac{K_{CSTR}}{K_{disp}} = 1.0 + \left[0.0020 \times (K_{disp}.t)^{3.0137} \times d^{-1.4145}\right] \tag{8.35}$$

- *For plug flow* ($R^2 = 0.956$)

$$\frac{K_{plug}}{K_{disp}} = 1.0 - \left[0.2414 \times (K_{disp}.t)^{0.4157} \times d^{0.1880}\right] \tag{8.36}$$

Narrower applicability range (d from 0.1 to 1.0; $K_{disp}.t$ from 0 to 5; n = 24 values from Tables 8.10 and 8.11):

- *For CSTR* ($R^2 = 0.994$)

$$\frac{K_{CSTR}}{K_{disp}} = 1.0 + \left[0.0540 \times (K_{disp}.t)^{1.8166} \times d^{-0.8426}\right] \tag{8.37}$$

- *For plug flow* ($R^2 = 0.987$)

$$\frac{K_{plug}}{K_{disp}} = 1.0 - \left[0.2425 \times (K_{disp}.t)^{0.3451} \times d^{0.3415}\right] \tag{8.38}$$

All fits were very good, as indicated by the high R^2 values obtained. The reason for having equations for two applicability ranges is that the wider-range equation is not very accurate for lower values of d or $K_{disp}.t$, therefore making the narrower-range equations more adequate under these circumstances. From the equations, it is seen that *K_{CSTR}/K_{disp} will always be greater than 1.0*, whereas *K_{plug}/K_{disp} will always be lower than 1.0.*

8.4.12 The influence of variable loads

8.4.12.1 General concepts

The comparison between the efficiencies presented in Section 8.4.10 was based on the steady-state assumption, in which the influent characteristics remain constant. In a wastewater treatment plant this constancy rarely occurs. The variation of the flow and concentration along the day is responsible for the fact that, in reality, the system always operates in a *dynamic state*. Besides this, various other factors can contribute to a greater variability, such as stormwater flow (especially in combined systems) and industrial discharges. The latter can occur without any established periodical pattern and can be responsible for shock loads at the works. The shock loads can be of various natures, such as hydraulic, organic, toxic, of a non-biodegradable substance, thermal etc. A wastewater treatment plant must be apt to receive overloads that occur routinely or frequently, as well as a major part of the unpredicted overloads.

In the situations in which this variability component is substantial, the conception of the system must take this fact into consideration, the importance of which could even surpass the efficiency considerations discussed in Section 8.4.10. The effects of shock loads are best evaluated through the study of transients, using dynamic mathematical models of the system. These simulations can use the typical or

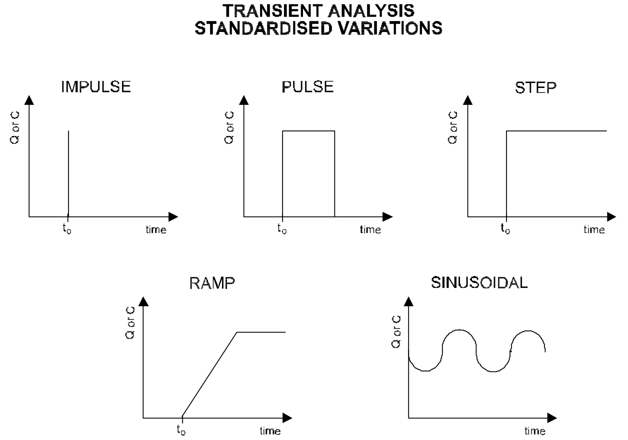


Figure 8.18. Transient analysis. Standardised influent variations.

expected variations of the influent characteristics, such as standardised variations. Some of the standardised variations of the influent characteristics normally used in transient analysis are (see Figure 8.18):

The analysis of these elements is outside the scope of this book (with the exception of the step function, covered in Sections 8.4.12.2 and 8.4.12.3). However, some generalisations can be made:

- *Toxic substances instantaneously added as spikes*. The peak in the effluent from the complete-mix reactor is the smallest, opposed to the plug-flow reactor, which presents the highest peak in the effluent. The good performance of the complete-mix reactor is caused by the large and instantaneous dilution provided at the entrance in the reactor. Additionally, the greater volume required for the single-cell complete-mix reactor contributes to the smoothing of the shock load. In the first reactor of a system with cells in series or at the head of a plug-flow reactor the toxic concentrations can be very high, due to the lower volumes involved.
- *Toxic substances with step increase*. The plug-flow reactor subjected to a step load of a conservative substance reaches a new equilibrium concentration after a time equal to 1 t_h. In the same period, the complete-mix reactor reaches only 63% of the equilibrium concentration, 3 t_h being required for the concentration in the reactor to reach 95% of the equilibrium concentration. This larger time can be fundamental for sustaining the system or for corrective operational control measures to be taken. Also in this case the larger volumes usually found in the complete-mix reactors contribute to a greater stability in the system.
- *Overload of biodegradable substances*. As already seen, for first-order reactions, the efficiency of a single complete-mix reactor is lower than that of

a system in series or a plug flow. However, the consideration of the volume of the unit plays an important role in the case of transients. In the first cell of a system in series or in the head of a plug-flow reactor, due to the lower volume involved, the effect of the overload can be more deleterious. In aerobic systems, if the oxygenation capacity in these volumes is not sufficient, the organic overload could even lead to anaerobic conditions.
- *Hydraulic overload*. When a sudden increase in the flow occurs, a dilution of the reactor contents also takes place, which can be responsible for the washing-out of biomass from the reactor. With the decrease in biomass concentration, there is a resulting reduction in the efficiency of the system. The smaller the reactor volume, the greater is its susceptibility to this wash-out. For this reason, single-cell complete-mix reactors are more stable than systems in series or plug flow.

Depending on the way in which the final effluent quality is monitored, the concept of efficiency can vary:

- *Composite samples*. Systems that verify the final effluent quality through composite samples cannot detect the concentration peaks in the effluent. In these cases, the greater stability provided by the complete-mix system may not be apparent
- *Simple samples*. Systems that verify the final effluent quality through simple (instantaneous or grab) samples are subject to the collection of a sample at a moment of peak concentration in the effluent. This can be sufficient for a WWTP to be detected as infringing the discharge standards. In this situation, the stability provided by the complete-mix system will be apparent.

In summary, the selection between one type of reactor and another is a compromise between **mean efficiency** and **stability**. Each case must be analysed individually.

8.4.12.2 Plug-flow reactor subjected to step variations in the influent concentration

In an ideal plug-flow reactor, in the cases when the influent concentration increases instantaneously to a new level (at which it stays), the behaviour is very similar to that described for constant concentrations (steady state). The main difference is in the sense that the change takes place while the plug with the new concentration flows downstream. The particles downstream still have the old concentration, while the particles upstream are already with the new higher concentration. In this ideal plug-flow reactor, the effluent concentration will only be altered after the complete flow of the plug, which takes exactly the same time as the hydraulic detention time. Figure 8.19 illustrates the behaviour of a plug-flow reactor subjected to a step increase in the influent concentration of conservative constituents and constituents that decay according to 0 and 1st order kinetics.

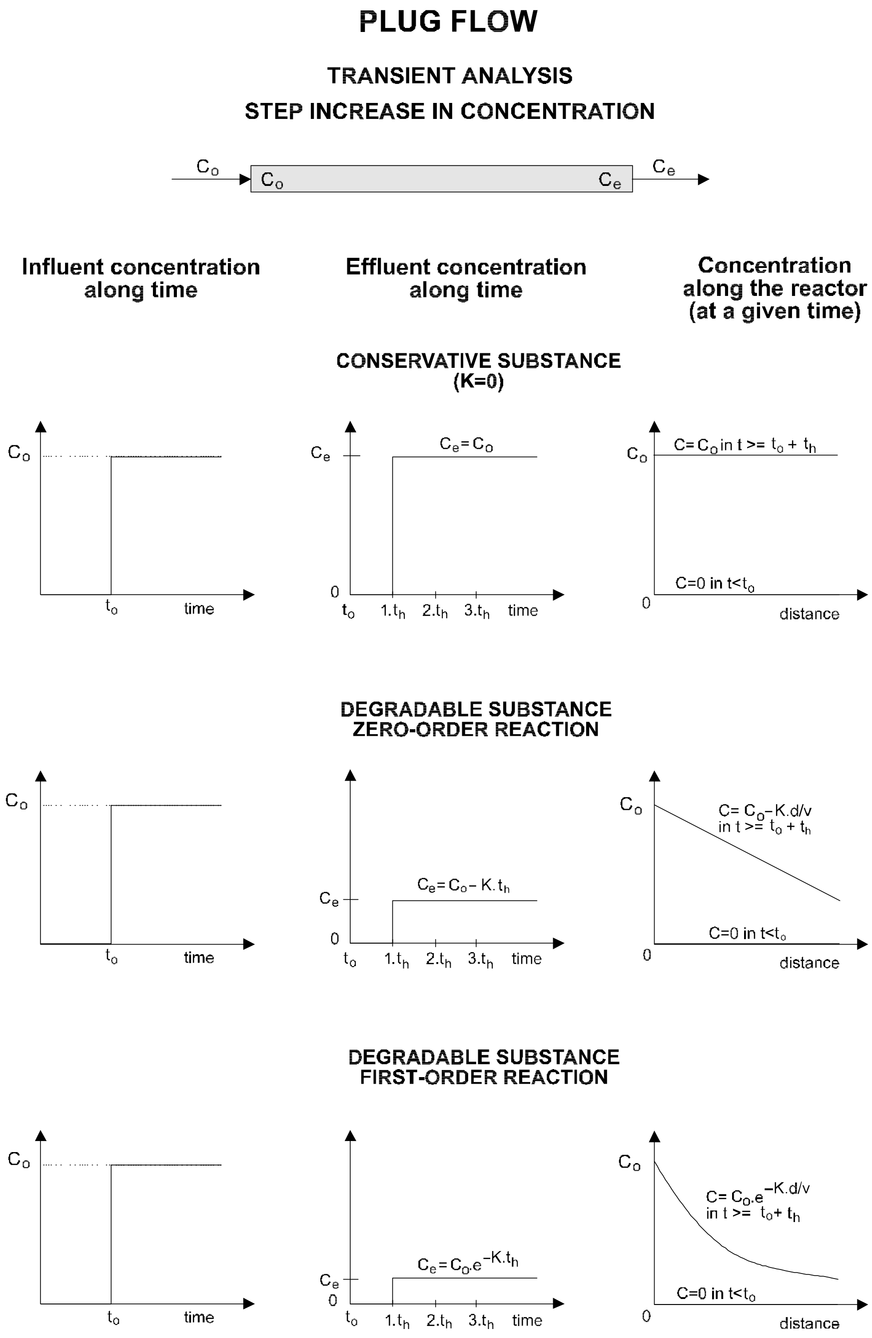


Figure 8.19. Transient analysis in a plug-flow reactor. Step increase in the concentration. (d = distance along the reactor).

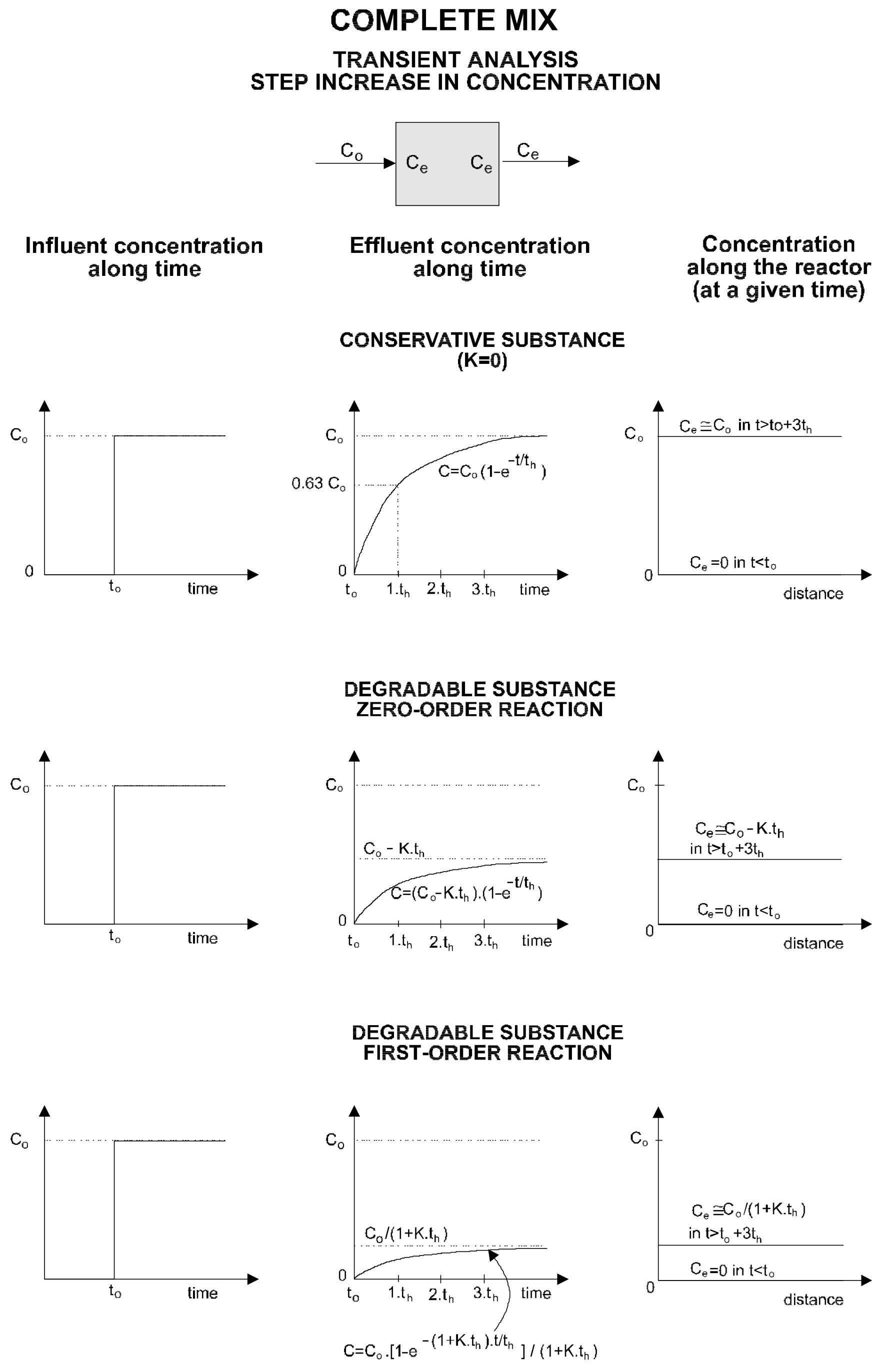


Figure 8.20. Transient analysis in a complete-mix reactor. Step increase in the concentration. (d = distance along the reactor).

8.4.12.3 Complete-mix reactor subjected to step variations in the influent concentration

In the cases that the influent concentration increases instantaneously to a new level (at which it stays), the behaviour of the complete-mix reactor is essentially different from the plug-flow reactor for any reaction order. This is due to the hydraulic characteristics of the complete-mix reactor, in which the influent substance is immediately dispersed in the tank, instantaneously appearing in the effluent. With the continuous arrival of the new higher influent concentration, the effluent concentration continues to increase until the transient conditions stop and a new level is reached. The effluent concentration remains at this new level, because a new steady state has been reached, until there is a new change in the influent characteristics.

Figure 8.20 shows the behaviour of a complete-mix reactor subjected to a step increase in influent concentration of conservative constituents and constituents that decay according to 0 and 1st order kinetics. The equations presented for the transients are asymptotic with relation to the new equilibrium value. Thus, in strict mathematical terms, a new equilibrium value will never be reached. For conservative substances, the utilisation of the equation $C = C_0.(1 - e^{-t/th})$ leads to the following values of the ratio C/C_0 (remaining concentration / influent concentration).

t/t_h	1.0	2.0	3.0	4.0	5.0
C/C_0	0.63	0.86	0.95	0.98	0.99

It can be observed that, after a time corresponding to the hydraulic detention time, the concentration of a conservative substance is 63% of the new equilibrium concentration (which is equal to the new influent concentration in the case of conservative substances). After a time equal to three times the hydraulic detention time, the concentration is equal to 95% of the equilibrium concentration. Hence, in practical terms, it can be considered that after a period greater than $3t_h$, a new equilibrium concentration will be reached. In any of the three equations (conservative substances, zero and first order reactions) presented in Figure 8.20 for the step function, when the time tends to infinity, the equations are converted into the steady-state form (presented in Table 8.4).

9

Conversion processes of organic and inorganic matter

9.1 CHARACTERISATION OF SUBSTRATE AND SOLIDS

9.1.1 Introduction

In sewage treatment, there is an interaction between various mechanisms, some occurring simultaneously, and others sequentially. The microbial action starts in the sewerage system and reaches its maximum in the sewage treatment works. In treatment plants, the conversion of organic matter to more oxidised or reduced forms takes place. Under aerobic conditions there is the oxidation of the organic matter (*carbonaceous matter*), that is, the organic carbon is converted into its most oxidised form (CO_2: carbon in the oxidation state of 4+). Under anaerobic conditions, the conversion reaction of the organic matter leads to the most oxidised form of carbon (CO_2), but also to its most reduced form (CH_4: carbon with an oxidation state of 4−). In sewage treatment under aerobic conditions, the conversion of ammonia (*nitrogenous matter*) into more oxidised forms of nitrogen ($NO_3{}^-$) can take place, and, under anoxic conditions, the subsequent conversion of these to reduced forms (N_2) can also happen. Biological wastewater treatment therefore includes oxidation (increase of the oxidation state) and reduction (decrease in the oxidation state) reactions.

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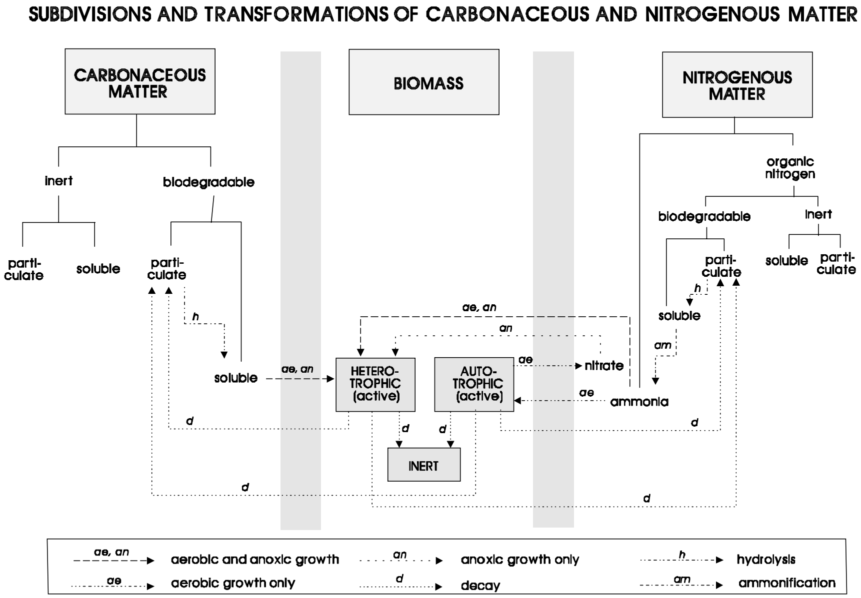


Figure 9.1. Subdivisions and transformations of carbonaceous and nitrogenous matter

The main transformation phenomena, together with the participation of the biomass, are presented in a schematic form in Figure 9.1 (interpreted from the mathematical model of the IAWPRC, 1987). It can be observed that there is a high complexity in the interrelation of the various compounds and biomass. The following items describe the main concepts and mechanisms related with the interaction of the biomass (central area of the figure) with the *carbonaceous* and *nitrogenous* (side areas) matter.

9.1.2 Characterisation of the carbonaceous matter

The carbonaceous matter (based on organic carbon) present in the wastewater to be treated can be divided in terms of biodegradability into (a) *inert* or (b) *biodegradable*.

- The **inert organic matter** (non-biodegradable) passes through the treatment system without changing its form. Two fractions can be identified with respect to the physical state:
  - *Soluble*. The non-biodegradable soluble organic matter does not undergo transformations and leaves the system with the same concentration that it entered.
  - *Particulate*. The non-biodegradable particulate organic matter (suspended) is involved by the biomass and is removed together with the

  sludge (excess sludge or the sludge that settles at the bottom of the reactors).
- The **biodegradable organic matter** is changed in its passage through the system. Two fractions can be identified, related to the biodegradability, which is also dependent on the physical state:
  - *Rapidly biodegradable*. This fraction is usually in a *soluble* form and consists of relatively simple molecules. These molecules can be directly used by the heterotrophic bacteria.
  - *Slowly biodegradable*. This fraction is usually in a *particulate* form, although slowly-biodegradable soluble organic matter may be present. The slowly-biodegradable matter consists of relatively complex molecules that are not directly used by the bacteria. For this to occur, the conversion into soluble matter is necessary, through the action of extracellular enzymes. This conversion mechanism, called *hydrolysis*, does not involve the use of energy, but results in the delay in the consumption of the organic matter (see Chapter 7).

9.1.3 Characterisation of the nitrogenous matter

The first major division in the nitrogenous matter that enters a sewage treatment works is by its organic state: the nitrogenous matter may be (a) *inorganic* or (b) *organic*.

- The **inorganic nitrogen** is represented by *ammonia*, either in its free form (NH_3) or in its ionised form (NH_4^+). Ammonia is present in the influent sewage because the hydrolysis and ammonification reactions, described below, have already started in the sewerage system.
- The **organic nitrogen** is divided in a similar form to the carbonaceous matter, as a function of the biodegradability: (a) *inert* and (b) *biodegradable*.
  - *Inert*. The inert fraction is divided into two fractions, according to the physical state:
    - *Soluble*. This part is usually negligible and does not need to be considered.
    - *Particulate*. This part is associated with the non-biodegradable carbonaceous organic matter, being involved by the biomass and removed with the excess sludge.
  - *Biodegradable*. The biodegradable fraction can be subdivided into the following three components:
    - *Rapidly biodegradable*. The rapidly-biodegradable organic nitrogenous matter is in a *soluble* form and is converted by heterotrophic bacteria into ammonia, through the process of *ammonification*.
    - *Slowly biodegradable*. The slowly-biodegradable organic nitrogenous matter is in a *particulate* form, being converted into a soluble

  form (rapidly biodegradable) through *hydrolysis*. This hydrolysis occurs in parallel with the hydrolysis of the carbonaceous matter.
- *Ammonia*. Ammonia (inorganic nitrogen) results from the hydrolysis and ammonification processes described above. Ammonia is used by heterotrophic and autotrophic bacteria.

9.1.4 Participation of the biomass

From the above, it is assumed that (a) the *carbonaceous matter* used directly by the bacteria is present in a *soluble* form (rapidly biodegradable), and that (b) the *nitrogenous matter* used directly by the bacteria will be present in the form of *ammonia*.

The biomass present in biological treatment systems can be divided into the following groups, as a function of its *viability*: (a) *active biomass* and (b) *inert residue*.

- The **inert residue** is formed through the decay of the biomass involved in the sewage treatment. The biomass decay can take place according to various mechanisms, which include endogenous metabolism, death, predation and others. As a result, slowly-degradable products are generated, as well as particulated products, inert to biological attack.
- The **active biomass** is that responsible for the biological degradation of the compounds. Depending on the carbon source, the biomass can be divided into (a) *heterotrophic* and (b) *autotrophic* (see Chapter 7):
  - *Active heterotrophic biomass*. The carbon source for the heterotrophic organisms is the carbonaceous organic matter. The heterotrophic biomass uses the rapidly biodegradable carbonaceous matter (soluble). Part of the energy associated with these molecules is incorporated into the biomass, while the remainder is used to supply energy for synthesis. In aerobic treatment, the growth of the heterotrophic biomass is possible in aerobic (using oxygen as an electron acceptor – see Chapter 7) or anoxic (absence of oxygen, by using the nitrate as an electron acceptor) conditions, but is very low in anaerobic conditions (absence of oxygen or nitrate). The heterotrophic bacteria use nitrogen in the form of ammonia for synthesis (in aerobic and anoxic conditions) and nitrogen in the form of nitrate as an electron acceptor (in anoxic conditions). The decay of the heterotrophic biomass generates, besides the inert residue, slowly degradable carbonaceous and nitrogenous matter. These subsequently need to undergo the hydrolysis process, to be converted into rapidly-biodegradable matter, which can be used again by the heterotrophic and autotrophic biomass.

- *Active autotrophic biomass*. The carbon source for the autotrophic organisms is carbon dioxide. The autotrophic biomass uses ammonia as the energy source (they are chemoautotrophic organisms, that is, they use inorganic matter as energy source). In aerobic conditions, these bacteria use ammonia in the nitrification process, in which ammonia is converted into nitrite and then nitrate. Similarly to the heterotrophic organisms, the decay of the autotrophic biomass also generates, besides an inert residue, slowly-degradable carbonaceous and nitrogenous matter. These subsequently need to undergo hydrolysis, to be converted to rapidly-biodegradable matter, which can be used again by the heterotrophic and autotrophic biomass.

9.1.5 Representation of the biomass and the substrate

9.1.5.1 Representation of the biomass

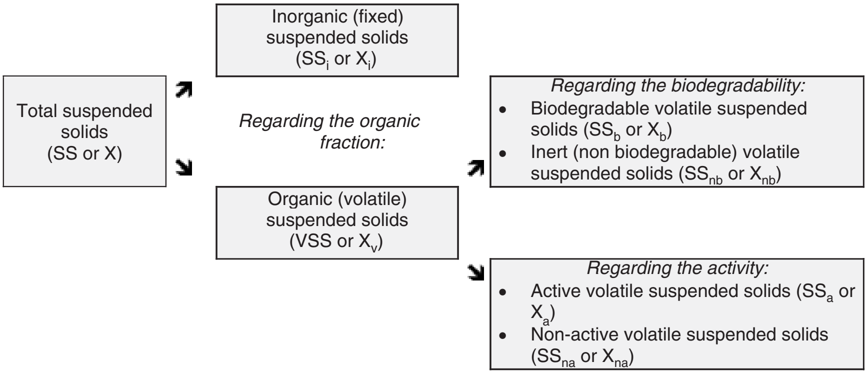
Due to the difficulty in characterising the biological solids and the substrate according to the above concepts, most of the mathematical models introduce simplifications in their representation. Such simplified representations are described in the present section.

The unit of mass of the microbial cells is normally expressed in terms of *suspended solids* (SS), since the biomass consists of solids that are suspended in the reactor (in the case of dispersed growth). However, not all the solids mass participates in the conversion of the organic substrate, as there is an inorganic fraction that does not play an active role in biological treatment. Therefore, the biomass is frequently expressed in terms of *volatile suspended solids* (VSS). These represent the organic fraction of the biomass – the organic matter can be volatised, that is, converted into gas by combustion (oxidation).

However, as mentioned, not all the organic fraction of the biomass is really active (Eckenfelder, 1980; Marais and Ekama, 1976; Grady and Lim, 1980; IAWPRC, 1987). Thus, the volatile suspended solids can be divided into an *active* and an *inactive* fraction. The active fraction is that which has the real participation in the conversion of the substrate. The main limitation of the use of the active solids in the design and operational control of a treatment plant relates to the difficulty in their determination. There are indirect processes, based on DNA, ATP, proteins, and others, but none compares to the simplicity of the direct determination of volatile suspended solids.

Besides considering the biomass activity, the solids can also be interpreted with relation to their biodegradability. Not all the volatile suspended solids are biodegradable, and there is a *biodegradable* and a *non-biodegradable* fraction.

In summary, the following distribution is frequently adopted for the suspended solids in a reactor:



9.1.5.2 *Representation of the organic matter*

As mentioned, the organic matter can be considered as having a *soluble* fraction, corresponding to the dissolved organic solids (most being rapidly biodegradable), and a *suspended* or *particulate* fraction, relative to the suspended solids (slowly biodegradable). In terms of carbonaceous matter, the present text adopts BOD_5 or COD as variables representing the substrate. In order to make the text more applicable, the treatment processes that have been traditionally designed using BOD_5 maintain it as the basic variable. On the other hand, the more recent processes that have been using COD for design are also described in terms of COD.

As a result, the following variables are adopted in terms of the representation of the substrate (carbonaceous matter):

- *Influent substrate S_o (influent BOD_5 or COD)*. Represents the *total* **BOD_5** (soluble BOD + particulate BOD) or *total* **COD** (soluble COD + particulate COD) influent to the biological reactor.

  Even in systems with primary sedimentation, around 1/3 of the suspended solids are not removed in this stage and enter the biological reactor. In the reactor, suspended solids are adsorbed by the biomass and are converted into soluble solids by hydrolysis mechanisms, after which they undergo the conversion reactions. Therefore, in the influent to the reactor, the soluble substrate as well as the particulate substrate must be computed as the influent substrate to be removed.

- Effluent substrate *S (effluent BOD_5 or COD)*. Represents the effluent *soluble* **BOD_5** or *soluble* **COD** from the reactor.

  Even though the effluent from the reactor could contain a high concentration of suspended solids (biological solids that compose the biomass), these solids are largely removed in the subsequent settling stage, when existent (e.g. secondary sedimentation tank or sedimentation lagoons). In the design of a reactor that receives recycled solids, there is no point in computing the effluent *total BOD* or *COD* from it, because it can be occasionally larger than the influent BOD or COD, due to the high concentration of particulate organic matter represented by the microbial population. The quality of the final effluent from the treatment plant depends on the (a) *soluble BOD or COD:* reactor performance; (b) *particulate BOD or COD:* performance of the final settling unit (when existent) or the concentration of the effluent solids from the reactor (when there is no final settling unit).

9.2 CONVERSION PROCESSES OF THE CARBONACEOUS AND NITROGENOUS MATTERS

9.2.1 Conversion of the carbonaceous matter

9.2.1.1 Aerobic conversion

The general equation of aerobic respiration can be expressed as:

$$\boxed{\underset{\text{organic matter}}{C_6H_{12}O_6} + 6\,O_2 \longrightarrow 6\,CO_2 + 6\,H_2O + \text{Energy}} \tag{9.1}$$

This equation is general and simplified, since, in reality, there are various intermediate steps. The composition of the organic matter is simplified and, in this case, the molecular formula of the glucose is assumed as a representation of the carbonaceous organic matter. By analysing the reaction, the following aspects can be highlighted, all important in sewage treatment (Branco, 1976):

- Stabilisation of the organic matter (conversion to inert products, such as carbon dioxide and water).
- Utilisation of oxygen.
- Production of carbon dioxide.
- Release of energy.

Equation 9.1 can be expressed in a generic way by an organic compound with molecular formula $C_xH_yO_z$, which permits the calculation of the oxygen consumption and the production of carbon dioxide (van Haandel and Lettinga, 1994):

$$C_xH_yO_z + \frac{1}{4}(4x + y - 2z)\,O_2 \rightarrow x\,CO_2 + \frac{y}{2}H_2O \tag{9.2}$$

As mentioned, Equations 9.1 and 9.2 are generic, covering only the oxidation of the *carbonaceous* organic matter. Other elements (such as nitrogen, phosphorus, potassium, etc.) are frequently part of the composition of the organic matter, which is still able to undergo biochemical oxidation.

The main agents responsible for the aerobic stabilisation of the carbonaceous matter contained in the sewage are the decomposing organisms, which are mainly represented by aerobic and facultative heterotrophic bacteria.

9.2.1.2 *Anaerobic conversion*

The conversion of the carbonaceous matter under anaerobic conditions follows the equation below:

$$\boxed{\underset{\text{organic matter}}{C_6H_{12}O_6} \longrightarrow 3\,CH_4 + 3\,CO_2 + \text{Energy}} \qquad (9.3)$$

This equation is also general and simplified, and represents only the final product from the intermediate stages. The following aspects can be highlighted in the equation:

- Non-exclusivity of the oxidation. The carbon of CO_2 is present in its highest state of oxidation (+4). However, the opposite occurs with CH_4, in which the carbon is in its most reduced state (−4), subsequently being able to be oxidised (for example, by combustion – methane is inflammable).
- No utilisation of oxygen.
- Production of methane and carbon dioxide.
- Release of energy (less than in aerobic respiration).

The organic matter was only converted to a more oxidised form (CO_2) and another more reduced form (CH_4). However, most of the CH_4 is released to the gaseous phase, which then leads to an effective removal of the organic matter.

Equation 9.3 can be expressed in a generic way for an organic compound $C_xH_yO_z$ as (van Haandel and Lettinga, 1994):

$$C_xH_yO_z + \frac{4x - y - 2z}{4}H_2O \rightarrow \frac{4x - y + 2z}{8}CO_2 + \frac{4x + y - 2z}{8}CH_4 \qquad (9.4)$$

The anaerobic conversion occurs in two stages:

- **Acidogenic phase**: conversion of the organic matter into organic acids by *acidogenic* organisms (acid-forming organisms). In this stage, there is only the conversion of organic matter, but no removal.
- **Methanogenic phase**: conversion of the organic acids into methane, carbon dioxide and water by methanogenic organisms (methane-forming organisms). The organic matter is converted again, but because CH_4 is transferred to the atmosphere, there is the removal of the organic matter.

Before the acidogenesis stage, the complex organic compounds (carbohydrates, proteins, and lipids) need to be converted into simple organic compounds, through the mechanism of **hydrolysis**.

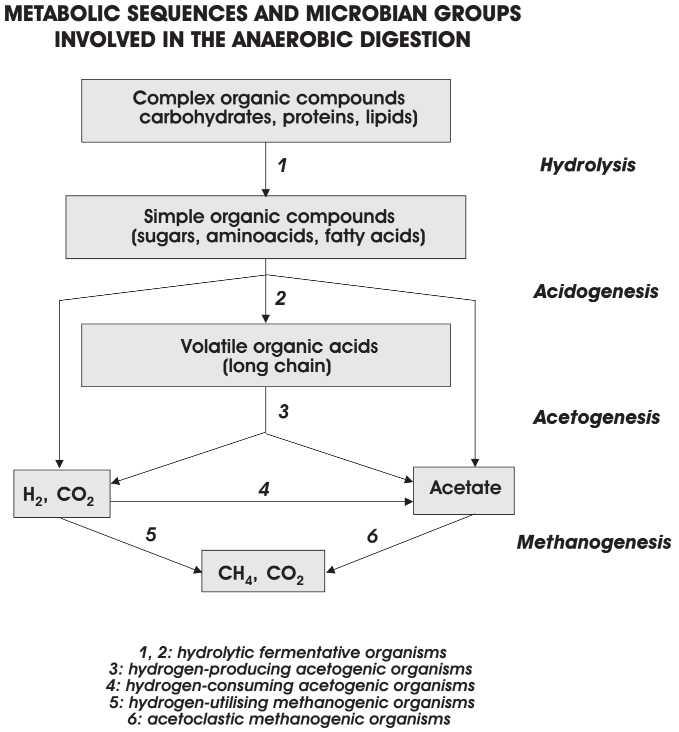


Figure 9.2. Metabolic sequences and microbial groups involved in anaerobic digestion (Chernicharo, 1995; Lubberding, 1995)

Figure 9.2 illustrates the sequence of stages involved in the anaerobic digestion of the organic matter.

Further details about the anaerobic conversion processes can be found in Chapter 24, relative to the anaerobic systems.

9.2.2 Conversion of nitrogenous matter

9.2.2.1 Oxidation of ammonia (nitrification)

In domestic sewage, the organic nitrogen is converted into ammonia, through the process of **ammonification**. This process does not change the quantity of nitrogen (TKN) in the wastewater, has no consumption of oxygen, and starts in the sewerage system itself, continuing in the primary and biological treatment units. In the end of the treatment, the quantity of organic nitrogen is small.

An important oxidation reaction that occurs in some wastewater treatment processes is the **nitrification**, in which the ammonia is transformed into nitrites and these nitrites into nitrates. Only some treatment processes are able to support a significant nitrification, because of their capacity of maintaining sufficient concentrations of the nitrifying bacteria.

The microorganisms involved in these processes are chemoautotrophs, for which carbon dioxide is the main source of carbon, and the energy is obtained through the oxidation of an inorganic substrate, such as ammonia, to mineralised forms.

The transformation of ammonia into nitrites is done by bacteria, such as those from the genus *Nitrosomonas*, according to the following reaction:

$$\boxed{2NH_4{}^+ + 3O_2 \xrightarrow{\textit{Nitrosomonas}} 2NO_2{}^- + 4H^+ + 2H_2O} \tag{9.5}$$

The oxidation of the nitrites to nitrates occurs mainly by the action of bacteria, such as those from the genus *Nitrobacter*, according to:

$$\boxed{2\,NO_2{}^- + O_2 \xrightarrow{\textit{Nitrobacter}} 2\,NO_3{}^-} \tag{9.6}$$

The global reaction of nitrification is the sum of Equations 9.5 and 9.6:

$$\boxed{NH_4{}^+ + 2\,O_2 \longrightarrow NO_3{}^- + 2\,H^+ + H_2O} \tag{9.7}$$

In the reactions 9.5 and 9.6 (as well as in the global reaction 9.7), the following points should be noted:

- Consumption of free oxygen. This consumption is called *nitrogenous demand.*
- Release of H^+, consuming the alkalinity of the medium and possibly reducing the pH.

Figure 9.3 shows a typical distribution of the nitrogen compounds in a treatment system after ammonification and the subsequent nitrification. The oxidised forms of nitrogen (nitrites and nitrates) are collectively called NO_x. It is seen that with nitrification there is *no removal of nitrogen* (total nitrogen remains the same), but only conversion of the nitrogen forms.

9.2.2.2 Reduction of nitrate (denitrification)

In anoxic conditions (absence of oxygen, but in the presence of nitrates), the nitrates are used by heterotrophic organisms as an electron acceptor instead of oxygen. In this process, called **denitrification**, *nitrate* is reduced to *nitrogen gas*, according to the following reaction:

$$\boxed{2\,NO_3{}^- + 2\,H^+ \longrightarrow N_2 + 2.5\,O_2 + H_2O} \tag{9.8}$$

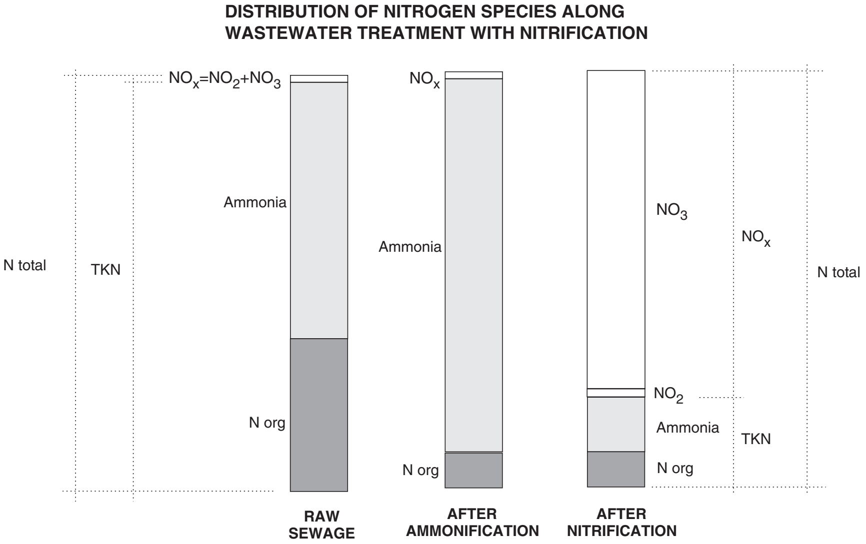


Figure 9.3. Distribution of nitrogen in a treatment system with nitrification

With the denitrification reaction, the following points should be noted:

- Economy of oxygen (the organic matter can be stabilised in the absence of oxygen)
- Consumption of H^+, implying an economy of the alkalinity and an increase in the buffer capacity of the medium

Figure 9.4 shows a typical distribution of the nitrogen forms in a treatment system with nitrification and denitrification. It is seen that, besides the conversion in the forms of nitrogen, there is also the *removal of nitrogen* (total nitrogen is decreased). In other words, denitrification leads to an effective removal of nitrogen from the liquid, corresponding to the nitrate that is converted to nitrogen gas, which escapes to the atmosphere.

9.3 TIME PROGRESS OF THE BIOCHEMICAL OXIDATION OF THE CARBONACEOUS MATTER

In a simplified manner and neglecting intermediate mechanisms, it can be said that the *aerobic* reactions for the stabilisation of the organic matter occur, in a closed system (such as the bottle used in the BOD test), in a sequence in which the two following main mechanisms are predominant (Eckenfelder, 1980):

- *Initial stage: predominance of synthesis (anabolism)*

  At the beginning, the organic matter present in the wastewater is used by the microorganisms for their metabolic activities of growth and energy

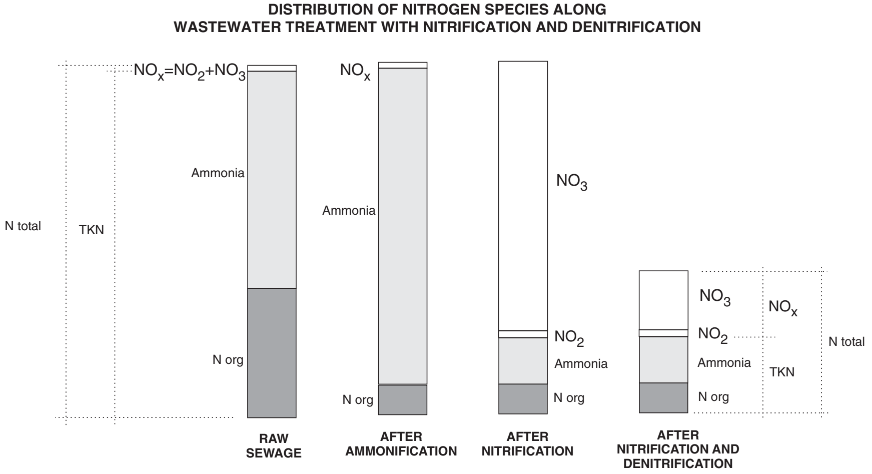


Figure 9.4. Distribution of nitrogen in a treatment system with nitrification and denitrification

conversion, therefore prevailing the activities related to *synthesis*. This phase results in oxygen consumption and in the increase in the microorganisms population, and can be represented by the generic equation (Hanisch, 1980):

$$\boxed{\underset{\substack{\text{organic}\\\text{matter}}}{8\,CH_2O} + NH_3 + 3\,O_2 \longrightarrow \underset{\substack{\text{cellular}\\\text{matter}}}{C_5H_7NO_2} + 3\,CO_2 + 6\,H_2O + \text{Energy}} \qquad (9.9)$$

In this equation, CH_2O represents the organic matter, in the same way that the equivalent formulation $C_6H_{12}O_6$ was also used to represent organic matter in Item 9.2.1. The cellular matter is expressed by the following empirical formula $C_5H_7NO_2$ (see Section 7.7.2).

- *Subsequent stage: predominance of endogenous respiration (catabolism)*

When the organic matter originally present in the wastewater is mostly removed, there is the predominance of the mechanisms of the second phase of oxidation. At the start of this phase, the microorganisms' population is at their maximum and, due to the low availability of substrate in the medium, the main food source becomes their own cellular protoplasm. Therefore, in this phase there is the predominance of auto-oxidation mechanisms or *endogenous respiration* (see Section 9.4, about bacterial growth). The simplified representative equation of this phase is:

$$\boxed{\underset{\text{cellular matter}}{C_5H_7NO_2} + 5\,O_2 \longrightarrow 5\,CO_2 + NH_3 + 2\,H_2O + \text{Energy}} \qquad (9.10)$$

Naturally in a system with mixed cultures, such as the reactors used in biological wastewater treatment, there are microorganisms with different growth and decay rates. Consequently, some microorganism species can be in one stage of synthesis or endogenous respiration, while other species are in earlier or later phases. The representation above regards only average conditions for heterotrophic microorganisms in the reactor.

The total oxygen consumed in both phases is defined as the *ultimate oxygen demand* (BOD_u). The addition of equations 9.9 and 9.10 leads to the simplified equation for the oxidation of the organic matter (identical to Equation 9.1):

$$\boxed{CH_2O + O_2 \longrightarrow CO_2 + H_2O + \text{Energy}} \qquad (9.11)$$

The value of the theoretical oxygen demand (ThOD) based on the stoichiometric relations of Equation 9.11 differs, in a certain way, from what is found for the ultimate demand, being, in reality, a little higher. This is because in the endogenous phase, when a bacteria dies, it becomes food for other bacteria, and thus a subsequent transformation to carbon dioxide, water and cellular material occurs. The living as well as the dead bacteria serve as food for higher organisms, such as protozoa. In each transformation, a new oxidation occurs, but in the general balance, a certain fraction of the organic matter, resistant to biological attack, remains. This fraction is the one responsible for the deviation between the values of the theoretical and the ultimate oxygen demand (Sawyer and Mc Carty, 1978).

The removal and oxidation of the organic matter present in the wastewater (first phase) normally has a duration of one to two days. The total oxidation of the cellular mass will take a very long time, but, in practical terms for domestic sewage, it can be considered complete around 20 days. The reaction rate in the assimilation phase is several times higher than in the second phase, of endogenous respiration (Eckenfelder, 1980).

Figure 9.5 presents the curves of the accumulated oxygen consumption, substrate concentration and bacteria concentration as a function of time

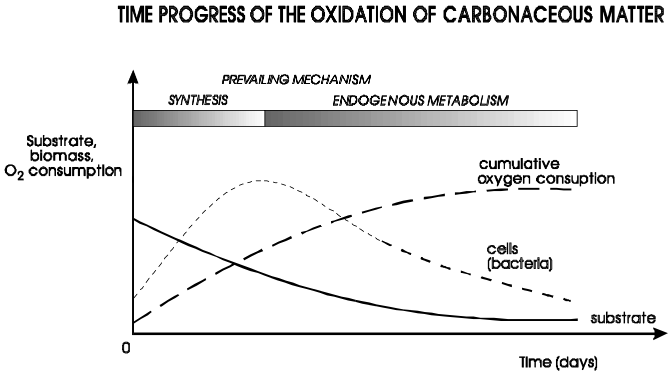


Figure 9.5. Oxidation of the carbonaceous matter along time

9.4 PRINCIPLES OF BACTERIAL GROWTH

9.4.1 Synthesis and endogenous respiration

As already mentioned, heterotrophic organisms use organic matter as a form of condensed energy that is necessary for their various metabolic processes, including growth and reproduction. With the use of oxygen (aerobic conditions – Equation 9.1) or another electron acceptor (e.g. nitrate, in anoxic conditions – Equation 9.8), these organisms oxidise the organic matter, with the production of more cellular matter (growth and reproduction) and energy release. This is the **synthesis** phase.

If the substrate available in the medium starts to become scarce, such as in sewage treatment, in which organic matter is progressively removed, the microorganisms need to find other organic matter or condensed energy sources. The main substrate directly available is their own cellular protoplasm, which the cells start to use according to Equation 9.10. In this stage, the balance is negative, that is, there is a reduction in the cellular matter or the bacterial concentration in the medium, characterising the **endogenous respiration** stage.

It is thus seen that there is a close relation between the substrate concentration in the medium, or the available food, and the microorganisms' population. When the availability of organic matter is sufficient, the bacteria are in a growing phase, and when it becomes insufficient, the bacteria enter a decreasing stage. This consideration is of large importance in sewage treatment, in which systems can be designed to operate with a high or low organic matter supply for the bacteria.

Besides, the form in which the two phases are located in the biological reactor depends on its hydraulic configuration (see Chapter 8). In a *plug-flow* reactor, the reaction time is associated with the physical location in the reactor. Hence, the sequencing between the two phases can take place along the inlet and outlet of the reactor. In a *complete-mix* reactor, the concentration of the substrate and the bacteria are the same at any point in the reactor. Thus, the relative predominance of one phase or another will depend on the prevalent concentration of the substrate in the reactor. If it is high, the synthesis phase prevails in the reactor as a whole. However, if the substrate concentration is low, the balance favours the mechanisms of endogenous respiration.

A simplified scheme of the heterotrophic bacterial metabolism is presented in Figure 9.6.

9.4.2 Bacterial-growth curve

The main reproduction mode for bacteria is by binary fission, in which, when the cell reaches a certain size, it splits into two cells, which will subsequently generate four new cells and so on. Thus, after n divisions the number of cells formed is 2^{n}. Assuming a typical generation time of 20 minutes, a population growth without limiting factors could lead to 2^{144} bacteria after 48 hours. This would correspond to a weight approximately 4000 times greater than the weight of the earth (La Riviére,

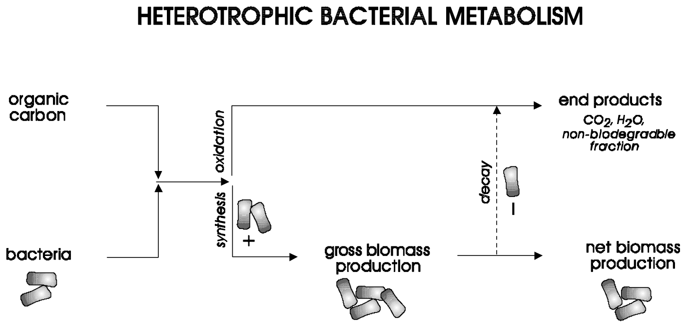


Figure 9.6. A simplified scheme of the heterotrophic bacterial metabolism

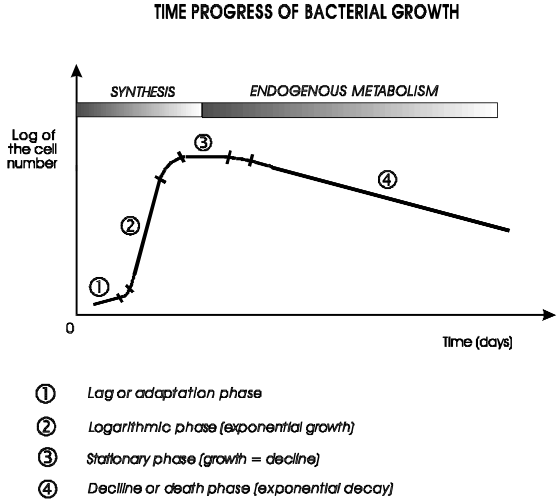


Figure 9.7. Typical bacterial-growth curve

1980). Naturally, in practice the growth is soon restricted due to the exhaustion of the nutrients in the medium.

When inoculating a liquid volume with a certain initial quantity of bacterial cells and a limited quantity of substrate, the number of bacteria will progress with time according to the typical bacterial growth-curve, expressed in Figure 9.7 (vertical logarithmic scale).

- **Lag phase**. The lag phase is a period for enzymatic adaptation of the bacteria to the new substrate supplied (von Sperling, 1983a). This phase can be reduced in the case of typical domestic sewage, in which the bacteria have already acquired the necessary enzymatic equipment.

- **Exponential-growth phase**. In the exponential growth phase the cells divide themselves at a constant rate. Plotted on a logarithmic scale, the number of cells grows linearly, justifying the alternative designation of *logarithmic phase*. There is an excess of the substrate in the medium, allowing the growth rate to reach its maximum, with the only limitation by the microorganisms' capacity to process the substrate. In parallel with the maximum growth rate, there is also the maximum substrate removal rate.
- **Stationary phase**. The stationary phase is when the food starts to be scarce in the medium, and the bacterial growth rate is equal to the death rate. Therefore, the number of cells is maintained temporally constant.
- **Decline or decay phase**.In the decline or decay phase, the availability of the substrate in the medium is reduced. In these conditions, *endogenous respiration* prevails, and the bacteria are forced to use their own cellular protoplasm as a substrate source. The dying cells allow their nutrients to diffuse into the medium, serving as food to other cells. The death rate is exponential and constant, leading to a straight line on the logarithmic scale.

As already mentioned, it is important to emphasise that this representation of the growth regards a *single* population of microorganisms growing at the expense of a *single* type of substrate. In reality, in the biological reactor of a sewage treatment works, there is a variety of microorganisms metabolising a variety of compounds. Hence, there will be an overlapping of various curves of different forms and types, developing at different times. This interaction characterises the ecology of wastewater treatment, covered in Chapter 7.

The design and operation of a sewage treatment plant uses these concepts of bacterial growth to place the operation inside a desired range. A generalisation is difficult due to the large variety of microorganisms and substrates that occur in practice, but the following tendencies can be observed:

- **Very high loading systems**. In the exponential growth phase, the substrate availability is high. This indicates that the concentration of the substrate (e.g. BOD) in the effluent will also be somewhat high. Thus, the majority of sewage treatment systems do not operate in this phase.
- **High loading systems** (e.g. conventional activated sludge, high rate trickling filters). The concentration of substrate in the effluent is lower, but the cellular mass has a high organic fraction, requiring the separate stabilisation of the excess sludge. Due to the high load, the volume required for the reactor is smaller than in the low loading systems.
- **Low loading systems** (e.g. activated sludge systems of the extended aeration variant, low rate trickling filters). The reasoning of these systems is to supply a minimum quantity of substrate to the organisms, in order to stimulate endogenous respiration. This leads to a self metabolism of the microorganisms, that is, they undergo a digestion of the cellular mass in the reactor itself. Besides the partial stabilisation of the cellular mass, the concentration of the substrate in the effluent is very low. The volume required for the reactor is larger than for the high loading systems.

9.4.3 Kinetics of bacterial growth

9.4.3.1 Specific gross bacterial growth

The bacterial growth can be expressed as a function of the bacteria concentration at a given time in the reactor. The *net* growth rate is equal to the gross growth rate minus the bacterial decay rate.

The growth rate of a bacterial population is a function of its number, mass or concentration at a given time. Mathematically, this relation can be expressed as:

$$\boxed{\frac{dX}{dt} = \mu.X} \tag{9.12}$$

where:

X = concentration of the microorganisms in the reactor, SS or VSS (g/m^3)
μ = specific growth rate (d^{-1})
t = time (d)

This formula, if integrated, assumes an exponential form, which, when plotted on a logarithmic scale, results in a straight line. This is the logarithmic phase shown in Figure 9.7.

The growth rate, such as expressed by Equation 9.12, is for a growth without limitation of substrate. However, it was seen in the previous sections that bacterial growth is a function of the availability of the substrate in the medium. When the substrate is present at a low concentration, the growth rate is proportionally low. In sewage treatment, the carbonaceous matter is usually the limiting growth factor.

The specific growth rate μ must be therefore expressed as a function of the substrate concentration. Monod, in his classic studies with bacterial cultures, presented this relation according to the following empirical formula:

$$\boxed{\mu = \mu_{max} \cdot \frac{S}{K_s + S}} \tag{9.13}$$

where:

μ_{max} = maximum specific growth rate (d^{-1})
S = concentration of the limiting substrate or nutrient (g/m^3)
K_s = half-saturation coefficient, which is defined as the substrate concentration for which $\mu = \mu_{max}/2$ (g/m^3)

Figure 9.8 shows a graphic representation of this equation. The *limiting nutrient* (S) is the one that, in case its concentration is reduced, will lead to a decrease in the population growth rate (as indicated in the figure through the reduction of μ). Conversely, if the concentration of S starts to increase, the population will consequently increase. However, if S continues to increase, there will be a point in

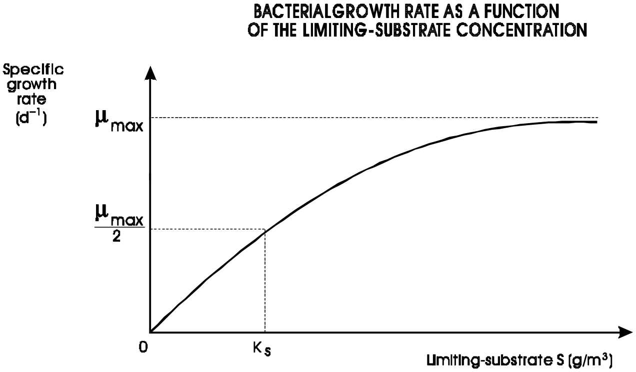


Figure 9.8. Specific growth rate as a function of the concentration of the limiting substrate

which it will be in excess in the medium, not being anymore the limiting factor for the population growth. In these conditions, another nutrient will probably start to control the growth, becoming the new limiting factor. This explains why μ tends to a maximum value, expressed by μ_{max}. At this point, even if the concentration S increases, μ will not increase further, since it is no longer limited by S. The basis of the Monod formulation are analysed in Chapter 7.

The interpretation of the half-saturation coefficient K_s is that, when the substrate concentration in the medium is equal to K_s ($K_s = S$), the term $S/(K_s + S)$ of Equation 9.13 becomes equal to $1/2$. Thus, the growth rate μ becomes equal to half the maximum rate ($\mu_{max}/2$). To compare different substrates, the value of K_s gives an indication of the *non-affinity* of the microorganisms for each substrate: the greater the value of K_s, the lower the growth rate μ or, else, the lower the affinity of the biomass to the substrate. To obtain high substrate removals in sewage treatment, it is desirable that the substrate has low values of K_s.

In the case of the heterotrophic bacteria involved in sewage treatment, the limiting substrate is usually organic carbon or, in other words, BOD or COD. This is because the reactors work with low organic carbon concentrations to produce an effluent with a low BOD concentration.

Under other conditions, there can be other limiting nutrients, or even a composition of two or more of them. This is the case of the growth rate of the nitrifying organisms. Due to the fact that their growth rate is small, either with low ammonia values as well as with low dissolved oxygen values, the Monod relation can be expressed as a double-inhibition function. Hence, instead of only one term $S/(K_s + S)$, there is the product of two terms $[S_1/(K_{s1} + S_1]$. $[S_2/(K_{s2} + S_2]$, in which S_1 and S_2 are the concentrations of the two limiting factors (in this case, ammonia and oxygen).

In the laboratory, the curve in Figure 9.8 (or a transformation of it) can be constructed, and the values of K_s and μ_{max} can be extracted. In domestic sewage treatment, values of K_s and μ_{max} in the following ranges have been reported:

- *Aerobic treatment (Metcalf & Eddy, 1991):*

$$\boxed{\mu_{max} = 1.2 \text{ to } 6\ d^{-1}}$$

$$\boxed{\begin{array}{l} K_s = 25 \text{ to } 100\ \text{mg}\,BOD_5/l \\ \text{or} \\ K_s = 15 \text{ to } 70\ \text{mgCOD}/l \end{array}}$$

- *Anaerobic treatment (van Haandel and Lettinga, 1994; Chernicharo, 1997):*

$$\boxed{\begin{array}{l} \mu_{max} = 2.0\ d^{-1} \text{ (acidogenic organisms)} \\ \mu_{max} = 0.4\ d^{-1} \text{ (methanogenic organisms)} \\ \mu_{max} = 0.4\ d^{-1} \text{ (combined biomass)} \end{array}}$$

$$\boxed{\begin{array}{l} K_s \approx 200\ \text{mgCOD}/l \text{ (acidogenic organisms)} \\ K_s \approx 50\ \text{mgCOD}/l \text{ (methanogenic organisms)} \end{array}}$$

Certain types of organisms could of course have different coefficients from these global values.

The Monod equation has the same form as the Michaelis–Menten equation for enzymatic relations (see Chapter 7). However, while the latter is based on theoretical principles, the Monod relation is essentially empirical. Another aspect is that the Monod equation was derived for a single organism metabolising a single substrate. However, in wastewater treatment this assumption is not valid, since there is a *multiple population* assimilating a *multiple substrate*. Due to these aspects, the Monod relation has been the target of criticism from the specialised literature. However, a more satisfactory relation has not yet been developed, and the Monod equation maintains its importance, being adopted in practically all mathematical models of biological wastewater treatment.

A great advantage of the Monod equation resides in its structure that permits the representation in a continuous form of the range of variation between the extremes of lack and abundance of nutrients in the medium. Therefore, depending on the value of S, the Monod equation can represent approximately the kinetics of zero and first orders, as well as the transition between them. In the case of a substrate removal reaction, when its concentration is still high and not limiting, the global removal rate approaches the *zero-order* kinetics. With the consumption of the substrate, the reaction starts to decrease, characterising a transition or mixed-order region. When the substrate concentration is very low, the reaction rate starts to be limited by its low availability in the medium. In these conditions, the reaction kinetics approach *first order*. These two situations occur depending on the relative values of S and K_s, as described below:

- Relative substrate concentration: **high**

$S >> K_s$	• *reaction approximately zero order* • *growth rate μ independent of S*

When the substrate concentration is much higher than the value of K_s, K_s can be neglected in the denominator of Equation 9.13, which is reduced to:

$$\mu = \mu_{max} \tag{9.14}$$

In these conditions, the growth rate μ is constant and equal to the maximum rate μ_{max}. The reaction follows a *zero-order kinetics*, in which the reaction rate is independent from the substrate concentration. In domestic sewage treatment, this situation tends to occur at the head of a plug-flow reactor, where the substrate concentration is still high.

- Relative substrate concentration: **low**

$S << K_s$	• *reaction approximately first order* • *growth rate μ is dependent on S (directly proportional)*

When the substrate concentration is much lower than the value of K_s, S can be neglected in the denominator of Equation 9.13, which is reduced to:

$$\mu = \mu_{max} \cdot \frac{S}{K_s} \tag{9.15}$$

As μ_{max} and K_s are constants, the term (μ_{max}/K_s) is also a constant, and can be substituted by a new constant K. Consequently, Equation 9.15 is reduced to:

$$\mu = K.S \tag{9.16}$$

In this situation, the growth rate is proportional to the substrate concentration. The reaction follows *first-order kinetics*. This situation is typical in the treatment of domestic sewage in a complete-mix reactor in which the substrate concentration in the medium is low due to the requirements of low values in the effluent.

Figure 9.9 presents the two extreme situations that represent the zero and first order kinetics.

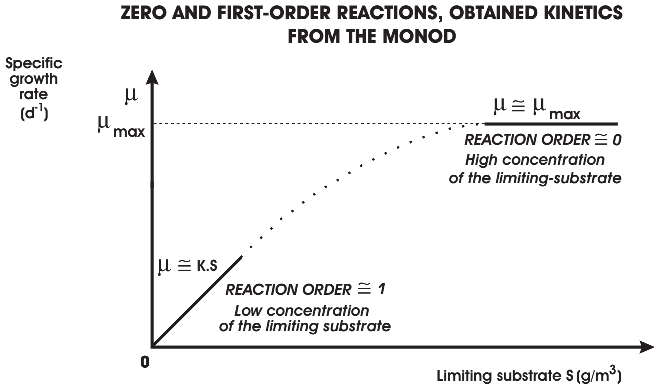


Figure 9.9. Extreme conditions in the saturation reaction (Monod kinetics)

Example 9.1

Express μ as a function of μ_{max} for the following conditions:

- domestic sewage; S = 300 mg/L (adopt K_s = 40 mg/L)
- domestic sewage; S = 10 mg/L (adopt K_s = 40 mg/L)
- glucose; S = 10 mg/L (adopt K_s = 0.2 mg/L)

Solution:

a) Domestic sewage (S = 300 mg/L)
From Equation 9.13:

$$\mu = \mu_{max} \cdot \frac{S}{K_s + S} = \mu_{max} \cdot \frac{300}{40 + 300} = 0.88\mu_{max}$$

Hence, $\mu = 0.88\ \mu_{max}$

In these conditions, in which S is large in comparison with K_s, the growth rate μ is close to μ_{max}. There is a great availability of the limiting nutrient and the population presents a high growth rate. The reaction is approximately zero order. This situation is not very frequent in the treatment of domestic sewage and occurs at the head of a plug-flow reactor, where the substrate concentration is still high.

b) Domestic sewage (S = 10 mg/L)

$$\mu = \mu_{max} \cdot \frac{S}{K_s + S} = \mu_{max} \cdot \frac{10}{40 + 10} = 0.20\ \mu_{max}$$

Thus, $\mu = 0.20\ \mu_{max}$

Example 9.1 (Continued)

As S is small in comparison with K_s, the growth rate is much lower than μ_{max}.This indicates that there is not much availability of the limiting nutrient in the medium. This situation is typical in a complete-mix reactor treating domestic sewage, in which the substrate is completely homogenised and is present at a concentration equal to the effluent one.

c) Glucose (S = 10 mg/L)

$$\mu = \mu_{max} \cdot \frac{S}{K_s + S} = \mu_{max} \cdot \frac{10}{0.2 + 10} = 0.98\,\mu_{max}$$

Hence, $\mu = 0.98\,\mu_{max}$

The concentration of S is the same as in item b, but since in the case of glucose K_s is very low (high affinity), the denominator of the expression is practically equal to S and, as a result, μ is almost equal to μ_{max}. Consequently, there is a high availability of the substrate and the growth rate is very close to the maximum.

9.4.3.2 Bacterial decay

The relations presented in the previous section correspond to the gross biomass growth. However, since the bacteria stay in the treatment systems for more than one or two days, there is also the endogenous metabolism stage. This implies that part of the cellular matter is destroyed by means of some of the mechanisms active in the endogenous respiration stage. To obtain the *net* growth rate, this loss should be discounted, which is also a function of the concentration or bacterial mass. For accuracy, only the *biodegradable fraction* of the biomass should be considered, since there is also an inert, non-biodegradable organic fraction, not subject to bacterial decay. For simplicity, the total VSS are considered in most of this chapter, and not the biodegradable VSS. In the chapters related to the activated sludge system (Part 5), this concept is deepened, and the concept of the biodegradable fraction is used.

The decay rate can be expressed as a first-order reaction:

$$\frac{dX}{dt} = -K_d.X \tag{9.17}$$

where:

K_d = endogenous respiration coefficient, or bacterial decay coefficient (d^{-1})

For typical domestic sewage, K_d varies in the following ranges:

- *Aerobic treatment:*

K_d = 0.04 to 0.10 mgVSS/mgVSS.d (base: BOD_5) (Metcalf & Eddy, 1991; von Sperling, 1997)
or
K_d = 0.05 to 0.12 mgVSS/mgVSS.d (base: COD) (EPA, 1993; Orhon and Artan, 1994)

- *Anaerobic treatment:*
  The values available in the literature appear to be not very reliable (Lettinga, 1995), although the value of 0.02 mgVSS/mgVSS.d (base: COD) has been cited by Lettinga et al (1996).

9.4.3.3 *Net bacterial growth*

The net growth is obtained by the sum of the Equations 9.12, 9.13 and 9.17:

$$\boxed{\frac{dX}{dt} = \mu.X - K_d.X} \tag{9.18}$$

or

$$\boxed{\frac{dX}{dt} = \mu_{max}\cdot\frac{S}{K_s + S}.X - K_d.X} \tag{9.19}$$

9.4.4 Production of biological solids

9.4.4.1 *Gross solids production*

Bacterial growth, that is, biomass production, can be also expressed as a function of the substrate used. The greater the substrate assimilation, the greater the bacterial growth rate. This relation can be expressed as:

Growth rate = Y (Substrate removal rate)

or

$$\frac{dX}{dt} = Y\frac{dS}{dt} \tag{9.20}$$

where:

X = concentration of microorganisms, SS or VSS (g/m^3)
Y = yield coefficient, or coefficient of biomass production; biomass (SS or VSS) produced per unit mass of substrate removed (BOD or COD) (g/g)
S = concentration of BOD$_5$ or COD in the reactor (g/m^3)
t = time (d)

Therefore, it can be observed that there is a linear relationship between the bacterial growth rate and the substrate utilisation rate, or the rate of BOD or COD removal.

The value of Y can be obtained in laboratory tests with the wastewater to be treated. For the biological treatment of domestic sewage, the Y value for the heterotrophic bacteria responsible for the removal of the carbonaceous matter varies between:

- *Aerobic treatment:*

$Y = 0.4$ to 0.8 g VSS/g BOD_5 removed (Metcalf & Eddy, 1991)
or
$Y = 0.3$ to 0.7 g VSS/g COD removed (EPA, 1993; Orhon and Artan, 1994)

- *Anaerobic treatment:*

$Y \approx 0.15$ gVSS/gCOD (*acidogenic bacteria*) (van Haandel and Lettinga, 1994)
$Y \approx 0.03$ gVSS/gCOD (*methanogenic archaea*) (van Haandel and Lettinga, 1994)
$Y \approx 0.18$ gVSS/gCOD (*combined biomass*) (Chernicharo, 1997)

Other systems can have different Y values. The anaerobic conversion of the organic substrate releases less energy and therefore the value of Y is lower, indicating a lower biomass production. The nitrifying bacteria (chemoautotrophs) do not extract their energy from the organic carbon, but from the oxidation of inorganic compounds. Thus, they also present lower Y values when compared with the aerobic heterotrophic organisms (Arceivala, 1981).

9.4.4.2 Net solids production

Equation 9.20 expresses the gross bacterial growth without taking into consideration the reduction of the biomass due to endogenous respiration. When including the endogenous respiration, the net solids production becomes:

$$\boxed{\frac{dX}{dt} = Y\frac{dS}{dt} - K_d.X} \qquad (9.21)$$

Example 9.2

Calculate the biological solids production in a treatment system, assuming steady state. Data:

- Reactor volume: $V = 9{,}000\ m^3$
- Hydraulic detention time: $t = 3$ d
- Influent substrate (total BOD_5): $S_o = 350$ mg/L
- Effluent substrate (soluble BOD_5): $S = 9.1$ mg/L
- Biomass in the reactor (VSS): $X_v = 173.3$ mg/L

Example 9.2 (Continued)

Coefficients of the model:

- Yield coefficient: $Y = 0.6$ mgVSS/mg BOD_5
- Endogenous respiration coefficient: $K_d = 0.06\ d^{-1}$

Solution:

Assuming finite time conditions within the steady-state hypothesis, Equation 9.21 can be rewritten as:

$$\frac{\Delta X_v}{\Delta t} = 0.6\frac{gVSS}{gBOD_5}.(350 - 9.1)\frac{gBOD_5}{m^3}.\frac{1}{3.0d} - 0.06\frac{gVSS}{gVSS.d}.173.3\frac{gVSS}{m^3}$$

$\Delta X_v/\Delta t = 68.2\ g/m^3.d - 10.4\ g/m^3.d = 57.8\ g/m^3.d = 0.058\ kg/m^3.d$

Since the reactor volume is 9,000 m^3, the global net production is:

$0.058\ kg/m^3.d \times 9{,}000\ m^3 = 522$ kgVSS/d

Therefore, the net production of biological solids in the system (expressed as VSS) as a function of the substrate utilisation is 522 kgVSS per day. In the calculations above, it can be seen that 68.2 g/m^3.d is the gross production and 10.4 g/m^3.d is the destruction by endogenous respiration. In this example, the net production is approximately 85% of the gross production.

In the example, numbers with decimals have been used only to clarify the calculations. In most practical applications, round figures are more frequently used for representing BOD and other variables.

9.4.4.3 *Substrate removal rate*

In a wastewater treatment system, it is also important to quantify the rate at which the substrate is removed. The greater the rate, the lower is the required volume for the reactor (for a certain concentration of the substrate) or the greater is the efficiency of the process (for a certain volume of the reactor).

Rearranging Equation 9.20, the substrate removal rate can be expressed as:

$$\frac{dS}{dt} = \frac{1}{Y}.\frac{dX}{dt} \tag{9.22}$$

The substrate removal is associated with the gross biomass growth. According with Equation 9.12, dX/dt = μ.X. Substituting dX/dt for μ.X in Equation 9.22, there is:

$$\boxed{\frac{dS}{dt} = \frac{\mu}{Y}.X} \tag{9.23}$$

or (expressing μ through Equation 9.13):

$$\boxed{\frac{dS}{dt} = \mu_{max} \cdot \frac{S}{K_s + S} \cdot \frac{X}{Y}} \tag{9.24}$$

9.5 MODELLING OF SUBSTRATE AND BIOMASS IN A COMPLETE-MIX REACTOR

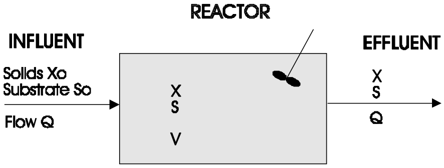
9.5.1 Mass balance in the reactor

The interactions that occur in a continuous-flow complete-mix reactor (homogenous concentration of biomass and substrate in all the reactor volume) without recirculation can be represented schematically as in Figure 9.10.

One of the characteristics of the ideal complete-mix reactor is that the effluent leaves with the same concentration as in the liquid in any part of the reactor. This implies that the values of S and X are the same in the reactor, as well as in the effluent.

X is the concentration of the solids. In the reactor, these solids are mainly biological solids, represented by the biomass (microorganisms) produced in the reactor at the expense of the available substrate. In contrast, in the influent to the reactor, the solids are those present in the wastewater, and the presence of biological solids is frequently neglected in the general mass balance. For simplicity, it is usually considered that $X_0 = 0$ mg/L (although this assumption does not apply in all situations).

Two mass balances can be done, one for the substrate and the other for the biomass. These mass balances are essential for design and operational control of the biological reactor, and are detailed in this section.



S_o = concentration of the total influent substrate (BOD or COD) (mg/L or g/m³)
S = concentration of the soluble effluent substrate (BOD or COD) (mg/L or g/m³)
Q = flow (m³/d)
X = concentration of the suspended solids in the reactor (mg/L or g/m³)
X_o = concentration of influent suspended solids (mg/L or g/m³)
V = reactor volume (m³)

Figure 9.10. Schematic representation of the mass balance in a complete-mix reactor (without recirculation)

The mass balance takes into consideration the *transport* (input and output) and the *reaction* (production and consumption) terms. The following equations are for a system composed by a single reactor, *without final settling and recirculation.*

Accumulation = Input − Output + Production − Consumption

- **Substrate balance:**

$$\boxed{\frac{dS}{dt} = \frac{Q}{V}.S_o - \frac{Q}{V}.S + 0 - \frac{\mu}{Y}.X} \tag{9.25}$$

where:

$$\boxed{\mu = \mu_{max}\cdot\frac{S}{K_s + S}} \tag{9.26}$$

or:

$$\frac{dS}{dt} = \frac{Q}{V}.S_o - \frac{Q}{V}.S + 0 - \mu_{max}\cdot\frac{S}{K_s + S}\cdot\frac{X}{Y} \tag{9.27}$$

- **Solids balance:**

$$\boxed{\frac{dX}{dt} = \frac{Q}{V}.X_o - \frac{Q}{V}.X + \mu.X - K_d.X} \tag{9.28}$$

or:

$$\frac{dX}{dt} = \frac{Q}{V}.X_o - \frac{Q}{V}.X + \mu_{max}\cdot\frac{S}{K_s + S}.X - K_d.X \tag{9.29}$$

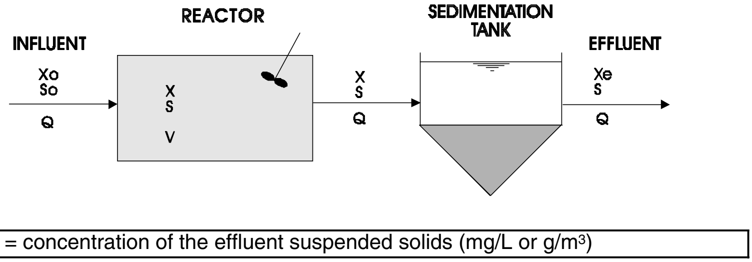
9.5.2 Systems with and without solids recirculation

9.5.2.1 Introduction

There are three possible combinations of reactors with dispersed-growth biomass, continuous flow and complete-mix hydraulic regime:

- Reactor without a final sedimentation unit and hence without recirculation of solids
- Reactor with a final sedimentation unit and without recirculation of solids
- Reactor with a final sedimentation unit and with recirculation of solids

The system composed of a reactor without a final sedimentation unit and without recirculation of solids was seen in Section 9.5.1. The other systems are covered in the present section.



X_e = concentration of the effluent suspended solids (mg/L or g/m^3)

Figure 9.11. Biological reactor followed by a settling unit (without sludge recirculation)

9.5.2.2 Reactor with a final sedimentation unit and without solids recirculation

When analysing Figure 9.10, it can be observed that the biological solids formed are present in the effluent in the same concentration as in the reactor. These solids are, ultimately, largely composed by organic matter and, if discharged into the receiving body, would undergo stabilisation similarly to the other forms of organic matter. Therefore, even that the soluble BOD may have undergone a substantial reduction in the reactor, the particulate BOD represented by the biological solids in the effluent can be responsible for the deterioration in the quantity of the effluent.

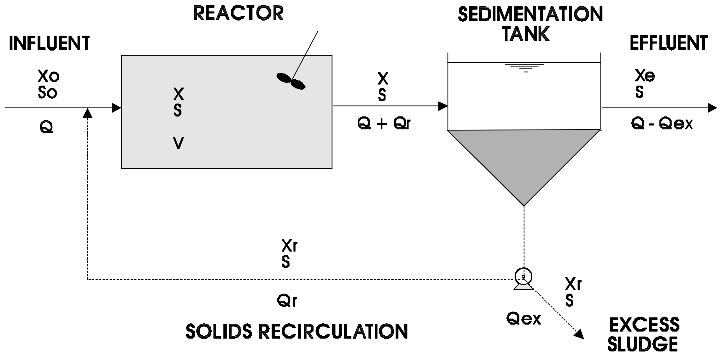
Based on this concept, various treatment systems incorporate a settling unit after the reactor in order to retain the biological solids and avoid that they reach the receiving body in the same concentration as found in the reactor. A system with a settling unit is shown in Figure 9.11.

The inclusion of a final settling unit results in a great improvement in the final effluent quality, thanks to the tendency presented by the bacteria responsible for the stabilisation of the organic matter to flocculate and settle. Thus, they have not just the property of removing BOD with efficiency, but they can be also removed by simple solid-liquid separation operations, such as sedimentation.

The capacity of a system in the removal of organic matter depends on the quantity of biomass present in the reactor. In the above system, the biomass concentration is limited by the quantity of substrate available in the influent: if the substrate increases, the bacteria population growth rate will increase, according to Monod kinetics, until a maximum limit given by μ_{max}. Hence, for a given substrate, the biomass concentration does not go above a certain maximum value.

9.5.2.3 Reactor with a final sedimentation unit and with solids recirculation

The sludge accumulated up to a certain period at the bottom of the settling unit consists mainly of bacteria that are still active in terms of their capacity to assimilate organic matter. Therefore, it is an attractive idea to use these bacteria to assist in



Q_r = recycle or return sludge flow (m^3/d)
Q_{ex} = excess (surplus or waste) sludge flow (m^3/d)
X_r = concentration of suspended solids in the return sludge (mg/L or g/m^3)

Figure 9.12. Biological reactor with recirculation of solids

the removal of the organic matter, based on the fact that, the greater the biomass concentration, the greater the substrate utilisation or, in other words, the greater the BOD removal. Therefore, if the settled sludge is returned, with a concentration higher than in the reactor, the system will be able to assimilate a much higher BOD load. This recirculation has also the important role of increasing the average time in which the microorganisms remain in the system. The recirculation of biomass is the basic principle of systems, such as activated sludge (accomplished by a recirculation pumping station) and UASB reactors (reached by the return of solids that settled in the sedimentation tank, situated above the digestion compartment). Figure 9.12 illustrates the concept of a system with sludge return.

The value of X_r is higher than X, that is, the return sludge has a greater suspended solids concentration, what allows the increase of SS concentration in the reactor.

In Figure 9.12 there is another flow line, which corresponds to the excess sludge (also called surplus sludge, biological sludge or waste sludge). This is based on the concept that the biomass production (bacterial growth) must be compensated for by the wastage of an equivalent quantity, for the system to be maintained in equilibrium. If there were no such a wastage, the mass of suspended solids in the reactor would progressively increase, and these solids would then be transferred to the settling tank, until a point when the settler would become overloaded. In this situation, the settling tank would not be capable of transferring the solids to the bottom, and the sludge blanket level would rise and eventually the solids would start to escape in the final effluent, thus deteriorating its quality. Therefore, in a simplified way, it can be said that the production of solids must be counterbalanced by an equivalent wastage of solids (mass per unit time). The excess sludge flow is very small compared with the influent and return sludge flows.

All biological treatment systems produce excess sludge. In the complete-mix systems without recirculation (Figure 9.10), the excess sludge leaves with the final effluent. In other systems (usually with large reactor volumes), the sludge remains stored in the system and is only removed after large time intervals. The system represented in Figure 9.11 could be according to this concept (e.g. complete-mix aerated lagoons followed by sedimentation ponds) or it could include a separate line for the continuous or periodic removal of the excess sludge.

9.5.3 Hydraulic detention time and solids retention time

In a system with solids recycling, such as in Figure 9.12, the solids are separated and concentrated in the final settling unit and subsequently returned to the reactor. The liquid, on the other hand, in spite of the recirculation (which is internal in the system), does not vary quantitatively, apart from the withdrawal of the excess sludge flow, which is negligible in the overall calculation ($Q_{ex} \approx 0$). Therefore, only the solids are retained in the system, owing to the separation, thickening and recycling. Thus, the solids remain longer in the system than the liquid. It is thus necessary to distinguish the concepts of *solids retention time* and *hydraulic detention time*. Other treatment systems retain solids without the need of separate settling tanks (e.g. sequencing batch reactors) or recycle pumps (e.g. sequencing batch reactors, UASB reactors).

The **hydraulic detention time** t (or hydraulic retention time – HRT) given by:

$$\text{hydraulic detention time} = \frac{\text{volume of liquid in the system}}{\text{volume of liquid removed per unit time}} \qquad (9.30)$$

Since the volume of liquid that enters is the same as the one that leaves, the following generalisation can be made:

$$\boxed{t = \frac{V}{Q}} \qquad (9.31)$$

Similarly, the **solids retention time** SRT (or *mean cell residence time* – MCRT or *sludge age* - θ_c) is given by:

$$\text{sludge age} = \frac{\text{mass of solids in the system}}{\text{mass of solids produced per unit time}} \qquad (9.32)$$

In the steady state, the quantity of solids removed from the system is equal to the quantity of sludge produced. Hence, the sludge age can also be expressed as:

$$\text{sludge age} = \frac{\text{mass of solids in the system}}{\text{mass of solids removed per unit time}} \qquad (9.33)$$

Since the biomass production can be represented by dX/dt, Equation 9.33 can be written as:

$$\boxed{\theta_c = \frac{V.X}{V.\left(\dfrac{dX}{dt}\right)} = \frac{X}{dX/dt}} \tag{9.34}$$

As seen in Section 9.4.3.3, the net bacterial growth (denominator of Equation 9.34) can be given by:

$$\frac{dX}{dt} = \mu.X - K_d.X = (\mu - K_d).X \tag{9.35}$$

Substituting Equation 9.35 into 9.34, the equation that represents the sludge age is obtained:

$$\boxed{\theta_c = \frac{1}{\mu - K_d}} \tag{9.36}$$

Depending on inclusion or not of sludge recycle, the following two conditions are obtained:

- Systems without solids retention: $t = \theta_c$
- Systems with solids retention: $t > \theta_c$

The fact that the biomass stays longer than the liquid in the system justifies the greater efficiency of systems with solids recirculation, compared with systems without solids recirculation. It can also be said that, for the same removal efficiency, systems with solids recirculation require much smaller reactor volumes than the systems without recirculation.

In all the above analyses, the following simplifying hypotheses have been adopted:

- *The biochemical reactions occur only in the reactor*. The reactions of the conversion of organic matter and of cellular growth in the settling unit can be neglected, when compared with those that occur in the reactor. The error resulting from this simplification can be considered negligible.
- *The biomass is assumed to be present only in the reactor*. In the calculation of the sludge age, the solids present in the final settling unit and in the recirculation line have not been considered. This is only a question of convention, and normally only the mass in the reactor is considered, due to the greater simplicity associated with this procedure (measurement of only the SS concentration in the reactor). If the component of the mass of

solids present in the settling tank is incorporated, this needs to be clearly stated when presenting the sludge age value.

- *The mechanisms take place according to the steady state*. This hypothesis greatly simplifies the real situation that occurs in a wastewater treatment plant, in which the true steady state will practically never occur. The continuous variation of the influent characteristics (throughout the day) is responsible for the predominance of the dynamic state in operation, with mass accumulations occurring in the reactor and settling tank. However, if the system is analysed in a broad time scale, these variations become less important. Thus, it can be said that, for designing or operational planning in long time horizons, the steady state assumption can be accepted. On the other hand, for the operation of a plant in a short time scale, the predominance of the dynamic state must be taken into consideration, and the above relations cannot be used as such. *In the dynamic state, the mass of solids produced is not equal to the mass wasted, which alters the interpretation of the sludge age concept.*
- *The influence of the solids in the influent sewage was not considered*. This is a simplifying assumption adopted in most books, but it can be far from reality in some sewage treatment plants with a lower production of biological solids.

Example 9.3

Calculate the hydraulic detention time and the sludge age in the sewage treatment system described in Example 9.2 (without a settling tank and solids recirculation). The main relevant data from Example 9.2 are:

Reactor volume: $V = 9{,}000\ m^3$
Input and output variables:

- Influent flow: $Q = 3{,}000\ m^3/d$
- Influent substrate (BOD_5 total): $S_o = 350$ mg/L
- Effluent substrate (BOD_5 soluble): $S = 9.1$ mg/L

Model coefficients:

- Maximum specific growth rate: $\mu_{max} = 3.0\ d^{-1}$
- Half-saturation coefficient: $K_s = 60$ mg/L
- Endogenous respiration coefficient: $K_d = 0.06\ d^{-1}$

Solution:

a) Hydraulic detention time
From Equation 9.31:

$$t = \frac{V}{Q} = \frac{9{,}000\,m^3}{3{,}000\,m^3/d} = 3.0\ d$$

***Example 9.3* (*Continued*)**

b) Sludge age
The value of μ is given by Equation 9.13:

$$\mu = \mu_{max} \cdot \frac{S}{K_s + S} = 3.0 . \frac{9.1}{60 + 9.1} = 0.395\,d^{-1}$$

The sludge age is given by Equation 9.36:

$$\theta_c = \frac{1}{\mu - K_d} = \frac{1}{0.395 - 0.06} = 3.0\,d$$

As expected, in the present example $t = \theta_c$, since the system has no solids recirculation. Chapter 31, about the activated sludge process, presents examples with recirculation of solids, in which $\theta_c > t$ is obtained.

9.5.4 Cell wash-out time

The time in which the bacteria stays in the treatment system (θ_c) must be higher than the time necessary for its duplication. If not, the cell is going to be washed out of the system before it has time to multiply itself, leading to a progressive reduction in the biomass concentration in the reactor, until the system collapses.

As previously seen, the reproduction of bacteria is by binary fission, and the net specific growth rate is as seen in Section 9.4.3:

$$\frac{dX}{dt} = (\mu - K_d).X \tag{9.37}$$

or

$$\frac{dX}{X} = (\mu - K_d).dt \tag{9.38}$$

The integration of this equation within the limits of $t = 0$ and $t = t$ leads to:

$$\ln \frac{X}{X_o} = (\mu - K_d).t \tag{9.39}$$

where:
$X =$ number or bacterial concentration at a time t
$X_0 =$ number or bacterial concentration at a time $t = 0$

This is the exponential growth phase, which, if plotted on a logarithmic scale, gives a straight line. The duplication time is that in which $X = 2X_0$, or:

$$\ln 2 = (\mu - K_d).t \tag{9.40}$$

Hence, the doubling time t_{dup} is given by:

$$\boxed{t_{dup} = \frac{\ln 2}{\mu - K_d} = \frac{0.693}{\mu - K_d}} \qquad (9.41)$$

The considerations here are also distinct for the systems with and without recirculation (Arceivala, 1981):

- *Systems with suspended biomass, without recirculation* ($\theta_c = t$). In this case, θ_c ($= t$) needs to be greater than or equal to t_{dup}. This condition needs to be satisfied in the case, for instance, of complete-mix aerated lagoons. In these lagoons, it is essential to ensure that the minimum hydraulic detention time is not less than the bacterial doubling time, at critical temperature conditions. In the design of units such as ponds in series, the minimum size of each pond is dictated by this requirement.
- *Systems with suspended biomass, with recirculation* ($\theta_c > t$). In these systems, the excess sludge flow can be adjusted in order to maintain $\theta_c > t_{dup}$, while the hydraulic detention time t can be maintained at a minimum (minimum volume of the reactor). Consequently, the sludge recirculation is a way of increasing θ_c without necessarily increasing t (or V).

For the aerobic removal of the carbonaceous matter, the solids retention time of the heterotrophic bacteria is usually much higher than the minimum time required. However, for the methanogenesis in anaerobic systems, as well as for the oxidation of ammonia in aerobic systems, greater care must be exercised. The reproduction rate of the methanogenic and nitrifying organisms is very slow and there is the risk of their wash-out of the system if the influent flow increases substantially or if their reproduction rate is reduced due to some environmental problem.

9.5.5 Concentration of suspended solids in the reactor

To obtain the biomass concentration in the reactor of a system **with solids recirculation**, equations 9.21 and 9.34 can be rearranged, assuming steady-state conditions:

$$\boxed{X = \frac{Y.(S_o - S)}{1 + K_d.\theta_c}.\left(\frac{\theta_c}{t}\right)} \qquad (9.42)$$

This equation is very important for the estimation of the solids concentration in a complete-mix reactor, once the other parameters or variables are known or have been estimated. The analysis of this equation also leads to interesting considerations about the influence of the sludge recirculation on the SS concentration in the reactor.

In a system **without recirculation** it was seen that $\theta_c = t$. Consequently, Equation 9.42 is reduced to:

$$\boxed{X = \frac{Y.(S_o - S)}{1 + K_d.t}} \tag{9.43}$$

It can be observed that the difference between both equations is the factor (θ_c/t), which exerts a multiplying factor on Equation 9.43, in the sense of increasing the suspended solids concentration in the reactor. In the design of a wastewater treatment plant, any increase in X allows a proportional decrease in the volume required for the reactor. Example 9.4 illustrates the calculation of the biomass concentration in a system without recirculation, while Example 9.5 shows the advantage of the recirculation in terms of the reduction of the reactor volume.

Example 9.4

Calculate the suspended solids concentration to be reached, under steady-state conditions, in the reactor described in Example 9.2. The relevant data from Example 9.2 are:

- Influent substrate (total BOD_5): $S_o = 350$ mg/L
- Effluent substrate (soluble BOD_5): $S = 9.1$ mg/L
- Hydraulic detention time: $t = 3.0$ days (obtained in Example 9.3)
- Yield coefficient: $Y = 0.6$ mgVSS/mgBOD$_5$
- Endogenous coefficient: $K_d = 0.06\ d^{-1}$

Solution:

From Equation 9.43:

$$X_v = \frac{Y.(S_o - S)}{1 + K_d.t} = \frac{0.6 \times (350 - 9.1)\,\text{mg/L}}{1 + 0.06\text{d}^{-1} \times 3\text{d}} = 173.3\ \text{mg/L}$$

This value was used as input data in Example 9.2.

Example 9.5

Calculate the biomass concentration in the reactor, for the following conditions:

- system without recirculation, $t = \theta_c = 5$ days (e.g.: complete-mix aerated lagoon)
- system with recirculation, $t = 0.25$ days (6 hours) and $\theta_c = 5$ days (e.g.: conventional activated sludge)

***Example 9.5* (*Continued*)**

Adopt $Y = 0.6$; $K_d = 0.07\ d^{-1}$; $S_o = 300$ mg/L; $S = 15$ mg/L

Solution:

a) System without recirculation

From Equation 9.43:

$$X_v = \frac{Y.(S_o - S)}{1 + K_d.t} = \frac{0.6 \times (300 - 15)\,mg/L}{1 + 0.07d^{-1} \times 5d} = 127\,mg/L$$

b) System with recirculation

From Equation 9.42:

$$X_v = \frac{Y.(S_o - S)}{1 + K_d.\theta_c}.\left(\frac{\theta_c}{t}\right) = \frac{0.6 \times (300 - 15)\,mg/L}{1 + 0.07d^{-1} \times 5d}.\left(\frac{5\,d}{0.25\,d}\right) = 2.540\,mg/L$$

c) Comments

It can be observed that in the system with sludge recirculation, the VSS concentration 2,540 mg/L is much higher than in the option without recirculation, in which VSS is equal to 127 mg/L. The ratio between the two concentrations is:

2540 mg/L/127 mg/L = 20

This is the same ratio between the hydraulic detention times in the reactors of the two systems:

5 d / 0.25 d = 20

In other words, it can be said in this example that, for the same effluent characteristics, the volume of the reactor in the system with recirculation is 20 times less than in the system without recirculation.

9.5.6 Effluent substrate

According with Equation 9.36, the sludge age can be expressed as:

$$\boxed{\theta_c = \frac{1}{\mu - K_d}} \tag{9.44}$$

The rearrangement of Equation 9.44 leads to:

$$\frac{1}{\theta_c} = \mu - K_d \tag{9.45}$$

or:

$$\frac{1}{\theta_c} = \mu_{max}.(\frac{S}{K_s - S}) - K_d \qquad (9.46)$$

Rearranging Equation 9.46 in terms of S:

$$\boxed{S = \frac{K_s.[(1/\theta_c) + K_d]}{\mu_{max} - [(1/\theta_c) + K_d]}} \qquad (9.47)$$

This is the general equation to estimate the soluble effluent BOD from a complete-mix reactor. An interesting aspect of this equation is that, mathematically, in a complete-mix system, the effluent BOD concentration S is independent of the influent concentration S_o (Arceivala, 1981). This is because K_s, K_d and μ_{max} are constants and therefore S depends only on the sludge age θ_c. This can be understood by the fact that, the greater the influent BOD, the greater is the production of biological solids, and, as a result, the greater is the biomass concentration X_v. Thus, when there is more food, there is a greater availability for the bacteria to assimilate it. It must be emphasised that this consideration is applicable only in the steady state. In the dynamic state, the increases in the influent BOD are not immediately followed by the corresponding increase in the biomass, since the process of biomass increase is slow. Hence, until a new equilibrium state is reached (if it will be reached at all), the effluent quality, in terms of BOD, will be deteriorated.

Theoretically, the minimum concentration of soluble substrate that can be reached in a system is when the sludge age θ_c tends to infinity. In these conditions, the term $1/\theta_c$ is equal to zero. By substituting $1/\theta_c$ for 0 in Equation 9.47, an equation that defines the minimum possible soluble effluent BOD (S_{min}) is found. If in a treatment system it is necessary to obtain a value lower than S_{min}, it will not be possible with only one complete-mix reactor (Grady & Lim, 1980). S_{min} is independent on the existence of recirculation and is only a function of the kinetic coefficients.

$$\boxed{S_{min} = \frac{K_s.K_d}{\mu_{max} - K_d}} \qquad (9.48)$$

Example 9.6

Calculate the soluble effluent BOD concentration, after steady-state conditions have been reached in the system described in Example 9.2. Since the system has no solids recirculation, the sludge age is equal to the hydraulic detention time. The relevant data for this example are:

- Sludge age (= equal to the hydraulic detention time): $\theta_c = t = 3.0$ d (according with Example 9.3)

***Example 9.6* (*Continued*)**

- Maximum specific growth rate: $\mu_{max} = 3.0\ d^{-1}$
- Half-saturation coefficient: $K_s = 60$ mg/L
- Endogenous respiration coefficient: $K_d = 0.06\ d^{-1}$

Solution:

From Equation 9.47:

$$S = \frac{K_s.[(1/\theta_c) + K_d]}{\mu_{max} - [(1/\theta_c) + K_d]} = \frac{60 \times [(1/3) + 0.06]}{3.0 - [(1/3) + 0.06]} = 9.1 mg/L$$

This value of S = 9.1 mg/L is the same as the one adopted in the preceding examples.

9.5.7 Loading rates on biological reactors

9.5.7.1 Sludge load (food-to-microorganism ratio)

A relationship widely used by designers and operators of wastewater treatment plants is the *sludge load* or *F/M* (food-to-microorganism) *ratio*. It is based on the concept that the quantity of food or substrate available per unit mass of microorganisms is related to the efficiency of the system. Hence, it can be understood that, the higher the BOD load supplied per unit value of the biomass (high F/M ratio), the lower is the substrate assimilation efficiency, but, on the other hand, the lower is the required reactor volume. Conversely, when less BOD is supplied to the bacteria (low F/M ratio), the demand for food is higher, which implies a greater BOD removal efficiency and a larger reactor volume requirement. In a situation in which the quantity of food supplied is very low, the mechanism of endogenous respiration becomes prevalent, which is a characteristic of low-rate (e.g. extended aeration) systems.

The food load supplied is given by:

$$F = Q.S_0 \tag{9.49}$$

The microorganism mass is calculated as:

$$M = V.X_v \tag{9.50}$$

where:

Q = influent flow (m^3/d)
S_0 = influent BOD_5 concentration (g/m^3)
V = reactor volume (m^3)
X_v = volatile suspended solids concentration (g/m^3)

Thus, the F/M ratio is expressed as:

$$\boxed{\frac{F}{M} = \frac{Q.S_0}{V.X_v}} \tag{9.51}$$

where:

F/M = sludge load (gBOD$_5$ supplied per day/g VSS)

The F/M ratio is sometimes expressed in terms of total suspended solids (TSS) instead of VSS. Care must be exercised when analysing values, in order not to mix nomenclatures and forms of expression. There is a relationship between VSS and SS, which is a function of the sludge age. High sludge ages (low F/M ratios) imply higher removals of the organic fraction, represented by the volatile suspended solids, leading to a lower VSS/SS ratio. Each wastewater treatment system has typical values of the VSS/SS ratio predominant in the biological reactor.

The relation Q/V in Equation 9.51 can be substituted by 1/t, which leads to another way of presenting the F/M ratio:

$$\boxed{\frac{F}{M} = \frac{S_o}{t.X_v}} \tag{9.52}$$

Accurately speaking, the F/M ratio has no direct association with the removal of the organic matter that really occurs in the reactor, since the F/M ratio constitutes only a representation of the *applied* (or available) load. The formula that expresses the relation between the available and the removed substrates is the *substrate utilisation rate* (U). In U, instead of including only S$_o$, the relation S$_o$ − S is included:

$$\boxed{U = \frac{Q.(S_o - S)}{V.X_v}} \tag{9.53}$$

where:

S = soluble effluent BOD$_5$ concentration (g/m^3)

Hence, it can be said that:

$$U = (F/M).E \tag{9.54}$$

where:

E = substrate removal efficiency of the system = $(S_o - S)/S_o$

Because the substrate removal efficiencies in sewage treatment systems are usually high and not far from the unity, it can be said that U ≈ F/M.

Analysing Equation 9.53, it can be seen that, after defining the design value for F/M (or U), adopting a value for X_v, and knowing the flow Q and the influent S_o and effluent S (desired) BOD concentrations, the necessary volume for the reactor can be calculated. Rearranging the formula leads to:

$$\boxed{V = \frac{Q.(S_o - S)}{X_v.U}} \tag{9.55}$$

Example 9.7 illustrates the use of the concepts of F/M and U to analyse the operational range of an existing works, while Example 9.8 presents U as a design parameter in the calculation of the reactor volume.

Example 9.7

Calculate the values of F/M and U in a wastewater treatment plant with sludge recirculation, as described in Example 9.5. Data:

$S_o = 300\ gBOD_5/m^3$
$S = 15\ gBOD_5/m^3$
$t = 0.25\ d$
$X_v = 2{,}540\ gVSS/m^3$

Solution:

a) Calculation of F/M

From Equation 9.52:

$$\frac{F}{M} = \frac{S_o}{t.X_v} = \frac{300\,gBOD_5/m^3}{0.25\,d\,.\,2{,}540\,gVSS/m^3} = 0.47d^{-1}$$

F/M = 0.47 $kgBOD_5/kgVSS.d$

b) Calculation of U

From Equation 9.53, substituting Q/V by 1/t:

$$\frac{F}{M} = \frac{S_o - S}{t.X_v} = \frac{(300 - 15)\,gBOD_5/m^3}{0.25\,d\,.\,2{,}540\,gVSS/m^3} = 0.45d^{-1}$$

U = 0.45 $kgBOD_5/kgVSS.d$

In comparison with Example 9.5, is can be seen that, in this case, the substrate utilisation rate U is equal to 0.45 $kgBOD_5/kgVSS.d$, corresponding to a sludge age of 5.0 days (calculated in Example 9.5).

Example 9.8

Calculate the reactor volume of an extended aeration activated sludge system, given that:

$U = 0.12\,\text{kgBOD}_5/\text{kgVSS.d}$ (adopted)
$Q = 5{,}000\,\text{m}^3/\text{d}$ (design data)
$S_o = 340\,\text{mg/L}$ (design data)
$S = 5\,\text{mg/L}$ (desired)
$X_v = 3{,}500\,\text{mg/L}$ (adopted)

Solution:

From Equation 9.55:

$$V = \frac{Q.(S_o - S)}{X_v.U} = \frac{5{,}000\,\text{m}^3/\text{d}.(340 - 5)\text{gBOD}_5/\text{m}^3}{3{,}500\,\text{gVSS}/\text{m}^3.0.12\,\text{kgBOD}_5/\text{kgVSS.d}} = 3{,}988\,\text{m}^3$$

9.5.7.2 Relationship between the substrate utilisation rate (U) and the sludge age (θ_c)

In the steady state, there is no accumulation of solids in the system, which makes the following relation valid:

Solids production rate = *Solids removal rate*
(biological solids generated) = *(excess sludge wasted)*

From Equation 9.21:

$$\boxed{\frac{dX}{dt} = Y.\frac{dS}{dt} - K_d.X} \tag{9.56}$$

or:

$$\boxed{\frac{\Delta X_v}{\Delta t} = Y.\frac{(S_o - S)}{t} - K_d.f_b.X_v} \tag{9.57}$$

Dividing by X_v:

$$\frac{\Delta X_v/\Delta t}{X_v} = Y.\frac{(S_o - S)}{X_v.t} - K_d.f_b \tag{9.58}$$

According to Equation 9.34, the first part of Equation 9.58 is equal to $1/\theta_c$. Hence:

$$\boxed{\frac{1}{\theta_c} = Y.\left(\frac{S_o - S}{X_v.t}\right) - K_d.f_b} \qquad (9.59)$$

According to Equation 9.53, the term in brackets is the substrate utilisation rate (U). Therefore, in the steady state:

$$\boxed{\frac{1}{\theta_c} = Y.U - K_d.f_b} \qquad (9.60)$$

Since $U = (F/M).E$, Equation 9.60 can also be also expressed as:

$$\boxed{\frac{1}{\theta_c} = Y.\left(\frac{F}{M}\right).E - K_d.f_b} \qquad (9.61)$$

Therefore, Equations 9.60 and 9.61 correlate θ_c and U (or F/M), once the values of the coefficients Y and K_d have been adopted. Thus, knowing θ_c, U can be calculated (or F/M), or vice-versa.

Equations 9.60 and 9.61 are presented in various texts in terms of X_V, without considering that only part of X_V is biodegradable. Hence, the correct is to express the decay in terms of X_b, or of $f_b.X_v$, since $f_b = X_b/X_v$, where X_b is the concentration of the biodegradable volatile suspended solids and f_b is the biodegradable fraction of VSS.

The reason for the consideration of X_b is reinforced by the following point. In systems with a high sludge age, such as the extended aeration activated sludge, if Equations 9.60 and 9.61 were presented without f_b, certain indeterminations could be generated (not present in conventional texts, because most of them are dedicated to systems with a conventional sludge age). When substituting Y and K_d by typical values in the literature and adopting F/M values representative of extended aeration, a negative value of $1/\theta_c$ (negative net sludge production) is obtained. Such incongruence occurs because the values of Y and K_d expressed in the literature are mainly associated to determinations undertaken in systems with a conventional sludge age. A solution for this problem is the adoption of coefficient values resulting from laboratory tests under operational conditions similar to those expected in practice. The main discrepancy found in long sludge age systems is because of the fact that, given the low substrate availability prevalent in the medium, a predominance of endogenous respiration occurs, which causes a decrease of the biodegradable fraction of the volatile solids. Consequently, the higher the sludge age, the larger is the inert fraction of the solids (due to the aerobic digestion that takes place in the reactor), which results in a lower biodegradable fraction f_b. The concept of f_b is detailed in Section 9.5.8 and used in the chapters relating to the activated sludge process presented in this book (Part 5).

The selection of the K_d value should also reflect the representation of the biomass. The values of K_d usually reported in the literature are associated with the decay of X_v. In this text, when representing the decay in terms of the biodegradable solids, K_d assumes higher values than usually found in the literature, because it will be subsequently multiplied by f_b. When multiplying the proposed values of f_b and K_d, in a system with a conventional sludge age, the results obtained correspond to the usual values of K_d, found in the literature.

The present approach, described by Eckenfelder (1989), allows the generalisation of systems such as activated sludge for the conventional as well as for the extended aeration mode. There are even more sophisticated models, such as those of the IWA (2000) – ASM1-1987, ASM2-1995, ASM3-2000 – based on the active fraction of the biomass. These models, in spite of their great explanatory power, scientifically based structure and wide acceptance, are more complex, being outside of the more general scope of this book. However, readers who want to deepen their knowledge into the mechanisms of C, N and P removal, are encouraged to consult these references, especially the latter, which is a consolidation of the previous two.

9.5.7.3 Volumetric organic load

Some sewage treatment systems are designed according to the *volumetric organic load*, which is represented by the equation:

$$\boxed{L_{VO} = \frac{Q.S_o}{V}} \tag{9.62}$$

where:

L_{VO} = volumetric organic load (gBOD/m^3.d)

Since Q/V is equal to 1/t, Equation 9.62 can be rewritten as:

$$\boxed{L_{VO} = \frac{S_o}{t}} \tag{9.63}$$

Adopting a value for L_{VO}, the required reactor volume can be calculated through the rearrangement of Equation 9.62:

$$\boxed{V = \frac{Q.S_o}{L_{VO}}} \tag{9.64}$$

It can be observed that the volumetric organic load differs from the F/M ratio by the fact that the former represents the load applied per unit reactor volume, while the latter represents the load applied per unit of biomass in the reactor.

Example 9.9

Calculate the volumetric organic load in the wastewater treatment system with sludge recirculation, as described in Example 9.5. Data:

$S_o = 300\ gBOD_5/m^3$
$t = 0.25\ d$

Solution:

From Equation 9.63:

$$L_{VO} = \frac{S_o}{t} = \frac{300gDBO_5/m^3}{0.25d} = 1,200g/m^3.d$$

9.5.7.4 *Volumetric hydraulic load*

Other wastewater treatment systems are designed based on the *volumetric hydraulic load* (**L_V**):

$$\boxed{L_V = \frac{Q}{V}} \tag{9.65}$$

where:
L_V = volumetric hydraulic load (m^3/d per m^3)

It should be observed that L_V is simply the reciprocal of the hydraulic detention time t (**$L_v = 1/t$**).

9.5.7.5 *Surface organic load*

Still other biological reactors, such as facultative ponds, are designed based on a *surface organic load*, or *surface loading rate* (L_s). In these reactors, the surface area plays a more important role than the volume itself. In the case of facultative ponds, it is through the surface area that the sunlight penetrates in the pond and allows the development of the required photosynthetic activity. Surface loading rates are expressed as:

$$\boxed{L_S = \frac{Q.S_o}{A}} \tag{9.66}$$

where:
L_S = surface organic load ($kgBOD/m^2.d$ or kgBOD/ha.d)
A = surface area (m^2 or ha)

See Chapter 13 for the application of L_v.

9.5.8 Distribution of the biological solids in the treatment

As already seen, the total suspended solids are composed by an *inorganic (fixed) fraction* (X_i) and an *organic (volatile) fraction* (X_v):

$$\boxed{X = X_i + X_v} \tag{9.67}$$

On the other hand, another division should still be established, because not all the volatile suspended solids are biodegradable. In the volatile solids, there is a fraction, which is *non-biodegradable (inert)* (X_{nb}), resulting from residues of endogenous respiration, and a *biodegradable* fraction (X_b). Thus:

$$\boxed{X_v = X_{nb} + X_b} \tag{9.68}$$

The sludge recirculation leads to an accumulation of the inorganic fraction X_i, as well as the non-biodegradable fraction X_{nb} in the system, since they are not affected by the biological treatment. The higher the sludge age, the lower the ratio X_b/X_v. This can be understood by the fact that at higher sludge ages there is the predominance of endogenous respiration with a greater self-oxidation of the cellular material, that is, stabilisation of the sludge

The volatile solids shortly after being produced ($\theta_c = 0$) are approximately 20% inert and 80% biodegradable. With their stay in the reactor ($\theta_c > 0$), the ratio X_b/X_v decreases. The ratio X_b/X_v (= f_b) can be expressed as (Eckenfelder, 1989):

$$\boxed{f_b = \frac{f'_b}{1 + (1 - f'_b).K_d.\theta_c}} \tag{9.69}$$

where:

f_b = biodegradable fraction of the VSS generated in the system, submitted to a sludge age θ_c (X_b/X_v)

f'_b = biodegradable fraction of the VSS immediately after their generation in the system, that is, with $\theta_c = 0$. This value is typically equal to 0.8 (= 80%)

For the various values of K_d and sludge age, Table 9.1 presents the f_b values resulting from Equation 9.69.

The values of f_b are used in various formulas in the activated sludge process, such as those related with the sludge production, oxygen consumption by the biomass and BOD associated with the suspended solids in the effluent.

The values presented in Table 9.1 are only related to the biological solids produced in the reactor. Raw sewage also contributes with fixed solids and volatile,

Table 9.1. Biodegradable fraction of the VSS (f_b), according to Equation 9.69, for various values of θ_c and K_d

θ_c	$X_b/X_v(= f_b)$ ratio			
(days)	$K_d = 0.05\ d^{-1}$	$K_d = 0.07\ d^{-1}$	$K_d = 0.09\ d^{-1}$	$K_d = 0.11\ d^{-1}$
4	0.77	0.76	0.75	0.74
8	0.74	0.72	0.70	0.68
12	0.71	0.68	0.66	0.63
16	0.69	0.65	0.62	0.59
20	0.67	0.63	0.59	0.56
24	0.65	0.60	0.56	0.52
28	0.63	0.57	0.53	0.50
32	0.61	0.55	0.51	0.47

non-biodegradable and biodegradable solids. Approximate values of the main relationships in raw sewage are (WEF/ASCE, 1992; Metcalf & Eddy, 1991):
Raw sewage:

- $VSS/TSS = 0.70 - 0.85$
- $SS_i/TSS = 0.15 - 0.30$

- $SS_b/VSS = 0.6$
- $SS_{nb}/VSS = 0.4$

The load relative to the contribution of these raw sewage solids should be taken into account, especially the inorganic and non-biodegradable organic fractions, which do not undergo transformation in the biological treatment. The load of biodegradable solids need not be taken into separate consideration, since these solids will be absorbed in the biological flocs in the reactor, where they will be hydrolysed and subsequently degraded, thus generating new biological solids and an oxygen consumption (in aerobic systems). Since this contribution is already included in the biological solids generated due to the influent BOD, the SS_b of the raw sewage should not be calculated separately. In systems with primary settling, the fraction of the raw sewage solids that is removed by sedimentation and do not enter the biological reactor should be discounted.

The *active fraction* of the volatile solids, that is, the fraction that is really responsible for the decomposition of the carbonaceous organic matter, is given by (IAWPRC, 1987; WEF/ASCE, 1992):

$$f_a = \frac{1}{1 + (1 - f_b').K_d.\theta_c} \tag{9.70}$$

where:

f_a = active fraction of the volatile suspended solids (X_a/X_v)

The fraction f_a can be also expressed as:

$$f_a = f_b/f'_b \tag{9.71}$$

Some mathematical models (IWA, 2000) express the kinetics of the removal of organic matter in terms of the active suspended solids. However, in the present book, the more conventional and simpler version of expressing the solids by VSS is adopted.

During the treatment, there is the generation and destruction of solids. This section presents below the various formulas related to the creation and destruction of the various types of solids present in the reactor (excluding the solids of the influent wastewater).

The **gross production of volatile suspended solids** (P_{xv}) is a result of the multiplication of the yield coefficient by the BOD_5 load removed:

$$\boxed{P_{xv}\,\text{gross} = Y.Q.(S_0 - S)} \tag{9.72}$$

From the suspended solids recently formed in the reactor, approximately 90% are organic (volatile) and 10% are inorganic (fixed) (Metcalf & Eddy, 1991), leading to the ratio VSS/TSS = 0.9 in the recently formed biological solids. With this ratio, the recently produced TSS load can be estimated (**gross production of total suspended solids**, without including destruction):

$$\boxed{P_x\,\text{gross} = P_{xv}/0.9} \tag{9.73}$$

As a result, the **production of fixed suspended solids** is:

$$\boxed{P_{xi} = P_x\,\text{gross} - P_{xv}\,\text{gross}} \tag{9.74}$$

Not all the volatile solids produced are biodegradable. Immediately after production ($\theta_c = 0$), the load of the biodegradable solids produced is equal to the product of the volatile solids produced (P_{xv}) by the biodegradable fraction of the solids recently formed (f'_b). It was seen above that typical values of f'_b are around 0.8. The determination of this load of **recently formed biodegradable suspended solids** has little practical value, since the solids remain in the reactor for a time greater than θ_c. Only as an illustration, the formula for its calculation is presented below:

$$\boxed{P_{xb}\,\text{recently formed} = (P_{xv}\,\text{gross}).f'_b} \tag{9.75}$$

As a result of the time that the solids remain in the reactor (θ_c), the biodegradability fraction f_b decreases. Hence, the **gross production of biodegradable**

suspended solids submitted to a detention time θ_c is equal to P_{xv} gross multiplied by the biodegradability fraction f_b. As seen above, f_b is a function of θ_c (Equation 9.69).

$$\boxed{P_{xb}\text{ gross} = (P_{xv}\text{ gross}).f_b} \tag{9.76}$$

The **production of non-biodegradable volatile suspended solids** (inert or endogenous) is obtained by the difference between the gross production of X_v and the gross production of X_b:

$$\boxed{P_{xnb} = P_{xv}\text{ gross} - P_{xb}\text{ gross}} \tag{9.77}$$

The **gross production of the active fraction of the volatile suspended solids** is given by:

$$\boxed{P_{xa}\text{ gross} = (P_{xv}\text{ gross}).f_b/f_b'} \tag{9.78}$$

Owing to endogenous respiration, part of the biodegradable solids are destroyed in the reactor. The **load of the biodegradable suspended solids destroyed** is a function of the sludge age, and is given by:

$$\boxed{P_{xb}\text{ destroyed} = (P_{xb}\text{ gross}).(K_{d.}.\theta_c)/(1 + f_b.K_d.\theta_c)} \tag{9.79}$$

Therefore, the **net production of biodegradable suspended solids** is:

$$\boxed{P_{xb}\text{ net} = P_{xb}\text{ gross} - P_{xb}\text{ destroyed}} \tag{9.80}$$

The **net production of volatile suspended solids** is equal to the net production of biodegradable solids plus the production of non-biodegradable organic solids.

$$\boxed{P_{xv}\text{ net} = P_{xb}\text{ net} + P_{xnb}} \tag{9.81}$$

The net production of the volatile suspended solids can also be obtained by using the concept of the observed yield (Y_{obs}). Y_{obs} already takes into consideration the destruction of the biodegradable solids and is expressed by:

$$\boxed{Y_{obs} = \frac{Y}{1 + f_b.K_d.\theta_c}} \tag{9.82}$$

Thus, the net production of VSS can also be given by:

$$\boxed{P_{xv}\ \text{net} = Y_{obs}.Q.(S_o - S)} \tag{9.83}$$

The **net production of the total suspended solids** is equal to the net production of volatile solids plus the net production of inorganic solids:

$$\boxed{P_x\ \text{net} = P_{xv}\ \text{net} + P_{xi}} \tag{9.84}$$

The *final ratio VSS/TSS* (due only to biological solids) in the reactor is obtained through:

$$\boxed{\text{VSS/SS} = (P_{xv}\ \text{net})/(P_x\ \text{net})} \tag{9.85}$$

The *percentage destruction of the biodegradable suspended solids* due to endogenous respiration is given by:

$$\boxed{\%\ \text{destruction}\ X_b = 100.(P_{xb}\ \text{destroyed})/(P_{xb}\ \text{gross})} \tag{9.86}$$

The *percentage destruction of the volatile suspended solids* is expressed by:

$$\boxed{\%\ \text{destruction}\ X_v = 100.(P_{xb}\ \text{destroyed})/(P_{xv}\ \text{gross})} \tag{9.87}$$

If it is desired to take into account the solids present in the influent wastewater, the load of the influent solids (inorganic solids and non-biodegradable solids) should be added to each one of the loads of biological solids produced. As mentioned above, it is not necessary to sum the contribution of the biodegradable solids of the influent wastewater, due to the fact that they are already included in the influent BOD. The influent BOD comprises the soluble BOD_5 fraction as well as the particulate BOD_5 fraction (resulting from the influent biodegradable solids). Consequently, the transformation of the total BOD_5 into biodegradable solids indirectly incorporates the contribution of the biodegradable solids from the influent wastewater.

Example 9.10 synthesises the relations described above for conventional and extended aeration activated sludge systems. In order to allow a comparison, the starting point in both cases is the removal of a BOD load equal to 100 $kgBOD_5/d$. Hence, the values of the production and destruction, as well as the distribution of the solids, are referenced to a value of 100 units. In both situations, only the biological solids formed in the reactor are analysed and the contribution of the solids from the influent wastewater are not taken into consideration. The influence of the influent solids is considered in the examples presented in Chapter 34.

Example 9.10

Calculate the distribution of suspended solids in a reactor for the following treatment systems: (a) conventional activated sludge with a sludge age of 6 days and (b) extended aeration with a sludge age of 22 days. Do not consider the solids in the influent sewage. Assume that the removed BOD_5 load is 100kg/d. Adopt:

- $Y = 0.6\,gVSS/gBOD_5$ removed
- $K_d = 0.09\,d^{-1}$
- VSS/SS ratio in the recently formed solids = 0.9 gVSS/SS
- f_b' (recently formed solids) = 0.8 $gSS_b/gVSS$

Solution:

a) Conventional activated sludge

- The BOD_5 load removed is equal to 100 $kgBOD_5/d$. Thus:

$$Q.(S_o - S) = 100\ kg/d$$

- Calculation of f_b (Equation 9.69)

$$f_b = \frac{f_b'}{1 + (1 - f_b').K_d.\theta_c} = \frac{0.8}{1 + (1 - 0.8) \times 0.09 \times 6} = 0.72$$

- *Gross production of volatile suspended solids (Equation 9.72)*

$$P_{xv}\ gross = Y.Q.(S_o - S) = 0.6 \times 100 = 60.0\ kgVSS/d$$

- *Gross production of total suspended solids (Equation 9.73)*

$$P_x\ gross = P_{xv}/(SSV/SS) = 60.0/0.9 = 66.7\ kgTSS/d$$

- *Production of inorganic solids (Equation 9.74)*

$$P_{xi} = P_x\ gross - P_{xv}\ gross = 66.7 - 60.0 = 6.7\ kgSS_i/d$$

- *Gross production of biodegradable suspended solids submitted to a residence time of θ_c (Equation 9.76)*

$$P_{xb}\ gross = (P_{xv}\ gross).f_b = 60.0 \times 0.72 = 43.2\ kgSS_b/d$$

Example 9.10 (Continued)

- *Load of biodegradable suspended solids destroyed (Equation 9.79)*

$$\begin{aligned} P_{xb}\text{ destroyed} &= (P_{xb}\text{ gross}).(K_{d.}\theta_c)/(1 + f_b.K_d.\theta_c) \\ &= 43.2 \times (0.09 \times 6)/(1 + 0.72 \times 0.09 \times 6) \\ &= 43.2 \times 0.39 = 16.8\ \text{kgSS}_b/\text{d} \end{aligned}$$

- *Net production of biodegradable suspended solids (Equation 9.80)*

$$P_{xb}\text{ net} = P_{xb}\text{ gross} - P_{xb}\text{ destroyed} = 43.2 - 16.8 = 26.4\ \text{kgSS}_b/\text{d}$$

- *Production of non-biodegradable volatile suspended solids (Equation 9.77)*

$$P_{xnb} = P_{xv}\text{ gross} - P_{xb}\text{ gross} = 60.0 - 43.2 = 16.8\ \text{kgSS}_{nb}/\text{d}$$

- *Net production of volatile suspended solids (Equation 9.81)*

$$P_{xv}\text{ net} = P_{xb}\text{ net} + P_{xnb} = 26.4 + 16.8 = 43.2\ \text{kgVSS/d}$$

- *Net production of total suspended solids (Equation 9.84)*

$$P_x\text{ net} = P_{xv}\text{ net} + P_{xi} = 43.2 + 6.7 = 49.9\ \text{kgTSS/d}$$

- Ratio VSS/TSS (Equation 9.85)

$$\text{VSS/TSS} = (P_{xv}\text{ net})/(P_x\text{ net}) = 43.2/49.9 = 0.87(87\%)$$

- Percentage destruction of the biodegradable suspended solids (Equation 9.86)

$$\begin{aligned} \%\text{ destruction } X_b &= 100.(P_{xb}\text{ destroyed})/(P_{xb}\text{ gross}) \\ &= 100 \times 16.8/43.2 = 39\% \end{aligned}$$

- Percentage destruction of the volatile suspended solids (Equation 9.87)

$$\begin{aligned} \%\text{ destruction } X_v &= 100.(P_{xb}\text{ destroyed})/(P_{xv}\text{gross}) \\ &= 100 \times 16.8/60.0 = 28\% \end{aligned}$$

Example 9.10 (*Continued*)

b) Extended aeration

- The BOD_5 load removed is equal to 100 $kgBOD_5/d$ (same as in item a). Thus:

$$Q.(S_o - S) = 100\ kg/d$$

- Calculation of f_b (Equation 9.69)

$$f_b = \frac{f_b'}{1 + (1 - f_b').K_d.\theta_c} = \frac{0.8}{1 + (1 - 0.8) \times 0.09 \times 22} = 0.57$$

- *Gross production of volatile suspended solids – same as in item a*

$$P_{xv}\ gross = 60.0\ kgVSS/d$$

- *Gross production of total suspended solids – same as in item a*

$$P_x\ gross = 66.7\ kgTSS/d$$

- *Production of inorganic suspended solids – same as in item a*

$$P_{xi} = 6.7\ kgSS_i/d$$

- *Gross production of biodegradable suspended solids submitted to a residence time θ_c (Equation 9.76)*

$$P_{xb}\ gross = (P_{xv}\ gross).f_b = 60.0 \times 0.57 = 34.2\ kgSS_b/d$$

- *Load of biodegradable suspended solids destroyed (Equation 9.79)*

$$\begin{aligned} P_{xb}\ destroyed &= (P_{xb}\ gross).(K_{d.}\theta_c)/(1 + f_b.K_d.\theta_c) \\ &= 43.2 \times (0.09 \times 22)/(1 + 0.57 \times 0.09 \times 22) \\ &= 34.2 \times 0.93 = 31.8\ kgSS_b/d \end{aligned}$$

- *Net production of biodegradable suspended solids (Equation 9.80)*

$$P_{xb}\ net = P_{xb}\ gross - P_{xb}\ destroyed = 34.2 - 31.8 = 2.4\ kgSS_b/d$$

- *Production of non-biodegradable volatile suspended solids (Equation 9.77)*

$$P_{xnb} = P_{xv}\ gross - P_{xb}\ gross = 60.0 - 34.2 = 25.8\ kgSS_{nb}/d$$

- *Net production of volatile suspended solids (Equation 9.81)*

$$P_{xv}net = P_{xb}net + P_{xnb} = 2.4 + 25.8 = 28.2\ kgVSS/d$$

***Example 9.10* (*Continued*)**

- *Net production of total suspended solids (Equation 9.84)*

$$P_x \text{ net} = P_{xv} \text{ net} + P_{xi} = 28.2 + 6.7 = 34.9 \text{ kgTSS/d}$$

- *Ratio VSS/TSS (Equation 9.85)*

$$\text{VSS/TSS} = (P_{xv} \text{ net})/(P_x \text{ net}) = 28.2/34.9 = 0.81\ (81\%)$$

- *Percentage destruction of the biodegradable suspended solids (Equation 9.86)*

$$\begin{aligned}\%\text{ destruction } X_b &= 100.(P_{xb} \text{ destroyed})/(P_{xb} \text{ gross})\\ &= 100 \times 31.8/34.2 = 93\%\end{aligned}$$

- *Percentage destruction of the volatile suspended solids (Equation 9.87)*

$$\begin{aligned}\%\text{ destruction } X_v &= 100.(P_{xb} \text{ destroyed})/(P_{xv} \text{ gross})\\ &= 100 \times 31.8/60.0 = 53\%\end{aligned}$$

c) Summary

Net production (kg/d)	Conventional activated sludge ($\theta_c = 6$ days)	Extended aeration ($\theta_c = 22$ days)
SS biodegradable volatile	26.4	2.4
SS non-biodegradable volatile	16.8	25.8
SS volatile (biodegradable + non biodegradable)	43.2	28.2
SS inorganic	6.7	6.7
SS total	49.9	34.9
Ratio VSS/SS (%)	*87*	*81*
Destruction of biodegradable SS (%)	*39*	*93*
Destruction of volatile SS (%)	*28*	*53*

d) Schematics of the production of the biological solids

See figure below.

e) Comments

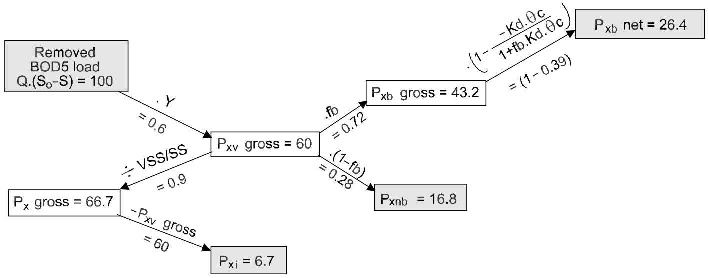
- The example demonstrates that the highest production of solids is in systems with lower sludge age (e.g. conventional activated sludge), compared with the production in systems with higher sludge age (e.g. extended aeration).
- If the inorganic and inert solids of the raw sewage had been considered (which is the case, in practice), the total production values would have been different, as well as the VSS/SS ratio.

Example 9.10 (*Continued*)

- The systems with a high sludge age (e.g. extended aeration) lead to a high solids digestion (in this example, 93% removal efficiency of biodegradable suspended solids and 53% removal of the volatile suspended solids produced), compared with systems with a low sludge age (e.g. conventional activated sludge – in this example, 39% efficiency in the removal of biodegradable suspended solids and 28% in the removal of the volatile suspended solids produced).

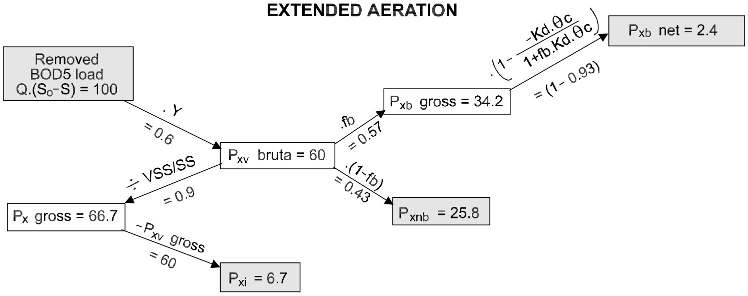
Production of biological solids (kg/d)
(Example 9.10)

CONVENTIONAL ACTIVATED SLUDGE



Final production
SS biodegradable volatile = 26.4 kg/d
SS non-biodegradable volatile = 16.8 kg/d
SS volatile = 26.4 + 16.8 = 43.2 kg/d
SS inorganic = 6.7 kg/d
SS total = 43.2 + 6.7 = 49.9 kg/d

EXTENDED AERATION



Final production
SS biodegradable volatile = 2.4 kg/d
SS non-biodegradable volatile = 25.8 kg/d
SS volatile = 2.4 + 25.8 = 28.2 kg/d
SS inorganic = 6.7 kg/d
SS total = 28.2 + 6.7 = 34.9 kg/d

10

Sedimentation

10.1 INTRODUCTION

Sedimentation is the physical operation that separates solid particles with a density higher than that of the surrounding liquid. In a tank in which the water flow velocity is very low, the particles tend to go to the bottom under the influence of gravity. As a result, the supernatant liquid becomes clarified, while the particles at the bottom form a sludge layer, and are then subsequently removed with the sludge. Sedimentation is a unit operation of high importance in various wastewater treatment systems.

The main applications of sedimentation in wastewater treatment are:

- **Preliminary treatment**. *Grit removal* (sedimentation of inorganic particles of large dimensions)
    - Grit chamber
- **Primary treatment**. *Primary sedimentation* (sedimentation of suspended solids from the raw sewage)
    - Conventional primary clarifiers, with frequent sludge removal
    - Septic tanks
- **Secondary treatment**. *Secondary sedimentation* (removal of mainly biological solids)
    - Final sedimentation tanks in activated sludge systems
    - Final sedimentation tanks in trickling filter systems
    - Sedimentation compartments in anaerobic sludge blanket reactors
    - Sedimentation ponds, after complete-mix aerated lagoons

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- **Sludge treatment**. *Thickening* (settling and thickening of primary sludge and/or excess biological sludge)
    - Gravity thickeners
- **Physical–chemical treatment**. *Settling after chemical precipitation*
    - Enhancement of the performance of primary clarifiers
    - Polishing of effluents from secondary treatment
    - Chemical nutrient removal
    - Physical–chemical treatment (chemical coagulation) of mainly industrial wastewater, but also domestic wastewater

Besides these, sedimentation occurs in various other wastewater treatment units, such as stabilisation ponds, even if they have not been specifically designed for this purpose.

The main objective in most of the applications is to produce a *clarified effluent*, that is, with a low suspended solids concentration. However, at the same time it is also frequently desired to obtain a *thickened sludge* to help its subsequent treatment.

Figures 10.1 and 10.2 present the schematics of two types of settling tanks, one rectangular with horizontal flow, and the other circular with central feeding. Details about the design of these settling tanks are presented in the relevant chapters of this book, related to the various wastewater treatment processes. In the present chapter, only the basic principles of sedimentation are presented.

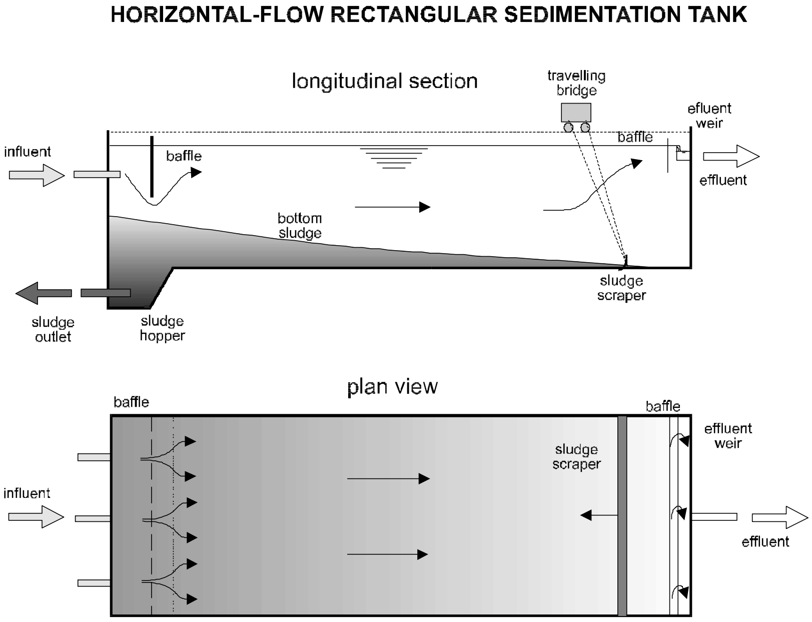


Figure 10.1. Schematics of a rectangular settling tank with horizontal flow

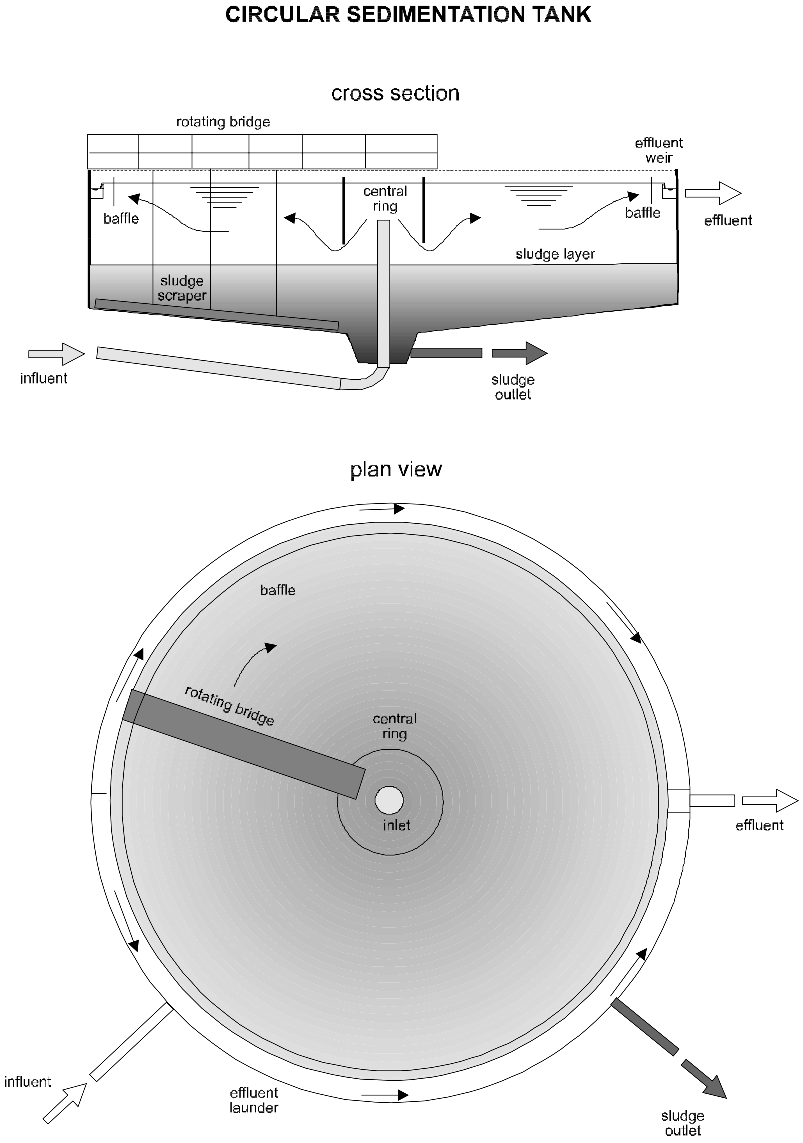
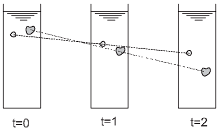
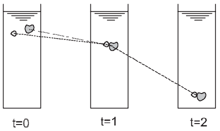
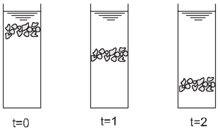
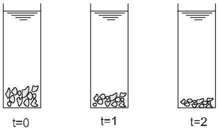


Figure 10.2. Schematics of a circular settling tank with central feeding

10.2 TYPES OF SETTLING

In wastewater treatment, there are basically the four different types of settling described in Table 10.1. It is probable that during a settling operation more than one type occurs at a given time.

Table 10.1. Settling types in wastewater treatment

Type	Scheme	Description	Example of application/ Occurrence
Discrete		The particles settle, maintaining their identity, that is, they do not coalesce. Hence, their physical properties such as shape, size and density are preserved.	• Grit chambers
Flocculent		The particles coalesce while settling. Their characteristics are changed, with an increase in size (floc formation) and, as a result, in the settling velocity.	• Primary sedimentation tanks • Upper part of secondary sedimentation tanks • Chemical flocs in physical–chemical treatment
Hindered (or zone)		When there is a high concentration of solids, a sludge blanket is formed, which settles as a single mass (the particles tend to stay in a fixed position with relation to the neighbouring particles). A clear separation interface can be observed between the solid phase and the liquid phase. The interface level moves downwards as a result of the settling of the sludge blanket. In this case, it is the settling velocity of the interface that is used in the design of the settling tanks.	• Secondary sedimentation tanks • Sludge gravity thickeners
Compression		If the solids concentration is even higher, the settling could occur only by compression of the particles' structure. The compression occurs due to the weight of the particles, constantly added because of the sedimentation of the particles situated in the supernatant liquid. With the compression, part of the water is removed from the floc matrix, reducing its volume	• Bottom of secondary sedimentation tanks • Sludge gravity thickeners

Source: adapted from Tchobanoglous and Schroeder (1985), Metcalf and Eddy (1991)

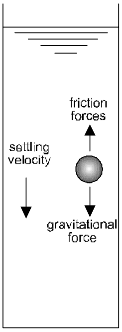


Figure 10.3. Interacting forces in a particle under discrete settling

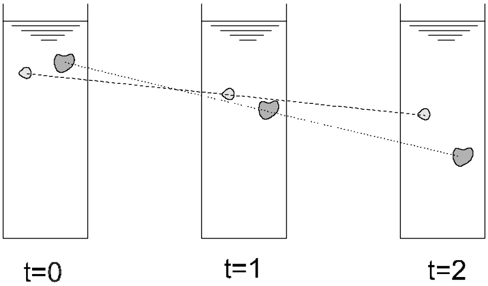


Figure 10.4. Discrete settling, showing constant settling velocity of the particles

10.3 DISCRETE SETTLING

10.3.1 Settling velocity

The sedimentation of discrete particles can be analysed through the classic laws of Newton and Stokes. According to these laws, the final velocity of a particle under sedimentation in a liquid is constant, that is, the frictional force is equal to the gravitational force. This terminal velocity is reached in the liquid medium in fractions of a second. Figure 10.3 shows the intervening forces on a settling particle, while Figure 10.4 emphasises the fact that the settling velocity of discrete particles is constant.

According to Stokes law, the discrete settling velocity of a particle (v_s) in laminar flow is given by:

$$\boxed{v_s = \frac{1}{18}.\frac{g}{\upsilon}.\frac{\rho_s - \rho_l}{\rho_l}.d^2} \tag{10.1}$$

where:

v_s = settling velocity of the particle (m/s)
g = acceleration due to gravity (m/s^2)
υ = kinematic viscosity of the liquid (m^2/s)

ρ_s = particle density (kg/m^3)
ρ_l = liquid density (kg/m^3)
d = particle diameter (m)

The kinematic viscosity ν and the density of water ρ_l are functions of the temperature T. However, the variation in the density of the water within the usual temperature ranges in wastewater treatment can be neglected (999.8 kg/m^3 and 992.2 kg/m^3 for temperatures of 0 °C and 40 °C, respectively) and a value of 1000 kg/m^3 can be adopted. The influence on the water viscosity is more representative, as seen in Table 10.2 (Tchobanoglous and Schroeder, 1985; Huisman, 1978).

Table 10.2. Kinematic viscosity of the water as a function of temperature

T (°C)	0	5	10	15	20
υ(m^2/s)	1.79×10^{-6}	1.52×10^{-6}	1.31×10^{-6}	1.15×10^{-6}	1.01×10^{-6}
T (°C)	25	30	35	40	
υ(m^2/s)	0.90×10^{-6}	0.80×10^{-6}	0.73×10^{-6}	0.66×10^{-6}	

In the range of T = 10 to 30 °C, von Sperling (1999) proposes the following equation for the viscosity as a function of the temperature ($R^2 = 0.986$):

$$\nu = 3.76 \times 10^{-6} \times T^{-0,450} \qquad (10.2)$$

When interpreting Equation 10.1, the following considerations are important:

- *v_s is proportional to $(\rho_s - \rho_l)/\rho_l$*
- *v_s is proportional to d^2*

The fact that v_s is proportional to the square of the particle diameter emphasises the importance of the increase in the size of the particles, aiming at a faster particle removal, and, consequently, smaller sedimentation tanks. As an example, when the particle diameter doubles, the settling velocity increases four times.

Example 10.1

Calculate the settling velocity of a sand grain using the following data:

- Grain diameter: d = 0.7 mm
- Sand density: $\rho_s = 2650$ kg/m^3
- Liquid density: $\rho_l = 1000$ kg/m^3
- Liquid temperature: T = 25 °C

Solution:

From Table 10.2, for the temperature of 25 °C, the kinematic viscosity of the water υ is 0.90×10^{-6} m^2/s. The diameter of the particle is 0.7×10^{-3} m. From Equation 10.1, assuming laminar flow:

$$v_s = \frac{1}{18}.\frac{g}{\upsilon}.\frac{\rho_s - \rho_l}{\rho_l}.d^2 = \frac{1}{18}.\frac{9.81}{0.90 \times 10^{-6}}.\frac{2650 - 1000}{1000}.(0.7 \times 10^{-3})^2$$
$$= 0.49 \text{ m/s}$$

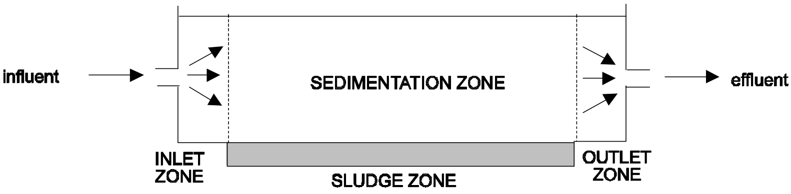


Figure 10.5. Schematic representation of the zones in a horizontal sedimentation tank (longitudinal section)

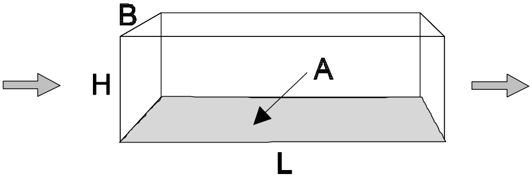


Figure 10.6. Dimensions in the sedimentation zone

10.3.2 The concept of an ideal sedimentation tank with horizontal flow

The discrete settling of a particle can be analysed in a settling column without flow as well as in a rectangular horizontal-flow tank with constant horizontal velocity (v_h). Figure 10.5 shows the representative zones of this ideal tank. The theoretical considerations apply to the zone where settling effectively occurs (sedimentation zone).

For the theoretical analysis of sedimentation, it is necessary to assume that:

- the particles are uniformly distributed in the inlet zone
- the particles that touch the sludge zone are considered removed
- the particles that reach the outlet zone are not removed by sedimentation

The main dimensions of the *sedimentation zone* are presented in Figure 10.6.

In an ideal sedimentation tank with constant horizontal velocity, the discrete settling of a particle occurs as in a sedimentation column (see Figure 10.7).

The *time* taken for a particle to reach the bottom is given by:

- *sedimentation column*: time = distance/velocity

$$t = \frac{H}{v_s} \tag{10.3}$$

- *horizontal flow tank*: time = volume/flow

$$t = \frac{V}{Q} = \frac{H.A}{Q} \tag{10.4}$$

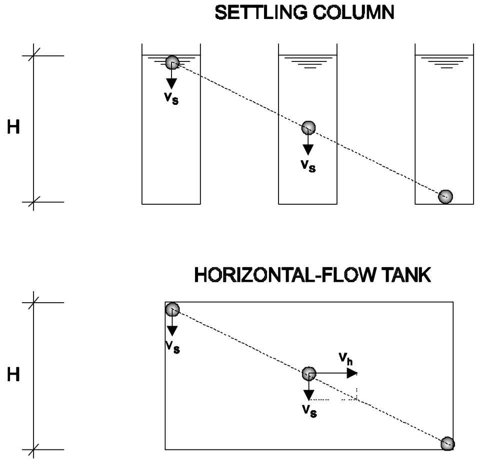


Figure 10.7. Discrete settling of a particle in a sedimentation column and in a horizontal tank

Combining Equations 10.3 and 10.4:

$$\boxed{v_s = \frac{Q}{A}} \tag{10.5}$$

This equation is very important in the design of sedimentation tanks. If it is desired to remove particles with settling velocities equal to or greater than v_s, and knowing the wastewater flow to be treated Q, the required surface area can be obtained from:

$$\boxed{A = \frac{Q}{v_s}} \tag{10.6}$$

The settling velocity to be adopted for design (v_s, or v_o) is also called *overflow rate* or *hydraulic surface loading rate*, and is expressed in units of velocity (m/h), or flow per unit area ($m^3/m^2.h$).

In the interpretation of Equation 10.5, it should be noted that:

- v_s can be obtained through experiments with the liquid to be treated or from literature values (in a design, v_s is a design parameter)
- the removal of discrete particles depends only on the surface area (A) and not on the height (H) and time (t).

The last point can be understood as follows. If A and Q are maintained constant, and if H doubles, the volume V doubles, and so does the time t (see Equation 10.4). The horizontal velocity v_h ($v_h = Q/(B.H)$) is reduced to half. Since v_s is constant (function only of the particle characteristics), the new trajectory of the particle

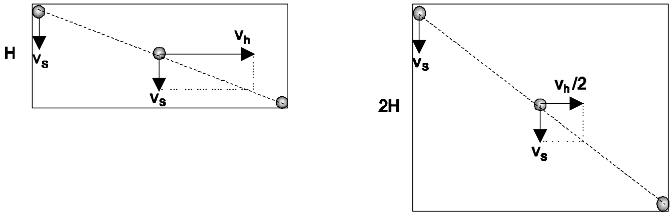


Figure 10.8. Visualisation of the non-influence of H on the removal of discrete particles.

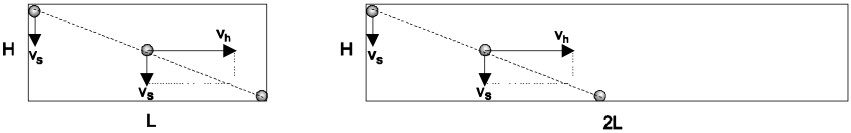


Figure 10.9. Visualisation of the influence of A on the removal of discrete particles

leads to its removal in the final extremity of the tank, identically to the tank with a lower height (see Figure 10.8).

However, if the surface area A doubles, for example through the duplication of the length L, v_h, and v_s remain constant. The trajectory of the particle is not altered, but the particle is removed in half of the tank length (see Figure 10.9). Hence, this new tank is able to receive particles with settling velocities lower than v_S. In summary, *for the* ***ideal discrete settling****, the surface area A is of fundamental importance, while H and t do not play any role.*

The particles to be removed in a sedimentation tank depend on the:

- settling velocity of the particle (compared with the design settling velocity v_S)
- height at which the particle enters the sedimentation zone

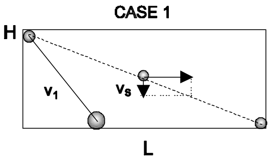
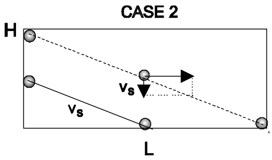
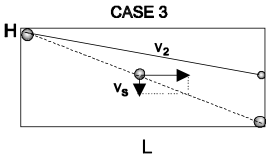
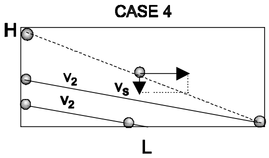
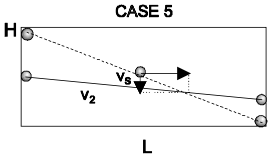
In a tank removing particles originating from different vertical positions and with different settling velocities there are the possibilities shown in Table 3.2.

10.3.3 Discrete settling tests

A large diversity of particle sizes occurs in a typical suspension of particulate matter. To determine the removal efficiency in a certain time, it is necessary to consider the whole range of the settling velocities found in the system. This is usually done through tests in a settling column, in which samples are extracted from various depths and times (Metcalf & Eddy, 1991).

The settling column (see Figure 10.10) needs sampling points at various levels. Typical analyses made with the samples are suspended solids (SS), which allow

Table 10.3. Discrete particles to be removed in a horizontal flow tank

Case	Particles removed or not removed
	*Particles removed:* • particles with a settling velocity equal to v_s that enter the tank at a height H • particles with a settling velocity $v_1 > v_s$ that enter the tank at a height H
	*Particles removed:* • particles with a settling velocity equal to v_s that enter the tank at a height lower than H
	*Particles not removed:* • particles with a settling velocity $v_2 < v_s$ that enter the tank at a height H
	*Particles that can be removed:* • particles with a settling velocity $v_2 < v_s$ that enter the tank at a height lower than H
	*Particles that may be not removed:* • particles with a settling velocity $v_2 < v_s$ that enter the tank at a height lower than H

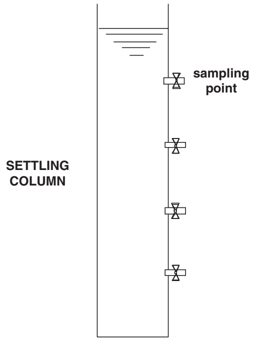


Figure 10.10. Settling column

the evaluation of the removal efficiency of the particulate matter. At the start of the test, the column should be full of a homogeneous mixture of the suspension. The samples are taken at different times, at the various sampling points along the column height. If the initial concentration in the column at the initial time $t_o = 0$ is C_o, and after a time t_i, the concentration reduces to C_i at the depth z_i, then $C_o - C_i$ of the original suspension has settling velocities greater then $z_i/(t_i - t_o)$. Repeating this concept for different depths and times, the cumulative curve of the proportion of particles with settling velocities lower than the X-axis value can be constructed (Wilson, 1981).

Example 10.2 (adapted from Wilson, 1981) presents a methodology for the determination of the removal efficiency of discrete particles based on a settling test.

Example 10.2

The results of a settling test done in a suspension with particles that present discrete settling led to the values presented below. Plot the cumulative profile of the settling velocity and calculate the fraction of particles removed for an overflow rate of $v_o = 1.0$ m/h.

Sample	Sampling depth (m)	Sampling time (h)	SS in the sample (mg/L)
1	0.0	0.0	222 (average)
2	0.0	0.0	
3	0.0	0.0	
4	1.0	1.0	140
5	1.0	3.0	108
6	1.0	6.0	80
7	2.0	1.0	142
8	2.0	3.0	110
9	2.0	6.0	106
10	3.0	1.0	142
11	3.0	3.0	130
12	3.0	6.0	124
13	4.0	1.0	147
14	4.0	3.0	126
15	4.0	6.0	114

Solution:

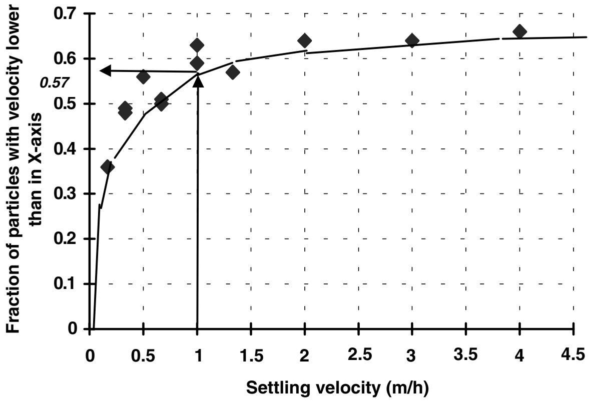
a) Plot the curve of the fraction of particles × settling velocity

Consider *sample 9* (depth of 2.0 m and sample time of 6.0 h). The settling velocities of the particles found in the sample are less than 2.0 m/6.0 h = 0.33 m/h. Hence, (106/222) = 0.48 = 48% of the particles have a settling velocity lower than 0.33 m/h. The removal efficiency in this sample is 1 − 0.48 = 0.52 = 52%. With this reasoning, the table below can be constructed.

Example 10.2 (*Continued*)

Sample	Velocity (m/h)	Fraction of the SS remaining
4	1.00	0.63
5	0.33	0.49
6	0.17	0.36
7	2.00	0.64
8	0.67	0.50
9	0.33	0.48
10	3.00	0.64
11	1.00	0.59
12	0.50	0.56
13	4.00	0.66
14	1.33	0.57
15	0.67	0.51

Based on the values from the table above, a graph of the settling velocities × remaining SS fraction can be constructed. Note that the sampling depth has no influence on the discrete settling test.



b) Determination of the fraction of particles removed

From the figure above, 0.57 (57%) of the particles have a settling velocity lower than 1.0 m/h. Thus, the fraction removed of these particles, if they started settling from the top of the column, would be 1 − 0.57 = 0.43 (43%).

There is still another fraction removed, which corresponds to the particles with a settling velocity lower than v_o, which did not start settling from the top of the column (or the top of the horizontal tank). The removal fraction of these particles is given by the area occupied between the Y-axis and the curve until x = 3.0 m/h. This can be obtained from the table below, which presents the

Example 10.2 (Continued)

calculation of the area based on a division into strips, with the width (dx_i) and the average velocity in the strip (vx_i):

Strip of dx_i *(Y axis)*	Width of the strip (dx_i) *(Y axis)*	Average velocity in the strip (vx_i) (m/h) *(X axis)*	$dx_i.vx_i$
0.50–0.57	0.07	0.80 (at $y = 0.54$)	0.056
0.40–0.50	0.10	0.36 (at $y = 0.45$)	0.036
0.30–0.40	0.10	0.14 (at $y = 0.35$)	0.014
0.20–0.30	0.10	0.05 (at $y = 0.25$)	0.005
0.10–0.20	0.10	$\simeq 0$ (at $y = 0.15$)	–
0.00–0.10	0.10	$\simeq 0$ (at $y = 0.05$)	–
Total	–	–	0.111

The fraction removed in this second way is:

$$\frac{\sum dx_i.vx_i}{v_0} = \frac{0.111}{1.0} = 0.11$$

The total fraction removed is $0.43 + 0.11 = 0.54$ (54%).

Therefore, for the overflow rate of 1.0 $m^3/m^2.h$, 54% of the particles in the sampled suspension are removed.

If desired, a curve showing the removal efficiencies as a function of the overflow rate (v_o) can be constructed, based on a repetition of the calculations for different values of v_o.

10.4 FLOCCULENT SETTLING

In flocculent settling, the particles agglomerate themselves and form flocs that tend to grow in size while settling. With the increase in the size of the particles (flocs), there is an increase in the settling velocity. Therefore, in flocculent settling the velocity is not constant as in discrete settling, but tends to increase. Figures 10.11 and 10.12 show the floc formation process and the increase in the settling velocity in a horizontal flow tank (Figure 10.11) and in a settling column (Figure 10.12).

Since the flocculation occurs while the particles go to the bottom, the greater the chance of contact they have, the greater will be the floc formation. As a result, *the removal efficiency in flocculent settling is increased with an increase in the depth H and in the time t* (differently from discrete settling).

Similarly to discrete settling, flocculent settling in an ideal horizontal flow tank can be compared with settling in a column without flow.

In the case of flocculent settling, the settling velocity of the individual particles is not analysed, as in the case of discrete settling. Settling column tests are also

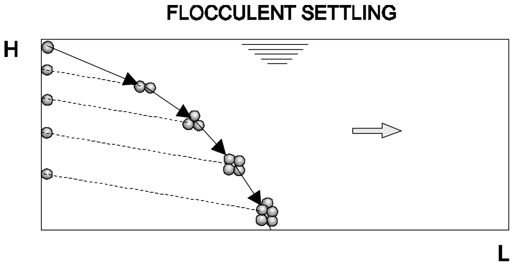


Figure 10.11. Flocculent settling in a horizontal flow tank

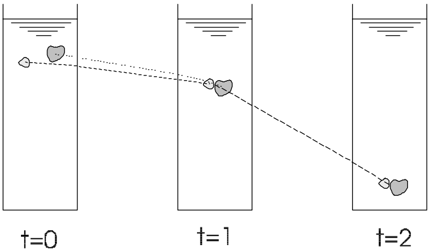


Figure 10.12. Flocculent settling in a settling column

useful here to permit the selection of the ideal overflow rate v_o. In the flocculent settling test, the results are presented in the form of curves or a grid, showing the particle removal percentages at certain depths and times (see Example 10.3, adapted from Wilson, 1981).

Example 10.3

Assuming the values presented in the table below, calculate the expected percentage removal in the following cases:

- 2.0 m deep tank, with a detention time of 1.50 h
- 2.0 m deep tank, with a detention time of 3.00 h
- 1.0 m deep tank, with a detention time of 3.00 h
- 2.5 m deep tank, with a detention time of 3.75 h

Percentage removal (%)

	Sampling time (h)							
Sampling depth (m)	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
1.0	26	44	49	55	63	66	71	77
2.0	20	34	44	51	56	60	62	64
3.0	19	27	37	45	51	57	60	68

***Example 10.3* (*Continued*)**

Solution:

The percentage removal efficiencies are calculated based on the initial concentration C_o, and on the concentration at the sampling time. For example, if the initial concentration were $C_o = 200$ mg/l, and the concentration at the depth and time of sampling is 132 mg/l, the removal efficiency would be $(200–132)/200 = 0.34 = 34\%$. This could have been the case, for example for a sample at a 2.0m depth and 1.0h of sampling.

a) Tank with 2.0 m depth and detention time of 1.50 h

For the depth of 2.0 m and sampling time of 1.5 h, the percentage removal can be obtained directly from the table. The value found is 44%.

b) Tank with 2.0 m depth and detention time of 3.0 h

For the depth of 2.0 m and sampling time of 3.0 h, the percentage removal shown in the table is 60%. The depth is the same as in item a, but the detention time (or the volume) is the double. Since both depths are the same, but the volumes are different, then the surface area of item b is the double of that in item a. The comparison with the calculation of item a, in terms of design, is that the maintenance of the same depth of 2.0 m but with a duplication of the volume, surface area or detention time (3.0 h compared with 1.5 h) leads to an increase of the removal efficiency from 44% to 60%.

c) Tank with 1.0 m depth and detention time of 3.0 h

For the depth of 1.0 m and sampling time of 3.0 h, the percentage removal expressed in the table is 66%. The detention time (or the volume) is the same as item b, but the depth is half of that. Since both volumes are equal, but the depth is different, then the surface area of item c is double that of item b. A comparison with the calculation of item b, in terms of design, is that the maintenance of the same volume but with a reduction in the depth and duplication of the surface area leads to an increase of the removal efficiency from 60% to only 66%.

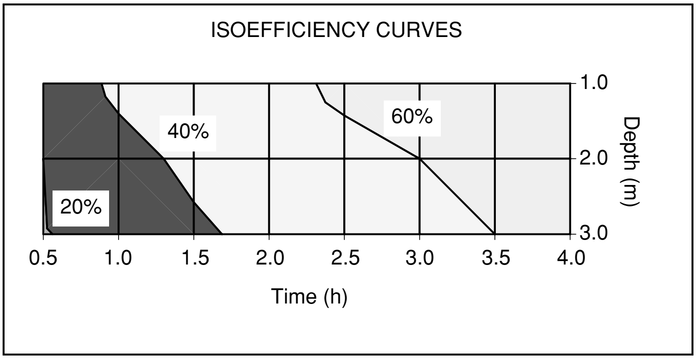
d) Tank with 2.5 m depth and detention time of 3.75 h

For the depth of 2.5 m and sampling time of 3.75 h, an interpolation in the grid is necessary. Adopting a linear interpolation:

$$\% \text{ removal} = \frac{1}{2}.\left(\frac{64+62}{2}+\frac{60+68}{2}\right) = 63.5\%$$

***Example 10.3* (*Continued*)**

As an illustration, the isoefficiency curves obtained directly from the data presented in the problem are shown below.



10.5 ZONE SETTLING

10.5.1 Settling in a column

When there is a high solids concentration, a blanket tends to form. This blanket settles as a single mass of particles (the particles tend to remain in a fixed position with relation to the neighbouring particles). A clear separation interface can be observed between the solid phase and the liquid phase, and the level of the interface moves downwards as a result of the sedimentation of the sludge blanket (see Figure 10.13). For the blanket to move downwards, the liquid situated underneath tends to move upwards. In the zone settling, it is the settling velocity of the interface that is used in the design of sedimentation tanks. *Zone settling* is also called *hindered settling*.

In a *settling column* completely homogenised with a high concentration of suspended solids, under quiescent conditions and after a short time, a clear interface

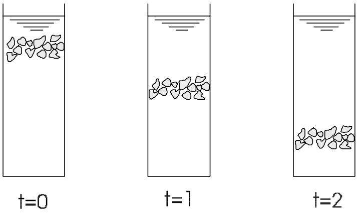


Figure 10.13. Zone settling of a solids mass in a settling column

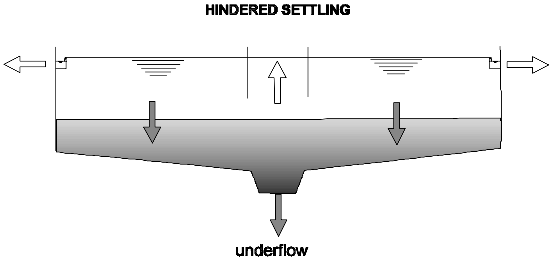


Figure 10.14. Sludge removal from the bottom in secondary sedimentation tanks

is formed. While the interface moves downwards, the supernatant liquid becomes clarified, and a layer with a higher concentration is formed at the bottom. The level of this highly concentrated layer moves upwards due to the continuous increase of the accumulated material at the bottom, which cannot leave the column from its bottom.

In a *sedimentation tank* with continuous withdrawal of the settled sludge from the bottom, the more concentrated layer does not propagate upwards. The reason is that the underflow velocity of the sludge (downward, from the bottom) counterbalances the expansion velocity (upwards). This situation occurs in tanks with continuous sludge removal from the bottom, such as secondary sedimentation tanks in the activated sludge process (see Figure 10.14).

Figure 10.15 presents schematically the behaviour of the layers created in these two distinct conditions (*without* and *with* sludge removal from the bottom).

10.5.2 The limiting solids flux theory

The solids flux theory describes the zone settling phenomenon that takes place in secondary sedimentation tanks and gravity thickeners. The solids flux theory is a result of the sequential development from many authors, but achieved a greater applicability in the context of wastewater treatment based on the works of Dick (1972). Its utilisation can be for design as well as for operational control. Within a global view of the treatment system, the theory can be used together with a mathematical model of the reactor in order to allow an optimal design of the system (Keinath et al, 1977; Catunda and van Haandel, 1987) or its optimal control (von Sperling, 1990).

The following items focus on the behaviour of a secondary sedimentation tank in an activated sludge plant, due to their greater importance when compared with gravity thickeners. However, the general principles are the same in both cases.

In this context, *flux* can be understood as the solids load per unit area (for example expressed as kgSS/m^2.h). In a continuous flow sedimentation tank

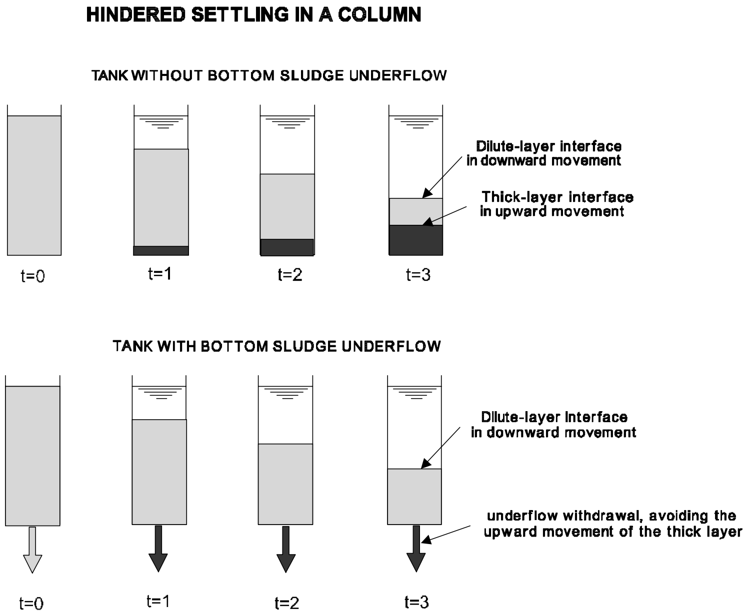


Figure 10.15. Schematic representation of the behaviour of the diluted and concentrated sludge layers in the zone settling. (a) Column without sludge removal from the bottom. (b) Settling tank with sludge removal from the bottom.

the solids tend to go to the bottom due to the simultaneous action of two fluxes:

- *gravity flux* (G_g), caused by the gravitational sedimentation of the sludge;
- *underflow flux* (G_u), caused by the downward movement originating from the removal of the return sludge from the bottom of the sedimentation tank.

The total flux (G_t) moving to the bottom of the sedimentation tank corresponds to the sum of these two components. The mathematical representation of these fluxes can be expressed as (Dick, 1972):

Total flux:

$$\boxed{G_t = G_g + G_u} \quad (10.7)$$

Gravity flux:

$$\boxed{G_g = C.v} \quad (10.8)$$

Underflow flux:

$$\boxed{G_u = C.\frac{Q_u}{A}} \quad (10.9)$$

where:

C = concentration of suspended solids in the sludge (kg/m^3)
v = settling velocity of the interface at the concentration C (m/h)
Q_u = underflow from the bottom (m^3/h)
A = surface area of the sedimentation tanks (m^2).

The settling velocity v is, on the other hand, a function of the concentration C itself, decreasing with the increase of C. There are various empirical relations to express v in function of C, but the most frequently used is:

$$\boxed{v = v_o.e^{-K.C}} \tag{10.10}$$

where:

v_o = coefficient, expressing the interface settling velocity at a concentration C = 0 (m/h)
K = settling coefficient (m^3/kg)

The flux of solids conveyed to the bottom of the sedimentation tank depends on the concentration C, according to the following conditions:

- **Low concentration of C**. With low values of **C**, the settling velocity of the interface **v** is high (Equation 10.10), but the product **C.v** is low, which results in a low value of the gravity solids flux (Equation 10.8).
- **Intermediate concentration of C**. While **C** increases, even with the decrease in **v**, the product **C.v** increases, that is, the gravity flux increases.
- **High concentration of C**. However, after a certain value of C, the reduction in the settling velocity **v** is such that the product **C.v** starts to decrease.

Figure 10.16a presents the curve of the gravity solids flux ($G_g = C.v$). The intercept of the straight line with slope Q_u/A, tangent to the descending reach of the flux curve, with the Y-axis characterises the **limiting flux** (G_L). This can be understood as the maximum flux that can be transported to the bottom of the sedimentation tank with the existing settleability, sludge concentration and underflow. The same interpretation can be obtained from Figure 10.16b, in which the total flux ($G_t = G_g + G_u$) is presented. The result is the same, but in this case, the limiting flux is obtained at the minimum of the total flux curve. This point of minimum indicates that, while the solids concentration increases in the settling tanks from the inlet to the bottom, there will be a concentration (limiting concentration C_L) that will bring about the lowest flux (limiting flux G_L). At this point, the settling tank is limited and cannot transmit to the bottom a quantity of solids higher than the limiting value. The construction of the solids flux curves is presented in Sections 10.5.3 and 10.5.5.

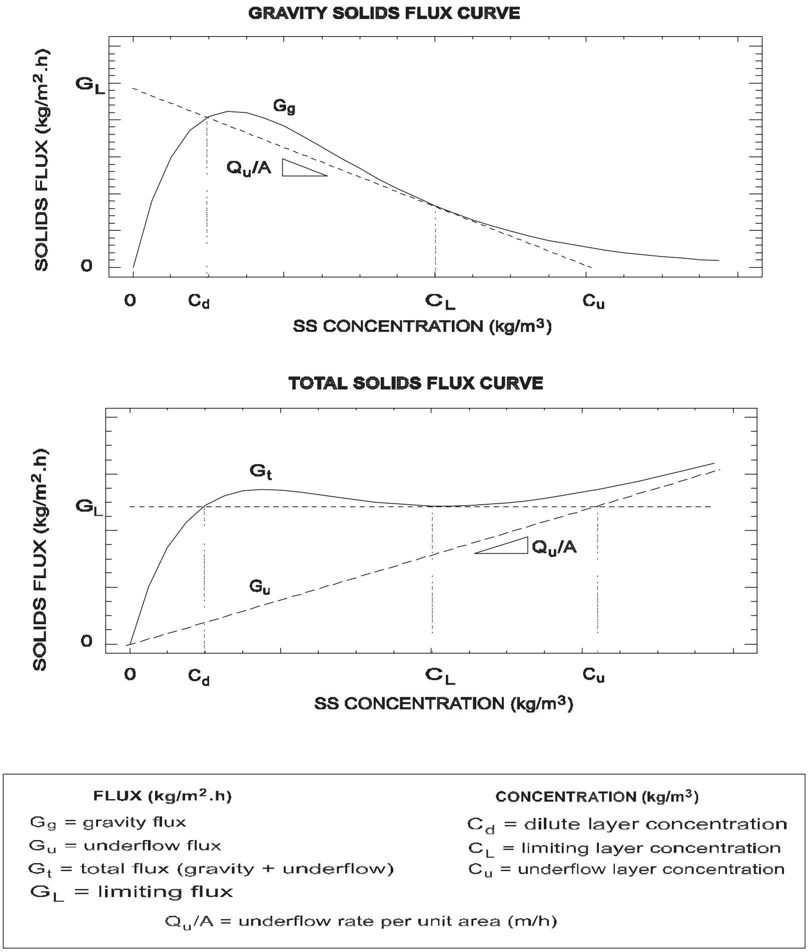


Figure 10.16. Solids flux curves. (a) Gravity solids flux. (b) Total solids flux.

The success of the design and operation of secondary sedimentation tanks depends on the relation between the *applied flux* and the *limiting flux*. The applied flux corresponds to the load of the influent suspended solids to the settling tank per unit surface area, given by (see Figure 10.17):

$$\boxed{G_a = \frac{Q_i + Q_r}{A}.C_o} \tag{10.11}$$

where:

G_a = applied solids flux (kg/m^2.h)

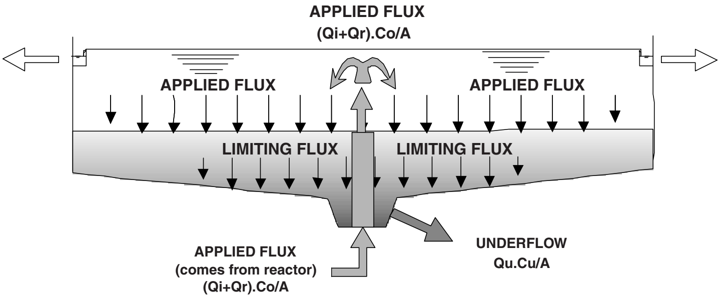


Figure 10.17. Applied solids flux on a secondary sedimentation tank

Q_i = influent flow to the sewage treatment plant (m^3/h)
Q_r = return sludge flow (approximately equal to the sludge underflow) (m^3/h)
C_o = concentration of influent suspended solids to the secondary sedimentation tank. Equal to the suspended solids concentration in the reactor, or mixed liquor suspended solids concentration (MLSS) (kg/m^3).

In practical terms, it can be considered that Q_r is equal to Q_u, since the excess sludge flow ($Q_u - Q_r$) is negligible in the mass balance on the secondary sedimentation tank.

In broad terms, the applied flux must be equal to or less than the limiting flux ($G_a \leq G_L$), so that the settling tank does not accumulate solids, which could eventually reach a quantity that would lead to their loss in the supernatant of the settling tank, thus deteriorating the final effluent quality. On the curve of the gravity flux (Figure 10.16a), the straight line of the applied flux can be drawn. This line starts at the Y-axis (at the value G_a) and goes downwards, in parallel with the line of the limiting flux (slope equal to Q_u/A).

Figure 10.18 presents an interpretation, by the author, of the theory presented by Dick (1972), Handley (1974), White (1976) and Keinath (1981), in terms of the relation between the gravity flux curve and the profile of the suspended solids concentration in the secondary sedimentation tank. In this figure, there are four columns, representing four distinct conditions: (a) *sedimentation tank with underload*, (b) *sedimentation tank with critical load*, (c) *sedimentation tank with thickening overload* and (d) *sedimentation tank with thickening and clarification overload*. The first line presents the curve of the **gravity flux** (equal in the four columns), the **limiting flux** (also equal in the four columns) and the **applied flux** (different in the four columns). The second line presents the vertical profile of the suspended solids concentration resulting from the interrelation between the applied and the limiting fluxes. The third line shows this profile as a cross-section in the settling tank.

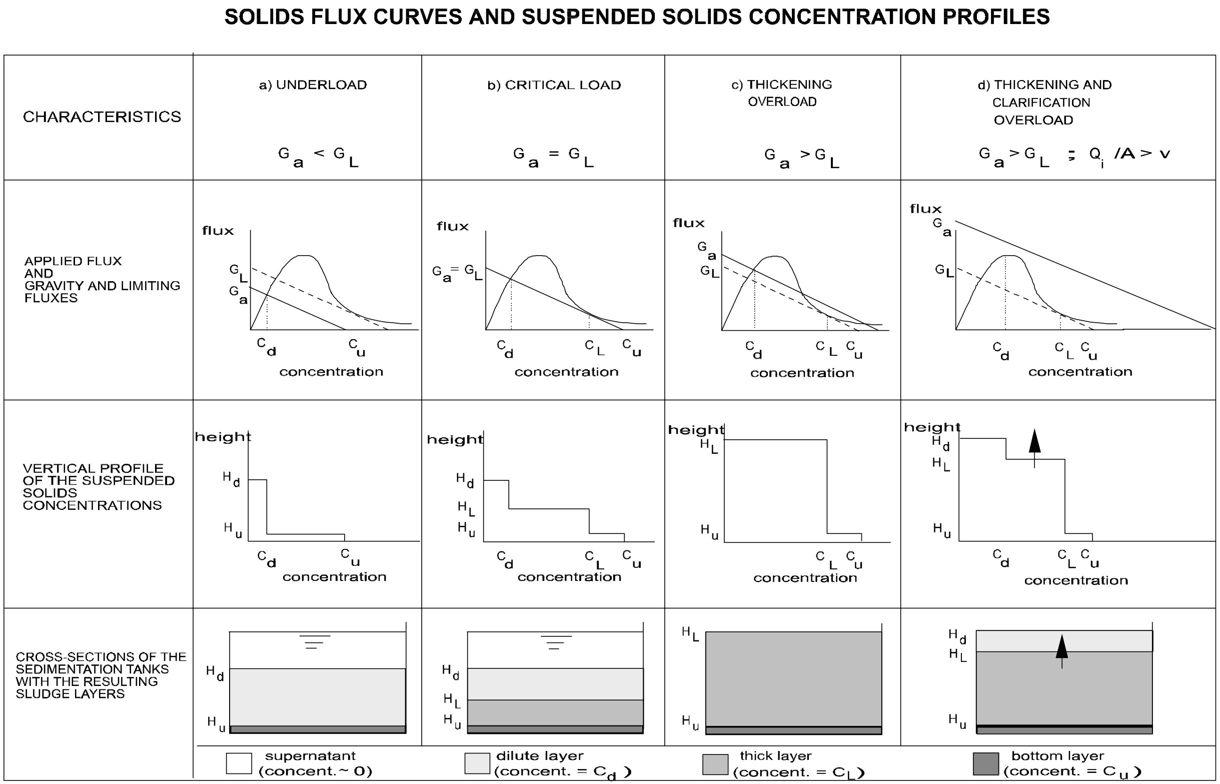


Figure 10.18. Solids flux curves and suspended solids concentration profiles in secondary sedimentation tanks, for various relations between the applied and the limiting fluxes.

The following points are associated with the interpretation of Figure 10.18:

- **Sedimentation tank with underload**. The settling tank will be underloaded when the applied flux is less than the limiting flux. In this condition, only a diluted layer with a low suspended solids concentration (C_d) will be formed. At the bottom of the settling tank a layer with concentration C_u (concentration of the sludge removed) will also develop, due to the support by the tank bottom.
- **Sedimentation tank with critical load**. The settling tank will be critically loaded when the applied flux is equal to the limiting flux. In this case, a thicker sludge layer (concentration C_L) will be formed.
- **Sedimentation tank with thickening overload**. The settling tank will be overloaded in terms of thickening of the sludge when the applied flux is greater than the limiting flux. In this condition, the concentration of the thick sludge layer will not go beyond C_L and, consequently the thick sludge layer will increase in volume, propagating upwards. Depending on the level reached by the sludge blanket, solids may be discharged with the final effluent.
- **Sedimentation tank with thickening and clarification overload**. The settling tank will be overloaded in terms of thickening and clarification when, besides having an applied flux greater than the limiting flux, the overflow rate (Q_i/A) is greater than the sludge settling velocity v. In this case, the diluted layer as well as the thick layer will propagate upwards, with a possible even faster deterioration of the effluent quality.

10.5.3 Determination of the interface settling velocity

The settling velocity of the interface, also called *zone settling velocity* (ZSV) can be determined experimentally through tests in a settling column. For this, the following simplified methodology is suggested:

- homogenise the liquid through mixing in the whole tank (column) volume
- interrupt mixing to allow sedimentation
- measure the interface level at various time intervals
- stop the measurement when the interface is not significantly settling any more
- plot a graph: interface height (Y axis) × time (X axis)
- determine the settling velocity of the interface by the slope of the straight-line reach in the graph (ignore the initial and final points that are not on the straight-line reach)

The test is commonly done in cylinders up to 0.5 m in height and 10 cm in diameter. However, whenever possible, it is desirable to use higher columns (around 2.0 m or greater), so that they are representative of the height of full-scale sedimentation tanks.

The test of the zone settling velocity can be done for various values of the initial concentration C_o, in order to allow the derivation of the parameters v_o and K from Equation 10.10.

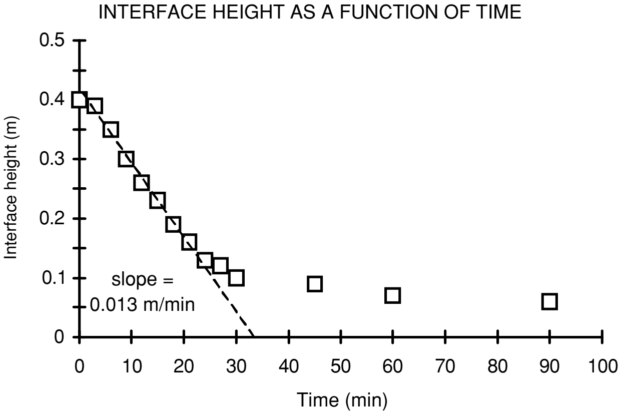
Example 10.4

Determine the zone settling velocity of a suspension of activated sludge. The initial solids concentration of the mixed liquor in the column was equal to 2,900 mg/L. The following values of the interface height were measured as a function of time:

t (min)	0	3	6	9	12	15	18	21	24	27	30	45	60	90
H (m)	0.40	0.39	0.35	0.30	0.26	0.23	0.19	0.16	0.13	0.12	0.10	0.09	0.07	0.06

Solution:

a) Plot the results on a graph



b) Determine the slope of the straight line reach in the curve H × t

Neglecting the initial point (t = 0 min) and the points starting from t = 21 min, which are not part of a straight line, the line of best fit is adjusted between the points t = 3 min and t = 18 min. The best line fit and its slope can be obtained graphically or through linear regression analysis. The values obtained graphically lead to the following value for the zone settling velocity:

$$v = 0.013\ \text{m/min} = 0.78\ \text{m/h}$$

To determine the values of the coefficients v_o and K from Equation 10.10, various settling tests should be undertaken for different values of the initial concentration C. In this way, distinct values of the pair v and C (zone settling sedimentation velocity × initial concentration) are obtained, allowing the determination of the coefficients v_o and K by graphical means or by a regression analysis.

Example 10.5

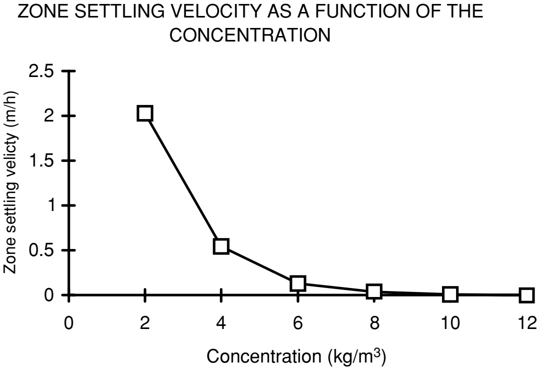
Determine the values of the coefficients v_o and K, based on the determination of the zone settling velocity v (according to the methodology of Example 10.4), for different values of the initial concentration C. The values obtained for v as a function of C were:

C (kg/m^3)	2.0	4.0	6.0	8.0	10.0	12.0
v (m/h)	2.03	0.55	0.13	0.04	0.01	0.00

Note: 1 kg/m^3 = 1000 g/m^3 = 1000 mg/l

Solution:

a) Plot the curve of v × C



b) Rearrange Equation 10.10 in a logarithmic form

Equation 10.10 is:

$$v = v_o.e^{-K.C}$$

Taking the natural logarithm of both sides of the equation:

$\ln v = \ln v_o - K.C$

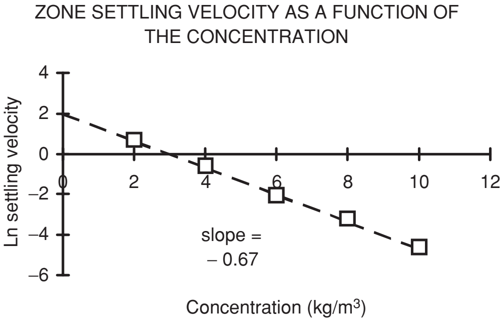
The intercept of the line with the Y-axis is: $\ln v_o$
The slope of the line is: $-K$

c) Plot the logarithmic form of Equation 10.10

The values of the logarithmic form of the equation are:

C (kg/m^3)	2.0	4.0	6.0	8.0	10.0	12.0
ln v	0.71	−0.60	−2.04	−3.21	−4.61	–

***Example 10.5* (*Continued*)**



From the graph:

The intercept of the line with the Y-axis is: $\ln v_o = 2.0 \rightarrow v_o = e^{2.0} = 7.4$
The slope of the line is: $-K = -0.67 \rightarrow K = 0.67$

The coefficients of Equation 10.10 are, therefore:

$v_o = 7.4$ m/h
$K = 0.67$ kg/m^3

These coefficients were, for didactic reasons, obtained graphically in the present example. However, it is preferable that these are obtained through regression analysis. The regression analysis can be linear (with the logarithmic transformation of Equation 10.10) or non-linear (with the original Equation 10.10).

10.5.4 Sludge volume index

The settleability of the sludge can be inferred through the settling curves, such as those presented in Section 10.5.3. However, frequently, in a wastewater treatment plant, only a simplified evaluation of the settleability is desired, aiming at using the data for the operational control of the plant. Under these conditions, the Sludge Volume Index (SVI) concept can be adopted.

The SVI is defined as the volume occupied by 1 g of sludge after settling for a period of 30 minutes. Hence, instead of determining the interface level at various time intervals, a single measurement at 30 minutes is made. The SVI is calculated through the following equation (see also Figure 10.19):

$$\boxed{SVI = \frac{H_{30} \times 10^6}{H_0.SS}} \qquad (10.12)$$

where:

SVI = Sludge Volume Index (mL/g)
H_{30} = height of the interface after 30 minutes (m)

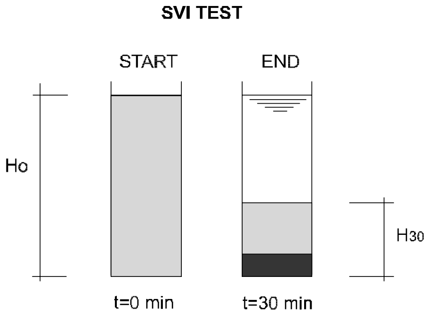


Figure 10.19. Schematics of the SVI test

H_0 = height of the interface at time 0 (height of the water level in the settling cylinder) (m)
SS = suspended solids concentration in the sample (mg/L)
10^6 = conversion from mg to g, and from mL to L

Some standardisations are done in the SVI test, resulting in the following most common variants of the test:

- *Test without stirring during the settling period* ***(SVI)***. The sample is left to settle without disturbances.
- *Test without stirring and with a dilution of the sample* ***(DSVI)***. The original sample is diluted with the final effluent of the works in ratios of 2 (e.g. $1/2$, $1/4$, $1/8$, etc.). The DSVI is calculated using the diluted sample that leads to an interface height after 30 minutes (H_{30}) of less than 20% of the initial height (and as close as possible to 20%, with a tolerance of approximately 4%). The DSVI is calculated from Equation 10.12 and is multiplied by the reciprocal of the dilution ratio (e.g. multiplied by 4 if the dilution ratio was $1/4$).
- *Test with stirring during the settling period* ***(SSVI)***. The stirring is mild and aims at reproducing the light stirring that occurs in a real scale settling tank. A thin vertical bar with a rotation of 1 or 2 rpm, situated between the centre and the periphery of the cylinder, causes the stirring in the cylinder.
- *Test with stirring and expression of the results in a standard concentration of 3.5 g/L (3500 mg/L)* ***(SSVI$_{3.5}$)***. The advantage is that the results are expressed in a standard concentration (the interpretation of the other SVI tests is subject to the influence of the initial SS concentration). The concentration of 3.5 g/L is selected, because it represents an usual value of the mixed liquor suspended solids (MLSS) concentration in aeration tanks of the activated sludge process. This test is undertaken for different initial concentrations (obtained through dilutions and concentrations of the sample), and the results are interpolated for a concentration of 3.5 g/L. This test is the most representative and less subject to distortions.

Table 10.4. Approximate interpretation of values of the Sludge Volume Index (for activated sludge)

	Range of values for the Sludge Volume Index (mL/g)			
Settleability	SVI	DSVI	SSVI	$SSVI_{3,5}$
Excellent	0–50	0–45	0–50	0–40
Good	50–100	45–95	50–80	40–80
Fair	100–200	95–165	80–140	80–100
Poor	200–300	165–215	140–200	100–120
Very poor	>300	>215	>200	>120

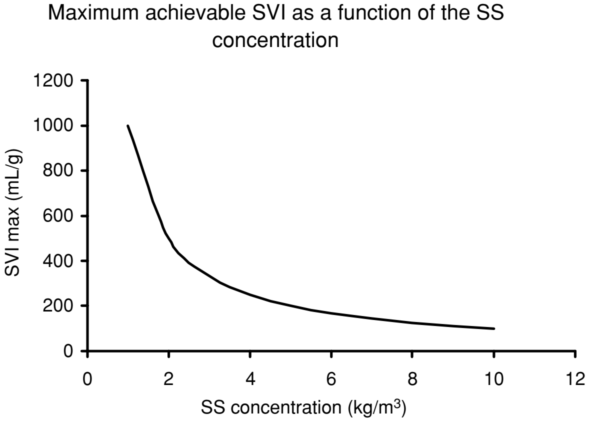


Figure 10.20. Maximum achievable SVI value (no sedimentation of the interface in the cylinder), as a function of the SS concentration.

The interpretation of the Sludge Volume Index is that, the larger the value, the lower the settleability of the sludge, that is, the sludge occupies a greater value in the secondary sedimentation tank. Besides this, the interpretation is also associated with the type of test. Typical approximate values are expressed in Table 10.4 (von Sperling, 1994; von Sperling and Fróes, 1999).

It should be emphasised that the traditional SVI test has a limitation because it is dependent on the initial solids concentration (denominator of Equation 10.12). For example, a sludge with a concentration of 1,000 mg/L that does not settle at all at the end of 30 minutes ($H_{30} = H_0$) will have a SVI of $10^6/1000 = 1.000$ mL/g. On the other hand, a sludge with a concentration of 10,000 mg/L that also does not settle after 30 minutes will have a SVI of $10^6/10{,}000 = 100$ mL/g. It is therefore clear the difficulty in the interpretation of the SVI results, because two sludges that do not settle at all have SVI values so different. Figure 10.20 shows the maximum SVI value (in which there is no settling in the cylinder) that can be obtained for sludges with different concentrations.

The values of DSVI and $SSVI_{3.5}$ are less susceptible to these influences, since they do not express the results in such varied concentrations. However, one should be always conscious of the fact that the SVI test and its variants only express the

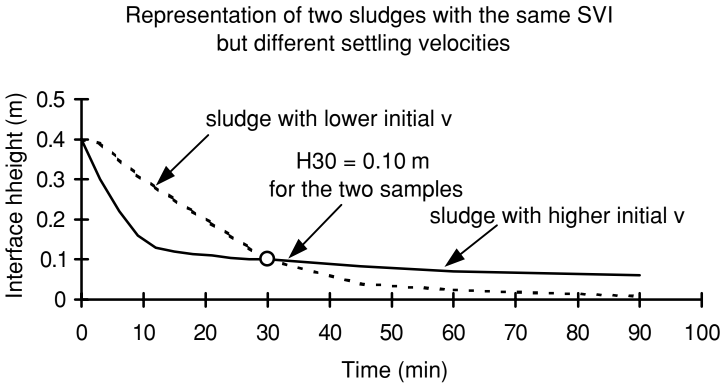


Figure 10.21. Representation of two sludge samples with different settling velocities, but with the same SVI (adapted from Wanner, 1994)

sedimentation after a defined period (30 minutes) and do not give a direct indication of the settling velocity. Two sludges with the same SVI could have different settling velocities, as shown in Figure 10.21.

Example 10.6

Calculate the SVI (without stirring and dilution) of the activated sludge sample from Example 10.4 in which the following values were given or obtained:

- $H_0 = 0.40$ m
- $H_{30} = 0.10$ m
- SS = 2,900 mg/L

Solution:

From Equation 10.12:

$$SVI = \frac{H_{30} \times 10^6}{H_0.SS} = \frac{0.10 \times 10^6}{0.40 \times 2920} = 86 \text{ mL/g}$$

Based on the interpretation of Table 10.4, the settleability of this sludge can be considered good.

10.5.5 Determination of the limiting solids flux

As seen in Section 10.5.2, the total solids flux transmitted to the bottom of the secondary sedimentation tank is composed of the following components:

- *gravity flux* (G_g), caused by the gravitational sedimentation of the sludge;
- *underflow flux* (G_u), caused by the movement of the sludge resulting from the removal of the return sludge from the bottom of the sedimentation tank.

The respective formulas are:

Total flux:

$$\boxed{G_t = G_g + G_u} \tag{10.13}$$

Gravity flux:

$$\boxed{G_g = C.v_o.e^{-K.C}} \tag{10.14}$$

Underflow flux:

$$\boxed{G_u = C.\frac{Q_u}{A}} \tag{10.15}$$

where:

C = suspended solids concentration in the sludge (kg/m^3)
v_o = coefficient, expressing the zone settling velocity at a concentration $C = 0$ (m/h)
K = sedimentation coefficient (m^3/kg)
Q_u = sludge underflow (m^3/h)
A = surface area of the sedimentation tanks (m^2).

The limiting flux corresponds to the minimum at the curve G_t versus C. The minimum can be obtained, for a given value of Q_u/A, through the calculation of the limiting concentration C_L, such that the first derivative of the total flux equation G_t (Equation 10.13) is equal to zero, and the second derivative is greater than zero, to configure a minimum. The respective equations are:

- Limiting solids flux

$$G_L = v_o.C.e^{-K.C} + \frac{Q_u}{A} \tag{10.16}$$

- First derivative

$$G_L' = v_o.e^{-K.C}.(-K.C + 1) + \frac{Q_u}{A} = 0 \tag{10.17}$$

- Second derivative

$$G_L'' = v_o.e^{-K.C}.(C.K^2 - 2.K) > 0 \tag{10.18}$$

However, the determination of the limiting flux based on Equation 10.16 cannot be done directly. Because Equation 10.17 is not explicit in terms of C, it needs to be solved numerically by iteration (e.g. Newton–Raphson method) and the final result substituted again into Equation 10.16. Even though this solution can be obtained without problems using computer programs, this section presents the simpler and more didactic approach of the graphic solution, which can be also implemented in computers, using simple spreadsheets.

For given values of the coefficients v_o and K and of the sludge underflow velocity (Q_u/A), curves of the gravity flux, underflow flux and total flux can be composed graphically. Example 10.7 illustrates the methodology to be employed.

Example 10.7

Based on the data from Examples 10.4 and 10.5, compose the solids flux curves and determine the values of : (a) limiting solids flux, (b) limiting solids concentration and (c) solids concentration at the bottom sludge. Determine if the sedimentation tank is overloaded or underloaded.

Data given in Examples 10.4 and 10.5:

- $v_o = 7.4$ m/h
- $K = 0.67$ m^3/kg
- MLSS: $C_o = 2{,}900$ g/m^3 $= 2.9$ kg/m^3

Additional data:

- total surface area of the secondary sedimentation tanks: $A = 500$ m^2
- influent wastewater flow to the works: $Q_i = 350$ m^3/h
- return sludge flow ($\approx$ underflow): $Q_u = 200$ m^3/h

Solution:

a) Calculate the fluxes for different values of solids concentration

The gravity flux, underflow flux and total flux are calculated below, for values of C varying from 0 to 20 kg/m^3.

C (kg/m^3)	v (m/h)	G_g (kg/m^2.h)	G_u (kg/m^2.h)	G_t (kg/m^2.h)
0.0	7.40	0.00	0.00	0.00
0.5	5.29	2.65	0.20	2.85
1.0	3.79	3.79	0.40	4.19
1.5	2.71	4.06	0.60	4.66
2.0	1.94	3.88	0.80	4.68
2.5	1.39	3.47	1.00	4.47
3.0	0.99	2.97	1.20	4.17
3.5	0.71	2.48	1.40	3.88
4.0	0.51	2.03	1.60	3.63
4.5	0.36	1.63	1.80	3.43
5.0	0.26	1.30	2.00	3.30
5.5	0.19	1.02	2.20	3.22
6.0	0.13	0.80	2.40	3.20
6.5	0.10	0.62	2.60	3.22
7.0	0.07	0.48	2.80	3.28
7.5	0.05	0.36	3.00	3.36
8.0	0.03	0.28	3.20	3.48
8.5	0.02	0.21	3.40	3.61

Example 10.7 (*Continued*)

C (kg/m^3)	v (m/h)	G_g ($kg/m^2.h$)	G_u ($kg/m^2.h$)	G_t ($kg/m^2.h$)
9.0	0.02	0.16	3.60	3.76
9.5	0.01	0.12	3.80	3.92
10.0	0.01	0.09	4.00	4.09
10.5	0.01	0.07	4.20	4.27
11.0	0.00	0.05	4.40	4.45
11.5	0.00	0.04	4.60	4.64
12.0	0.00	0.03	4.80	4.83
12.5	0.00	0.02	5.00	5.02
13.0	0.00	0.02	5.20	5.22
13.5	0.00	0.01	5.40	5.41
14.0	0.00	0.01	5.60	5.61
14.5	0.00	0.01	5.80	5.81
15.0	0.00	0.00	6.00	6.00
15.5	0.00	0.00	6.20	6.20
16.0	0.00	0.00	6.40	6.40
16.5	0.00	0.00	6.60	6.60
17.0	0.00	0.00	6.80	6.80
17.5	0.00	0.00	7.00	7.00
18.0	0.00	0.00	7.20	7.20
18.5	0.00	0.00	7.40	7.40
19.0	0.00	0.00	7.60	7.60
19.5	0.00	0.00	7.80	7.80
20.0	0.00	0.00	8.00	8.00

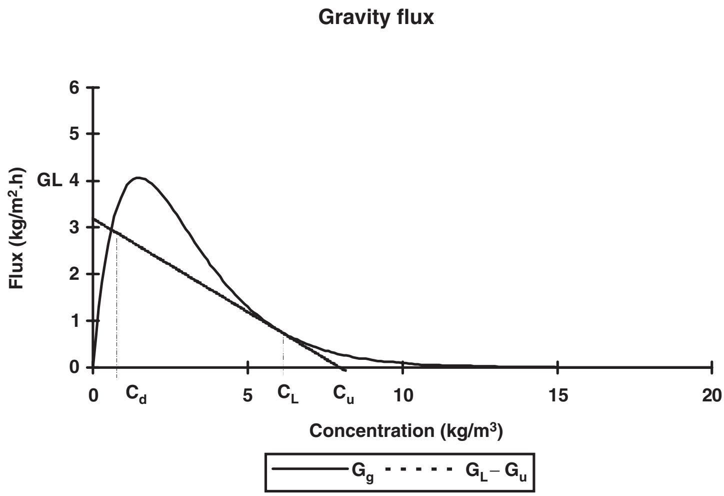
where:

- *C = suspended solids concentration, varying from 0 to 20 kg/m^3*
- *v = zone settling velocity (m/h)*
  Given by Equation 10.10: $v = v_o.e^{-K.C}$
  In example 10.4, the values of the coefficients v_o and K were determined (v_o = 7.4 m/h and K = 0.67 kg/m^3)
- *G_g = gravity solids flux ($kg/m^2.h$)*
  Given by Equation 10.14: $G_g = C.v_o.e^{-K.C}$
- *G_u = underflow flux ($kg/m^2.h$)*
  Given by Equation 10.15: $G_u = C.\ (Q_u/A)$
  The value of Q_u/A is calculated based on the data given in the problem: $Q_u/A = (200\ m^3/h)\ /\ (500\ m^2) = 0.4\ m/h$
- *G_t= total solids flux ($kg/m^2.h$)*
  Given by Equation 10.13: $G_t = G_g + G_u$

b) Plot the gravity flux (G_g)

As shown in Figure 10.16, one method for determining the limiting flux is through the graph of the gravity flux, presented below. The data necessary for the composition of the graph are taken from the previous table.

Example 10.7 (*Continued*)



The curve of the flux G_g is taken directly from the values in the previous table. The dashed line (characterised as $G_L - G_u$) has a slope of Q_u/A, and is located as tangent to the curve G_g. This line presents four important points:

- intercept with the Y-axis: *limiting flux* G_L (obtained on the Y-axis)
- intercept with the curve G_g: *concentration of the diluted layer* C_d (obtained on the X-axis)
- tangent to the curve G_g: *limiting concentration* C_L (obtained on the X-axis)
- intercept with the X-axis: *underflow sludge concentration* C_u (obtained on the X-axis)

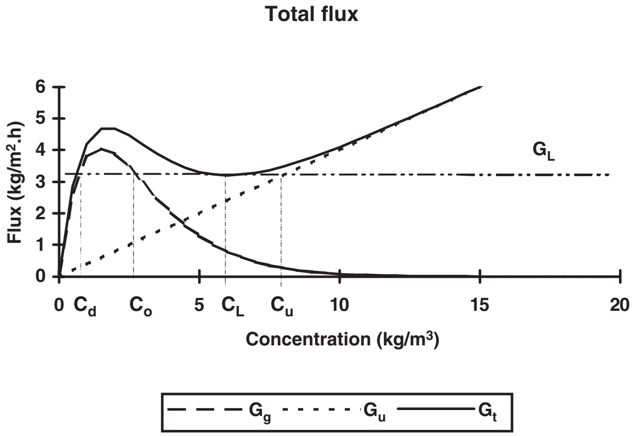
The values obtained graphically are:

- *limiting flux* G_L: 3.2 kg/m^2.h
- *concentration of the diluted layer* C_d: 0.6 kg/m^3
- *limiting concentration* C_L: 6.0 kg/m^3
- *underflow sludge concentration* C_u: 8.0 kg/m^3

c) Plot the total flux (G_t)

An alternative form of determining the limiting flux is through the total flux graph, discussed in Figure 10.16. The necessary values for the composition of the graph are taken directly from the previous table (G_g, G_u and G_t).

Example 10.7 (*Continued*)



The limiting flux corresponds to the minimum of the total flux curve G_g. The other parameters (C_d, C_L and C_u) can be obtained based on the intercepts of the tangent to this point of minimum (parallel with the X-axis) with the curve G_g, as seen in the graph. Naturally, the values obtained are the same as those already obtained in the previous graph presented in item b.

d) Interpretation of the flux curves

The mixed liquor enters the sedimentation tank with a concentration of $C_o = 2.9$ kg/m^3 (MLSS). While the sludge thickens and its concentration increases, the capacity to transmit solids to the bottom of the tank decreases (see the curve of G_t in the above graph). This occurs because in this range the increase of C brings about a reduction in the settling velocity v (see Equation 10.10). The capacity to transfer these solids to the bottom decreases until the concentration of 6 kg/m^3 (limiting concentration C_L) is reached. In these conditions there is the maximum value of the flux that can be transferred to the bottom, that is, the limiting flux ($G_L = 3.2$ kg/m^2.h) (see the point of minimum on the curve G_t). Subsequently, the sludge concentration increases until it reaches the concentration of the underflow sludge at the bottom ($C_u = 8.0$ kg/m^3). In this range, the transmission capacity of the solids to the bottom starts to increase again (see curve G_t above), due to the contribution of C in the gravity flux G_g (see Equation 10.14). In spite of this increase of the flux transmitted to the bottom, the sedimentation tank is previously limited by its limiting flux capacity. If a solids flux greater than the limiting flux is applied in the tank, the applied flux will not be able to be totally transmitted to the bottom of the tank, because it is greater than the limiting flux. In these conditions, only the limiting flux is transferred, and the flux

Example 10.7 (*Continued*)

in excess generates an expansion in the volume occupied by the sludge (rising of the sludge blanket level).

e) Evaluation of the loading conditions

To determine if the sedimentation tank is overloaded or underloaded, the applied and limiting fluxes must be compared. As seen, the limiting flux is $G_L = 3.2$ kg/m^2.h. The applied flux G_a is given by Equation 10.11:

$$G_a = \frac{Q_i + Q_r}{A}.C_o = \frac{(350 + 200)}{500} \times 2.9 = 3.2\,\text{kg/m}^2\text{.h}$$

It can be observed that the applied flux is equal to the limiting flux, or $G_a = G_L$. In these conditions, the sedimentation tank is in equilibrium, and the level of the sludge blanket remains constant.

Example 10.8

The influent flow to the plant analysed in Example 10.7 suffers an increase from 350 m^3/h to 450 m^3/h. Analyse the impact of this increase and propose a control measure.

Solution:

a) Evaluation of the loading conditions

Since the settleability of the sludge (coefficients v_o and K) and the underflow (Q_u) were not altered, the limiting flux remains the same ($G_L = 3.2$ kg/m^2.h). However, the applied flux increased due to the increase of Q_i. The new applied flux becomes:

$$G_a = \frac{Q_i + Q_r}{A}.C_o = \frac{(450 + 200)}{500} \times 2.9 = 3.8\,\text{kg/m}^2\text{.h}$$

In these conditions, $G_a > G_L$. The sedimentation tank cannot transmit to the bottom the total applied flux (3.8 kg/m^2.h), but only the limiting flux (3.2 kg/m^2.h). The exceeding flux ($3.8 - 3.2 = 0.6$ kg/m^2.h) will not be able to go to the bottom and will cause an increase in the volume of the sludge layer.

If no operational measures are taken, the persistence of this excess flux will cause a continuous expansion of the sludge blanket, until the solids start to leave with the final effluent. After that, a new equilibrium situation will be reached, with the excess flux leaving with the final effluent, and causing a greater deterioration in its quality. The solids load that leaves with the effluent

Example 10.8 (*Continued*)

will be 0.6 kg/m^2.h × 500 m^2 = 300 kg/h. This load represents a concentration in the final effluent of (300 kg/h) ÷ (450 m^3/h) = 0.67 kg/m^3 = 670 mg/L. This concentration is obviously unacceptable for the final effluent of a wastewater treatment plant.

b) Control measures

To avoid the expansion of the sludge blanket, the following operational measures can be taken:

- increase of the underflow rate
- reduction of the MLSS concentration

Increase of Q_u. If the underflow rate is increased, the limiting and applied fluxes will increase. However, there is a value of Q_u that permits both fluxes to be the same. Adopting the methodology of Example 10.7 for different values of Q_u, the value that leads to this new equilibrium condition can be obtained. In this case, the value found was Q_u = 315 m^3/h. The limiting and applied fluxes increase to 4.44 kg/m^2.h. Since both are equal, there will be no expansion of the sludge blanket. Hence, the increase of the underflow rate (that is, also of the return sludge flow) is an effective measure for the control of a secondary sedimentation tank subjected to an increase in the influent load.

Reduction of MLSS. One way of decreasing the applied flux is by reducing the influent concentration to the sedimentation tank, that is, MLSS. In this case, the applied flux is reduced and the limiting flux remains the same. If MLSS is reduced from 2.9 kg/m^3 to 2.45 kg/m^3, the new applied flux will be 3.2 kg/m^2.h, equal to the limiting flux (determined in Example 10.7). The reduction in the MLSS concentration is accomplished by the increase in the wastage flow of the excess (surplus) sludge. However, the capacity of the sludge treatment system to receive this increased load of the excess sludge needs to be verified.

Although these analyses seem laborious, it should be remembered that these calculations are easily implemented in a computer program or spreadsheet.

It should be also remembered that these analyses are applicable to steady-state conditions. However, the principles of the limiting solids flux theory are also applicable to dynamic models.

11

Aeration

11.1 INTRODUCTION

Aeration is a unit operation of fundamental importance in a large number of aerobic wastewater treatment processes. When a liquid is deficient in a gas (oxygen, in this case), there is a natural tendency of the gas to pass from the gas phase, where it is present in sufficient concentrations, to the liquid phase, where it is deficient. Oxygen is a gas that dissolves poorly in the liquid medium. For this reason, in various wastewater treatment systems it is necessary to accelerate the natural process, in such a way that the oxygen supply may occur at a higher rate, compatible with the biomass utilisation rate. Among the wastewater treatment processes that use artificial aeration are aerated lagoons, activated sludge and its variants, aerated biofilters and other more specific processes. In terms of sludge treatment, aerobic digesters also use artificial aeration.

There are two main forms of producing artificial aeration:

- introduce air or oxygen into the liquid (*diffused air aeration*)
- cause a large turbulence, exposing the liquid, in the form of droplets, into the air, and also permitting the entrance of atmospheric air into the liquid medium (*surface or mechanical aeration*)

Within these two types there are variants, which are described in Sections 11.7 and 11.8. Figure 11.1 presents schematically the principles of aeration by diffused air and mechanical aeration.

In the end of this chapter, another way of transferring oxygen to the liquid medium is described: gravity aeration (steps, weirs, cascades).

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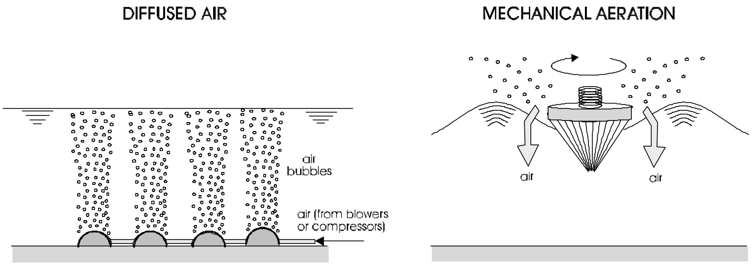


Figure 11.1. Schematic representation of diffused air and mechanical aeration systems.

11.2 FUNDAMENTALS OF GAS TRANSFER

11.2.1 Saturation concentration of a gas

When a liquid is exposed to a gas, there is a continuous exchange of molecules from the liquid phase to the gas phase and vice versa. As soon as the solubility concentration in the liquid phase is reached, both fluxes become equal in magnitude, in such a way that no overall change in the gas concentrations in both phases occurs (Figure 11.2). This dynamic equilibrium is associated with the **saturation concentration** of the gas in the liquid phase.

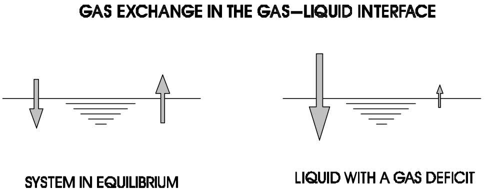


Figure 11.2. Gas exchanges in (a) a system in equilibrium and (b) a system with a deficit in the dissolved gas

In equilibrium conditions, the velocities of absorption (v_g) and release (v_l) of the gas are equal, that is:

$$v_g = v_l \tag{11.1}$$

The saturation concentrations in the two phases are proportional to these velocities:

$$k_g.C_g = k_l.C_s \tag{11.2}$$

where:

k_g and k_l = constants of proportionality
C_g = concentration of the gas in the gas phase (mg/L)
C_s = concentration of the gas in the liquid phase (mg/L)

Hence,

$$C_s = \frac{k_g}{k_l}.C_g \tag{11.3}$$

Making $k_g/k_l = k_D$:

$$\boxed{C_s = k_D.C_g} \tag{11.4}$$

Therefore, the saturation concentration is directly proportional to the concentration in the gas phase (Henry's Law). The coefficient k_D is called the *distribution coefficient*, and depends on the nature of the gas and the liquid, and the temperature.

For the diffusion of oxygen in water, k_D values are (Pöpel, 1979):

Table 11.1. Values of the distribution coefficient k_D for oxygen

Temperature (°C)	k_D
0	0.0493
10	0.0398
20	0.0337
30	0.0296

Therefore, it can be seen that, the greater the temperature, the lower the solubility of the gas in the liquid medium. The larger agitation of the molecules in the water contributes to the transfer of the gas to the gas phase.

The concentration C_g can be obtained from the universal gas law:

$$pV = nRT \tag{11.5}$$

where:

p = partial pressure of the gas in the gas phase (Pa)
V = volume occupied in the gas phase (m^3)
n = number of moles of gas in the volume V (mol/m^3)
R = universal constant (8.3143 J/K.mol)
T = temperature (K)

Developing Equation 11.5 and introducing it into Equation 11.4 and, at the same tine correcting it for the water vapour pressure, the equation that establishes the saturation concentration of a gas in water as a function of the temperature and pressure is obtained:

$$C_s = k_D.d_v.(P_a - P_v).\frac{MW}{R.T} \tag{11.6}$$

where:

d_v = volumetric distribution of oxygen in atmospheric air (0.21 or 21% of the air, in volume, is represented by oxygen)

MW = molecular weight of oxygen (32 g/mol)
P_a = atmospheric pressure (101,325 Pa in standard temperature and pressure)
P_v = water vapour pressure (see Table 11.2)

Table 11.2. Water vapour pressure values (P_v)

Temperature (°C)	P_v (Pa)
0	611
10	1,230
20	2,330
30	4,240

Example 11.1

Calculate the saturation concentration of oxygen in pure water, in the following conditions:

- temperature = 20 °C
- atmospheric pressure at sea level

Solution:

Based on the values presented above:
$k_D = 0.337$
$d_v = 0.21$
$P_a = 101{,}325$ Pa
$P_v = 2{,}330$ Pa
MW = 32 g/mol
R = 8.3143 J/K.mol
T = 293 K(= 20 °C)
Using Equation 11.6:

$$C_s = k_D.d_v.(P_a - P_v).\frac{MW}{R.T}$$

$$= 0.0337 \times 0.21 \times (101{,}325 - 2{,}330).\frac{32}{8.3143 \times 293} = 9.2\ \text{mg/L}$$

In order to avoid this sequence of laborious calculations, there are some empirical formulas available in the literature (the majority based on regression analysis) that give directly the value of C_s(mg/L) as a function, for example, of the temperature T (°C). A formula frequently used is:

$$\boxed{C_s = 14.652 - 4.1022 \times 10^{-1}.T + 7.9910 \times 10^{-3}.T^2 - 7.7774 \times 10^{-5}.T^3} \quad (11.7)$$

Altitude exerts an influence on the solubility of a gas, because it is inversely proportional to the atmospheric pressure. The greater the altitude, the lower the atmospheric pressure and the lower the pressure for the gas to be dissolved in the water. This influence can be computed by the following relation (Qasim, 1985):

$$\boxed{f_H = \frac{C'_s}{C_s} = \left(1 - \frac{H}{9450}\right)} \tag{11.8}$$

where:

f_H = correction factor for the DO saturation concentration by the altitude (−)
C'_s = saturation concentration at the altitude H (mg/L)
H = altitude (m)

The salinity also affects the solubility of the oxygen. The influence of dissolved salts can be computed by the following empirical formula (Pöpel, 1979):

$$\boxed{\gamma = 1 - 9 \times 10^{-6}.C_{sal}} \tag{11.9}$$

where:

γ = solubility reduction factor ($\gamma = 1$ for pure water)
C_{sal} = concentration of dissolved salts (mg Cl^-/l)

Table 3.7 (Chapter 3) presents the saturation concentration of oxygen in clean water for different temperatures and altitudes.

11.2.2 Gas transfer mechanisms

11.2.2.1 Molecular diffusion

There are two basic mechanisms for the transfer of oxygen from the gas phase to the liquid phase:

- *molecular diffusion*
- *turbulent diffusion*

Molecular diffusion can be understood as the tendency of any substance to spread itself uniformly in the space available.

For a water body of unlimited depth, exposed to the gas phase through a surface A, the mass transfer rate dM/dt due to the diffusion of the gas molecules in the liquid phase is defined by Fick's law (Pöpel, 1979):

$$\frac{dM}{dt} = -D.A.\frac{\partial C}{\partial x} \tag{11.10}$$

where:

D = coefficient of molecular diffusion (m^2/s)
A = surface area (m^2)

x = distance from the interface (m)
$\partial C/\partial x$ = concentration gradient ($g/m^3.m$)

It is important to note that, for a certain gas, only the concentration gradient determines the diffusion rate per unit area. The negative sign indicates that the direction of diffusion is opposite to the positive concentration gradient.

For oxygen, the values of the diffusion coefficient are presented in Table 11.3.

Table 11.3. Values of the diffusion coefficient D

Temperature (°C)	D (10^{-9} m^2/s)
10	1.39
20	1.80
30	2.42

Two theories frequently used to explain the gas transfer mechanism are (Pöpel, 1979):

- *Two-film theory.* In the gas–liquid interface there are two films, a gas film and a liquid film. The gas is absorbed and transported by molecular diffusion and mixing (convection) by the gas film and subsequently by the liquid film. The films are considered as stagnant and with a fixed thickness. The two-film theory is simpler but provides a good answer in most cases (Metcalf & Eddy, 1991).
- *Penetration theory.* The penetration theory does not assume stagnant films, but fluid elements that are momentarily exposed to the gas phase in the liquid interface. During this exposure time the gas diffuses in the fluid elements *penetrating* the liquid. Differently from the two-film theory, the penetration process is described by an unsteady diffusion. The exposure time is considered very short (< 0.1 s) for steady diffusion conditions to prevail. The penetration theory is more soundly theoretically based.

According with the penetration theory, the following formulas for the gas transfer can be obtained:

- Absorption rate of the gas:

$$\frac{dM}{dt} = A.(C_s - C_o).\sqrt{\frac{D}{\pi.t}} \qquad (11.11)$$

- Penetration depth of the gas:

$$x_p = \sqrt{\pi.D.t} \qquad (11.12)$$

where:
M = mass of the gas absorbed through the area A during the time t (g)
A = interfacial exposure area (m^2)
t = exposure time (s)

C_o = initial concentration of the gas in the bulk of the liquid mass (g/m^3)
x_p = penetration depth of the gas in the liquid mass (m)

Example 11.2

A tank under quiescent conditions, completely deprived of oxygen, with a temperature of 20 °C, at sea level, is exposed to air.

- What is the absorption rate of oxygen?
- What is the penetration depth of oxygen?

Solution:

a) Oxygen absorption rate

$C_o = 0.0\ g/m^3$
$C_s = 9.2\ g/m^3$ (see Table 3.7)
$D = 1.8 \times 10^{-9}\ m^2/s$ (see Table 11.3)

$$\frac{dM}{dt.A} = (C_s - C_0).\sqrt{\frac{D}{\pi.t}} = (9.2 - 0.0).\sqrt{\frac{1.8 \times 10^{-9}}{\pi.t}}$$

After 1 second (t = 1 s) $= 220 \times 10^{-6}\ g/m^2.s$
1 minute (t = 60 s) $= 28 \times 10^{-6}\ g/m^2.s$
1 hour (t = 3,600 s) $= 3.7 \times 10^{-6}\ g/m^2.s$
1 day (t = 86,400 s) $= 0.75 \times 10^{-6}\ g/m^2.s$

b) Penetration depth of the oxygen

$$x_p = \sqrt{\pi.D.t} = \sqrt{\pi.1.8 \times 10^{-9}.t}$$

After 1 second (t = 1 s) $= 0.075 \times 10^{-3}$ m
1 minute (t = 60 s) $= 0.582 \times 10^{-3}$ m
1 hour (t = 3,600 s) $= 4.51 \times 10^{-3}$ m
1 day (t = 86,400 s) $= 22.1 \times 10^{-3}$ m

The objective of the present example is to emphasise the fact that the transfer of oxygen by molecule diffusion is extremely slow. In wastewater treatment, the high oxygen demand cannot by supplied simply by molecular diffusion.

11.2.2.2 Turbulent diffusion

In sewage treatment with artificial aeration, the main gas transfer mechanism occurs through the *creation* and *renewal* of the interfaces.

The turbulent flux generated by artificial aeration consists of a complex secondary movement that surpasses the primary movement of the liquid mass. The turbulence is characterised by oscillations and eddies that transport fluid particles from one layer to another, with variable velocities. The turbulent movement, which is erratic in direction, magnitude and time, can be defined only probabilistically (O'Connor and Dobbins, 1958).

As mentioned, gas transfer by turbulent diffusion is much higher than by molecular diffusion. The basic structure of the gas transfer formulation can be maintained, with adaptations only in the sense of simplifying its presentation. The transfer coefficient incorporates other constants, as shown in Sections 11.3 and 11.4.

11.3 KINETICS OF AERATION

The disadvantage of the mentioned formulas is that the diffusion coefficient D, the exposure time to the interfacial area t_c and the interfacial area A must be known, in order to allow the estimation of the gas transfer rate. Consequently, it is necessary to adopt a more practical approach, as discussed below.

Under steady-state conditions, the diffusion coefficient D and the exposure time t_c can be assumed as constants, resulting in a constant gas transfer coefficient (K_L). Besides this, the surface area A and the specific area a (= area/volume = A/V) may also be assumed as constants. Under these conditions, a constant K_La can be defined:

$$K_L\,a = 2.\sqrt{\frac{D}{\pi.t_c}}.\frac{A}{V} \tag{11.13}$$

where:

K_La = overall oxygen mass transfer coefficient (s^{-1})

Thus, the mass of oxygen transferred per unit time and volume can be expressed in the following simplified form, through a simple rearrangement of Equation 11.13:

$$\frac{m}{V} = K_La.(C_s - C) \tag{11.14}$$

or:

$$\boxed{\frac{dC}{dt} = K_La.(C_s - C)} \tag{11.15}$$

where:

dC/dt = rate of change of the oxygen concentration ($g/m^3.s$)

C = concentration at any time t (g/m^3)

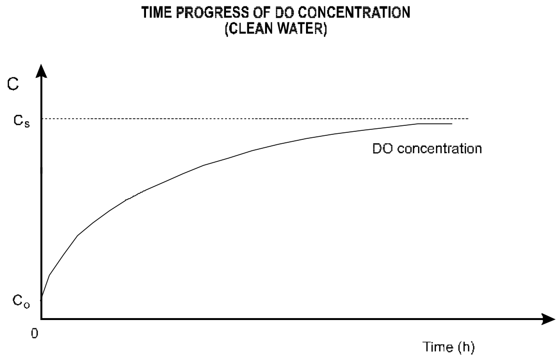


Figure 11.3. Temporal progression of the DO concentration during aeration (*without* oxygen consumption)

Through Equation 11.15, it can be seen that *the lower the oxygen concentration (C), or the higher the oxygen deficit ($C_s - C$), the greater is the oxygen transfer rate (dC/dt).*

Integrating Equation 11.15 between the limits of $t = 0$ to $t = t$ and $C = 0$ to $C = C$ leads to (ASCE, 1990):

$$\boxed{\frac{C_s - C}{C_s - C_o} = e^{-K_L a.(t - t_o)}} \tag{11.16}$$

If there is *no oxygen consumption* in the liquid medium under aeration (example: clean water), the concentration C increases according to a decreasing exponential rate (observe the negative sign on the exponent of e). The concentration tends asymptotically to the steady-state value, that is, the saturation concentration C_s. The formula of this trajectory is obtained through the rearrangement of Equation 11.16, being expressed in Equation 11.17 and represented in Figure 11.3.

$$\boxed{C = C_s - (C_s - C_o).e^{-K_L a.(t - t_o)}} \tag{11.17}$$

In case there is oxygen consumption in the liquid medium, which occurs in aeration tanks and aerated lagoons, the highest value that can be reached by the oxygen concentration is lower than the saturation value. Designating the oxygen consumption rate by **r** ($g/m^3.s$) and the maximum value to be reached by C as C_∞, the equation of the trajectory of DO in a reactor *with oxygen consumption* is (see Figure 11.4):

$$\boxed{C = C_\infty - (C_\infty - C_o).e^{-K_L a.(t - t_o)}} \tag{11.18}$$

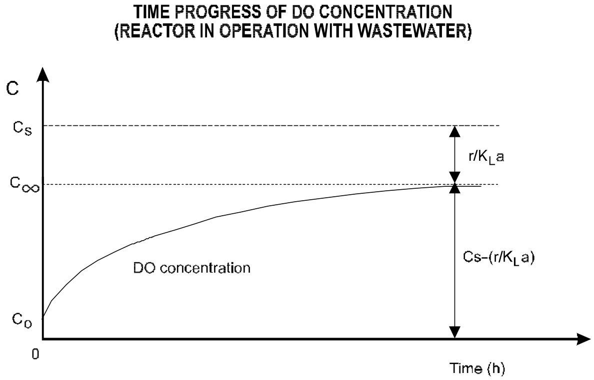


Figure 11.4. Temporal progression of the DO concentration during aeration (*with* oxygen consumption)

In the steady state, in a reactor under operation, the oxygen consumption rate (or oxygen utilisation rate) by the bacteria is equal to the oxygen production rate by the aeration system. Hence:

$$\text{O}_2 \text{ production rate} = \text{O}_2 \text{ consumption rate}$$

$$K_L a.(C_s - C) = r \qquad (11.19)$$

or:

$$\boxed{C = C_s - \frac{r}{K_L a}} \qquad (11.20)$$

The value of C obtained in Equation 11.20 corresponds to C_∞, presented in Figure 11.4.

The experimental determination of the coefficient $K_L a$ is discussed in Section 11.9.

11.4 FACTORS OF INFLUENCE IN OXYGEN TRANSFER

The oxygen transfer rate of the aeration equipment to be installed in a wastewater treatment plant is frequently determined in different conditions under which it will operate (*operating conditions*). Therefore, it is important to be able to quantify the factors that influence the oxygen transfer rate, to allow the estimation of the transfer rate under operating conditions, based on results obtained in tests undertaken under standardised conditions.

The factors of major influence on the oxygen transfer rate are:

- temperature
- atmospheric pressure (altitude)
- dissolved oxygen concentration

- characteristics of the wastewater
- characteristics of the aerator and the geometry of the reactor

a) Temperature

The influence of temperature occurs according to two apparently opposite directions:

- *Influence on the saturation concentration* C_s. The increase of the temperature causes a **reduction** in the saturation concentration C_s, which implies a **reduction** in the transfer rate dC/dt.
- *Influence on the mass transfer coefficient* K_La. The increase in the temperature causes an **increase** in the coefficient K_La, which implies an **increase** in the transfer rate dC/dt.

The influence on the saturation concentration was discussed in Section 11.2.1. The influence on K_La can be expressed by:

$$K_La_{(T)} = K_La_{(20\,^\circ C)}.\theta^{(T-20)} \tag{11.21}$$

where:

$K_La_{(T)}$ = coefficient K_La at any temperature T (s^{-1})
$K_La_{(20)}$ = coefficient K_La at a temperature of 20 °C (s^{-1})
θ = temperature coefficient. Usually adopted as 1.024.

b) Atmospheric pressure (altitude)

The influence of the altitude is manifested in the oxygen saturation concentration (the greater the altitude, the lower the atmospheric pressure and, therefore, the lower the saturation concentration). The correction factor for the altitude was discussed in Section 11.2.1.

c) Dissolved oxygen concentration

Under steady-state conditions, the greater the dissolved oxygen concentration (C) maintained in the reactor, the lower the value of $C_s - C$, that is, the lower is the oxygen transfer rate (see Equation 11.15). For example, in activated sludge systems, the DO concentration maintained in the reactor is usually in the range of 1.0 to 2.0 mg/L.

The correction of the influence of the DO concentration is obtained by:

$$\text{Correction factor for the DO concentration} = \frac{C_{sw} - C_L}{C_s(20\,^\circ C)} \tag{11.22}$$

where:

C_{sw} = saturation concentration of DO in the liquid in the reactor (mg/L) (see item d below)
C_L = DO concentration maintained in the liquid in the reactor (mg/L)
$C_s(20\,^\circ C)$ = saturation concentration of DO in clean water at 20 °C (mg/L)

d) Wastewater and reactor characteristics

The specific characteristics of the wastewater being treated and the configuration of the reactor, which are different from the test conditions in which the oxygen transfer is measured, also exert an influence on the actual transfer rate in the field, under operating conditions. This influence occurs in two ways:

- influence on the oxygen saturation concentration in the liquid in the reactor (C_{sw})
- influence on the oxygen transfer coefficient (K_La)

Influence on C_{sw}. The presence of salts, particulate matter and detergents affect the saturation concentration of the liquid in the reactor. This influence can be quantified through the following correction factor:

$$\beta = \frac{C_{sw}(\text{wastewater})}{C_s(\text{clean water})} \tag{11.23}$$

The values of β vary from 0.70 to 0.98, but the value of 0.95 is frequently adopted (Metcalf & Eddy, 1991).

Influence on K_La. The oxygen transfer coefficient is influenced by the characteristics of the wastewater as well as the geometry of the reactor and mixing level. The correction factor is:

$$\alpha = \frac{K_La\ (\text{wastewater})}{K_La\ (\text{clean water})} \tag{11.24}$$

Typical values of α vary from 0.6 to 1.2 for mechanical aeration and from 0.4 to 0.8 for diffused air aeration (Metcalf & Eddy, 1991).

11.5 OXYGEN TRANSFER RATE IN THE FIELD AND UNDER STANDARD CONDITIONS

The oxygen transfer rate will vary from place to place, for the same equipment, due to the simultaneous interaction of the various factors covered in Section 11.4. Therefore, it is important that the transfer rate may be expressed under standard conditions, in order to allow a uniform presentation of the values. Therefore, there are the following two ways of expressing the oxygen transfer rate (oxygenation capacity):

Standard conditions	*Operating conditions (field)*
• clean water	• wastewater
• liquid temperature = 20 °C	• real temperature of the liquid
• altitude = 0 m (sea level)	• real altitude of the plant
• aeration system installed in a test tank	• aeration system installed in the actual reactor

The conversion of one form to the other is done with the correction factors presented in Section 11.4. Incorporating all these factors, the general conversion equation is obtained:

$$\boxed{OTR_{standard} = \frac{OTR_{field}}{\dfrac{\beta.f_H.C_s - C_L}{C_s(20\,^\circ C)}.\alpha.\theta^{T-20}}} \tag{11.25}$$

where:

$OTR_{standard}$ = Standard Oxygen Transfer Rate – **SOTR** (kgO_2/h)

OTR_{field} = Oxygen Transfer Rate in the field, under operating conditions (kgO_2/h)

C_s = oxygen saturation concentration in clean water, at the operating temperature in the field (g/m^3)

C_L = average concentration of oxygen maintained in the reactor (g/m^3)

$C_s(20\,^\circ C)$ = saturation concentration of oxygen in clean water, under standard conditions (g/m^3)

f_H = correction factor C_s for the altitude (= 1− altitude/9450) (see Equation 11.8)

β = see comments for Equation 11.23

α = see comments for Equation 11.24

θ = see comments for Equation 11.21

T = liquid temperature (°C)

The relation between the standard oxygen transfer rate (SOTR or $OTR_{standard}$) and the oxygen transfer coefficient (K_La) can be obtained through rearrangement of Equation 11.15, in which C = 0 (standard conditions), leading to:

$$\boxed{OTR_{standard} = \frac{K_La.C_{s(20\,^\circ C)}.V}{1000}} \tag{11.26}$$

Due to the various influencing factors, the OTR_{field} is lower than the $OTR_{standard}$. Thus, in the designs, usually OTR_{field} is estimated as a function of the oxygen requirements and subsequently the $OTR_{standard}$ is calculated using Equation 11.25.

Example 11.3

In a wastewater treatment plant the supply of 100 kgO_2/h is necessary under operating conditions, using a mechanical aeration system. Determine the Standard Oxygen Transfer Rate knowing that:

- Liquid temperature: T = 23 °C
- Altitude = 800 m
- DO concentration to be maintained in the liquid: $C_L = 1.5$ mg/L

***Example 11.3* (*Continued*)**

Solution:

Adopt the following values for the parameters of Equation 11.25:

C_s (20 °C) = 9.2 mg/L (see Table 3.7, Chapter 3, column 0 m altitude, for T = 20 °C)
C_s = 8.7 mg/L (see Table 3.7, Chapter 3, column 0 m altitude, for T = 23 °C)
$\alpha = 0.90$ (see comments for Equation 11.24)
$\beta = 0.95$ (see comments for Equation 11.23)
$\theta = 1.024$ (see comments for Equation 11.21)
According to Equation 11.8 the value of f_H is:

$$f_H = 1 - \frac{\text{altitude}}{9450} = 1 - \frac{800}{9450} = 0.92$$

According to Equation 11.25 the value of OTR standard is:

$$\begin{aligned} OTR_{standard} &= \frac{OTR_{field}}{\dfrac{\beta . f_H . C_s - C_L}{C_s(20\,°C)} . \alpha . \theta^{T-20}} \\ &= \frac{100}{\dfrac{0.95 \times 0.92 \times .8.7 - 1.5}{9.2} . 0.9 \times 1.024^{23-20}} = \frac{100}{0.62} \\ &= 161 kgO_2/h \end{aligned}$$

The final results are:

OTR_{field} = 100 kgO_2/h (given in the problem)
$OTR_{standard}$ = 161 kgO_2/h
Ratio $OTR_{field}/OTR_{standard} = 100/161 = 0.62 = 62\%$

Therefore, it can be seen that in the field the aeration system is capable of supplying only 62% of the capacity under standard conditions. For this reason, to obtain the value of 100 kgO_2/h in the field, a system that supplies 161 kgO_2/h under standard conditions must be specified.

11.6 OTHER AERATION COEFFICIENTS

11.6.1 Oxygenation efficiency

The *oxygenation efficiency* (OE) represents the oxygen transfer rate (kgO_2/h) per unit power consumed (kW), and is expressed in the units of kgO_2/kWh.

$$\boxed{OE = \frac{OTR_{standard}}{P}} \tag{11.27}$$

where:

OE = oxygenation efficiency (kgO_2/kWh)
P = power consumed (kW)

Tables 11.5 and 11.6 (Sections 11.7 and 11.8) present typical values of the oxygenation efficiency (standard conditions) for the most commonly used aeration systems.

The power consumed is related to the voltage and amperage by (Boon, 1980):

$$P = \frac{\sqrt{3}.\text{volt}.\text{i}.\cos\phi}{1000} \tag{11.28}$$

where:

volt = voltage (V)
i = current intensity (A)
$\cos\phi$ = power factor (−)

In the case of mechanical aeration, it must be made clear if the power consumed does or does not include the efficiencies of the motor and the reducer.

In the case of diffused air aeration, the power required by the blowers can be expressed in terms of airflow and the pressure to be overcome by (Pöpel, 1979):

$$P = \frac{Q_g.\rho.g.(d_i + \Delta H)}{\eta} \tag{11.29}$$

where:

P = required power (W)
ρ = density of the liquid (1000 kg/m^3)
g = acceleration due to gravity (9.81 m/s^2)
d_i = immersion depth of the diffusers (m)
ΔH = head loss in the air distribution system (m)
η = efficiency of the motor and blower (−)

11.6.2 Oxygen transfer efficiency

In diffused air aeration systems, the ratio of oxygen utilisation (ROU) is expressed as the quantity of oxygen absorbed per m^3 of air applied (Pöpel, 1979):

$$\boxed{ROU = \frac{OTR_{\text{standard}}}{Q_g}} \tag{11.30}$$

where:

ROU = ratio of oxygen utilisation (kgO_2 absorbed/m^3 air applied)
Q_g = air flow (m^3/h)

The *standard oxygen transfer efficiency* (**SOTE**) represents the oxygen absorption efficiency in percentage terms. Since dry air contains 20.95% of oxygen on a volumetric basis, the molecular weight of oxygen is 32 g/mol and a gas occupies 0.0224 m^3/mol, then the concentration of oxygen in the gas is $= 0.2095 \times 32/0.0224 = 299\ gO_2/m^3$. The SOTE can therefore be calculated through (Pöpel, 1979):

$$\boxed{\text{SOTE} = 100.\frac{\text{ROU}}{C_g} = 100.\frac{\text{ROU}(g/m^3)}{299(g/m^3)} = 0.334.\text{ROU}\ (\%)} \tag{11.31}$$

where:

SOTE = standard oxygen transfer efficiency (%)

In order to take into consideration the depth of the diffuser d_i, the ratio of oxygen utilisation can be related to d_i, leading to the parameter *ratio of oxygen utilisation per unit immersion* (ROU/d_i), expressed as $gO_2/m^3.m$.

Table 11.5 (Section 11.8) presents typical SOTE values (standard conditions) for the more commonly used diffused air aeration systems.

11.6.3 Power level

The basic functions of an aeration system in most of the aerated wastewater treatment systems are:

- oxygenation of the wastewater under treatment
- liquid mixing, in order to maintain the biomass in suspension

To achieve the second objective, it is necessary to introduce a power per unit volume sufficient to avoid the settlement of the solids. This relation is represented through the concept of the *power level* (PL or ϕ), expressed as:

$$\boxed{PL = \frac{P}{V}} \tag{11.32}$$

where:

PL = power level (W/m^3)
P = power input (W)
V = reactor volume (m^3)

The greater the power level, the greater the quantity of suspended solids that can remain dispersed in the liquid medium (see Table 11.4). The values presented in the table are only estimates, since the mixing intensity also depends on the number and distribution of aerators (in the case of mechanical aeration) and on the size and geometry of the tank.

Owing to the higher suspended solids concentrations in activated sludge reactors, the power level to be adopted should usually be higher than 10 W/m^3.

Example 11.4 illustrates the calculation of the various aeration coefficients for a diffused air system. The calculation for a mechanical aeration system is

Table 11.4. Suspended solids concentrations that can be maintained dispersed in the liquid as a function of the power level

Power level (W/m^3)	SS (mg/L)
0.75	50
1.75	175
2.75	300

Source: Eckenfelder (1980)

simpler because the main coefficients are only OE and PL. Chapter 34 (activated sludge) shows the design sequence for the aeration system, starting from the oxygen requirements.

Example 11.4

Determine the main parameters of the following diffused air aeration system (medium size bubbles)

- Net reactor volume : $V = 500\ m^3$
- Air flow: $Q_g = 0.6\ m^3/s$
- Immersion depth of the diffusers: $d_i = 4.0$ m (tank height)
- Head loss in the air distribution system: $\Delta H = 0.4$ m
- Standard oxygen transfer rate: $OTR_{standard} = 60\ kgO_2/h$
- Efficiency of the motor and blower: $\eta = 0.60$

Solution:

a) Ratio of oxygen utilisation

$$Q_g = 0.6\ m^3/s \times 3{,}600\ s/h = 2,\ 160\ m^3/h$$

From Equation 11.30:

$$ROU = \frac{OTR_{standard}}{Q_g} = \frac{60{,}000\ gO_2/h}{2{,}160\ m^3/h} = 27.8\ gO_2/m^3air$$

b) Ratio of oxygen utilisation per unit immersion

$$\frac{ROU}{d_i} = \frac{27.8\ gO_2/m^3air}{4.0\ m} = 7.0\ gO_2/m^3.m$$

c) Standard oxygen transfer efficiency

From Equation 11.31:

$$SOTE = 0.334.ROU = 0.334 \times 27.8 = 9.3\%$$

***Example 11.4* (*Continued*)**

d) Required power

From Equation 11.29:

$$P = \frac{Q_g.\rho.g.(d_i + \Delta H)}{\eta} = \frac{0.6 \times 1000 \times 9.81 \times (4.0 + 0.4)}{0.60}$$
$$= 43{,}164\ W = 43.2\ kW$$

e) Oxygenation efficiency

From Equation 11.27:

$$OE = \frac{OTR_{standard}}{P} = \frac{60 kgO_2/h}{43.2\ kW} = 1.39\ kgO_2/kWh$$

f) Power level

From Equation 11.32:

$$PL = \frac{P}{V} = \frac{43{,}164\ W}{500\ m^3} = 86\ W/m^3$$

11.7 MECHANICAL AERATION SYSTEMS

The main mechanisms of oxygen transfer by mechanical surface aerators are (Malina, 1992):

- Atmospheric oxygen transfer to the droplets and the fine films of liquid sprayed in the air
- Oxygen transfer at the air-liquid interface, where the falling drops enter into contact with the liquid in the reactor
- Oxygen transfer by air bubbles transported from the surface to the bulk of the liquid medium

The more commonly used mechanical aerators can be grouped according to:

Classification as a function of the rotation shaft:

- *vertical shaft aerators*
    - low speed, radial flow
    - high speed, axial flow
- *horizontal shaft aerators*
    - low speed

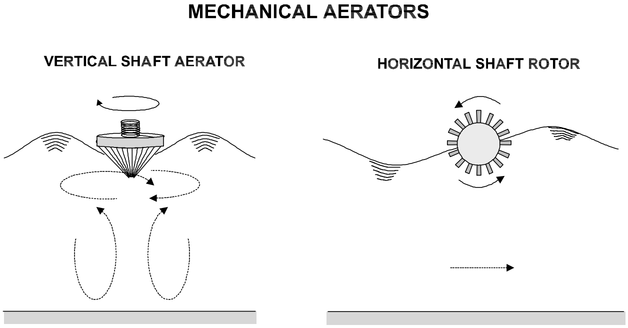


Figure 11.5. Schematic representation of vertical and horizontal shaft aerators

Classification as a function of the supporting:

- *fixed aerators*
- *floating aerators*

Figure 11.5 shows schematically mechanical aerators with vertical and horizontal shafts. Table 11.5 presents a comparison between the three main types: (a) vertical shaft – low speed, (b) vertical shaft – high speed and (c) horizontal shaft.

The power of mechanical aerators usually varies between 5 HP and 100 HP, although, in special conditions, lower and higher values can be found.

In mechanical aerators, the submergence of the impellers in relation to the water level is a very important aspect in terms of oxygen transfer and energy consumption. The following situations can occur:

- *Adequate submergence*. The performance is optimal. There is good turbulence and absorption of air with relation to the oxygen consumption.
- *Submergence above the optimal*. The unit tends to function more as a mixer than as an aerator. The energy consumption increases without being accompanied by a substantial increase in the oxygen transfer rate.
- *Submergence below the optimal*. Only a surface spray is formed in the vicinity of the aerator, without creating an effective turbulence. The energy consumption and the oxygen transfer rate decrease.

The installation of the aerator must follow the manufacturer's instructions. Besides this, local tests should be carried out in order to obtain the optimal submergence in the reactor in question.

In many activated sludge plants, the oxygen transfer rate can be varied in such a way as to adjust itself to the variations in the oxygen utilisation rate. The variation can be manual or automated, by means of timers or sensors for dissolved oxygen in the reactor. Listed below are some of the most common forms of varying the

Table 11.5. Characteristics of the main mechanical aeration systems

Type of aerator	Characteristics	Application	Components	Advantages	Disadvantages	Standard oxygenation efficiency (kgO_2/kWh)
Low speed, radial flow	Similar to a high flow and low head pump. The flow of the liquid in the tank is radial in relation to the axis of the motor. Most of the oxygen absorption results from an induced hydraulic jump. Rotation speed 20–60 rpm.	Activated sludge and variants. Aerobic digesters. Large aeration units with depths up to 5 m.	Motor, reducer, impeller. Fixation units (bridges or platforms) for the fixed aerators (more common).	High oxygen transfer. Good mixing capacity. Flexibility in the design of the tank. High pumping capacity. Easy access for maintenance.	High initial costs. Careful maintenance of the reducers is necessary.	1.4 – 2.0
High speed, axial flow	Similar to a high flow and low head pump. The flow of the liquid pumped is upwards and follows the axis of the motor, passing through the volute before reaching a diffuser, where it is dispersed perpendicularly to the axis of the motor in the form of a spray. Most of the oxygen absorption occurs due to spray and turbulence. Rotation speed: 900 – 1400 rpm.	Activated sludge and variants. Aerobic digesters. Aerated lagoons.	Motor, impeller, float (a reducer is not needed).	Lower initial costs. Easily adjustable to variations in the water level. Flexible operation.	Difficult access for maintenance. Lower mixing capacity. Oxygen transfer not very high.	1.0 – 1.4
Horizontal shaft	The rotation is around the horizontal shaft. When rotor is rotating, a large number of fins perpendicular to the shaft cause aeration by spray and incorporation of air, besides providing the horizontal movement of the liquid in the reactor. Rotation speed: 20 – 60 rpm.	Activated sludge oxidation ditches (depth less than 2.5 m)	Motor, reducer, rotor.	Moderate initial cost. Easy to fabricate locally. Easy access for maintenance.	Limited shape of the tank. Low depth requirement. Possible problems with long shaft rotors. Oxygen transfer not very high.	1.2 – 2.0

Source: Arceivala (1981), Qasim (1985), Metcalf & Eddy (2002), Malina (1992), WEF & ASCE (1992) oxygen transfer rate in mechanical aerator systems:

- switch on and off certain aerators
- vary the rotation speed of the aerators
- vary the submergence of fixed aerators through the alteration of the level of the outlet weir (change in the water level)
- vary the submergence through the change of the level of the aerator shaft

11.8 DIFFUSED AIR AERATION SYSTEMS

The diffused air aeration system is composed of diffusers submerged in the liquid, air distribution piping, air transport piping, blowers, and other units through which the air passes. The air is introduced close to the bottom of the tank and the oxygen is transferred to the liquid medium while the bubble rises to the surface.

The main diffused air systems can be classified according to the porosity of the diffuser and the size of the bubble produced:

- **porous diffuser** (*fine* and *medium bubbles*): plate, disc, dome, tube (ceramic, plastic, flexible membrane)
- **non-porous diffuser** (*coarse bubbles*): nozzles or orifices
- **other systems**: jet aerator, aspirating aerator, U-tube aerator

Figure 11.6 presents an schematics of the aeration by porous diffusers and aspirating devices. Aspirating devices have an impeller at the lower end (immersed in the liquid), which, when rotating, create a negative pressure, sucking in atmospheric air through a slot situated at the upper end (outside the liquid). Air is diffused into the liquid medium in the form of small bubbles, which are responsible for the oxygenation and mixing of the liquid mass. The aspirating aerators are presented in some texts as mechanical aerators, since they have motors that rotate outside the liquid, and in other texts as diffused air aerators, because they generate air bubbles in the liquid medium.

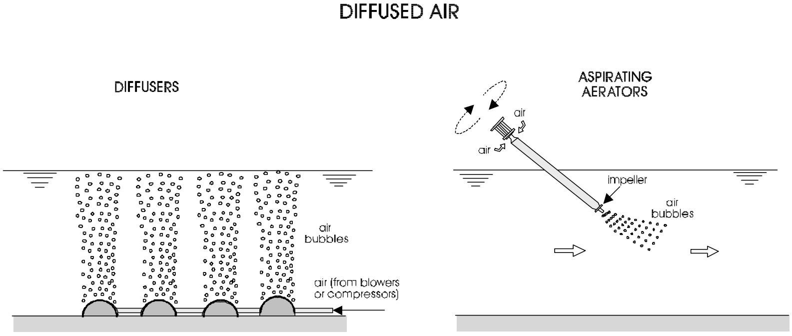


Figure 11.6. Diffused air aeration by porous diffusers and by aspirating devices

The diameters of the bubbles considered in the classification of the aeration type are (ABNT, 1989):

- *fine bubble*: diameter less than 3 mm
- *medium bubble*: diameter between 3 and 6 mm
- *coarse bubble*: diameter greater than 6 mm

In general, the smaller the size of the air bubbles, the greater the surface area available for gas transfer, that is, the greater the oxygenation efficiency. For this reason, aeration systems with fine bubbles are the most efficient in the transfer of oxygen.

The oxygen transfer efficiency of the porous diffusers decreases with the use, due to the internal or external clogging. The internal clogging is due to impurities in the air that are not removed by the filter. The external clogging is due to bacterial growth on the surface, or the precipitation of inorganic compounds.

The oxygen transfer rate can be changed to adjust itself to the oxygen consumption through the control of the blowers and the air distribution system, thus allowing energy savings.

Table 11.6 presents the characteristics of the main diffused air aeration systems.

11.9 AERATION TESTS

Wastewater treatment plants are designed based on a desired oxygen transfer rate from the aeration system. Normally this transfer rate, whether expressed in standard conditions or in field operating conditions, is part of the specification for purchasing the aeration equipment. Unfortunately, it has not been a common practice the undertaking of aeration tests to verify if the equipment being supplied satisfies the required oxygen demand. Even with the tests carried out in the manufacturer's laboratory, the transformation of the standard condition values to the real situation in the treatment plant is difficult, because of the various influencing factors, such as the tank shape, number and placing of the aerators and others.

In the existing treatment plants it is very important to know the oxygenation capacity of the installed equipment. In the same way that the influent quality is monitored in order to allow the estimation of oxygen ***consumption*** (BOD), it is equally important to have the knowledge of *the real capacity of oxygen* ***production*** *available in the reactor under operational conditions*.

This aspect becomes even more important, considering that there is an optimal operating point that leads to the greatest oxygen transfer efficiency (mass of O_2 supplied per unit of energy consumed). For instance, in reactors with mechanical aeration, this point is obtained at a certain submergence of the aerators, which can be achieved through the adjustment of the level of the outlet weir (which may be also variable during the day). Hence, it is important that aeration tests be carried out under operational conditions, aiming at determining the level of the outlet weir that leads to the supply of the *required O_2 mass* within the *greatest possible transfer efficiency*. Considering that the greatest energy costs in a treatment plant are related with aeration, the economy resulting from this procedure can be considerable.

Table 11.6. Characteristics of the main diffused air systems

Aeration type	Characteristics	Application	Advantages	Disadvantages	Average standard oxygen transfer efficiency (%)	Standard oxygenation efficiency (kgO_2/kWh)
Fine bubbles	The bubbles are produced in plates, discs, tubes or domes, made of a ceramic, glass or resin medium	Activated sludge	High oxygen transfer. Good mixing capacity. High operational flexibility through the variation of the airflow.	High initial and maintenance costs. Possibility of clogging of the diffusers. Air filters are necessary.	10–30	1.2–2.0
Medium bubbles	The bubbles are produced in perforated membranes or perforated tubes (coated stainless steel or plastic)	Activated sludge	Good mixing capacity. Reduced maintenance costs.	High initial costs. Air filters could be necessary.	6–15	1.0–1.6
Coarse bubbles	The bubbles are produced in orifices, nozzles, or injectors.	Activated sludge	No clogging. Low maintenance costs. Competitive initial costs. Air filters are not necessary.	Low oxygen transfer. High-energy requirements.	4–8	0.6–1.2
Aspirating aerators	The bubbles are produced by a propeller rotating at high speed at the bottom of a tube, which sucks in atmospheric air through the orifice at the upper end of the tube.	Aerated lagoons, activated sludge	No clogging. Air filters are not necessary. Conceptual simplicity. Maintenance relatively simple.	Lower oxygenation efficiency compared to mechanical aeration or fine bubble systems.	–	0.6–1.2

Source: Qasim (1985), Metcalf & Eddy (1991), Malina (1992), WEF & ASCE (1992)

The aeration tests in an activated sludge plant can be done according to one of the following methods:

- *test with clean water*
    - steady-state method
    - unsteady-state method
- *test under operating conditions*
    - steady-state method
    - unsteady-state method

The **clean water test** requires that the tank is emptied and filled with clean water. The dissolved oxygen (DO) in the medium is removed through the addition of sodium sulphite in the reactor, with the aerators switched off. The aeration capacity is calculated based on the rate of increase in the DO concentration after turning off the aerators. This method is expensive due to the requirement of large volumes of treated water and chemical products, besides being often inpractical in operating plants.

The **test under operating conditions** is accomplished with the reactor in operating conditions containing the mixed liquor. The oxygen consumption results from the respiration of the biomass in the mixed liquor. This method is cheaper, since it does not require the addition of external products, and can be done with the treatment plant in operation. Even though the results can be less accurate than those with clean water (if the DO concentration remains low, even with the aerators turned on), the values obtained provide a direct indication of the Oxygen Transfer Rate really available in the system.

The **steady-state method** is that in which all the conditions in the reactor are constant (or practically constant), that is, there are no variations during the test period. Under these conditions, the oxygen consumption is equal to its production. The aeration capacity can be then estimated through the determination of the oxygen utilisation rate by the biomass.

The **unsteady-state** method (reaeration method) consists in turning on the aerators, aiming at increasing the dissolved oxygen concentration in the medium. The Oxygen Transfer Rate is associated with the measured rate of increase in the DO concentration.

The references Boon (1980), de Korte and Smits (1985), Stephenson (1985), WPCF & ASCE (1988), ASCE (1990), WEF & ASCE (1992) and von Sperling (1993) present descriptions of the aeration tests. Under operating conditions (more frequent and practical situation), the tests can be carried out in the following simplified way:

Steady-state method:

1. determination of the oxygen consumption rate (r)
2. production rate = consumption rate
3. calculation of the oxygen transfer coefficient K_La:

$$\boxed{K_La = \frac{r}{(C_s - C)}} \qquad (11.33)$$

Unsteady-state method:

1. determination of the oxygen consumption rate (r)
2. production rate: determination by the DO trajectory after switching on the aerators
3. calculation of the oxygen transfer coefficient K_La:

$$\boxed{C = C_\infty - (C_\infty - C_o).e^{-K_La.(t-t_o)}} \tag{11.34}$$

$$\boxed{C_\infty = C_s - (r/K_La)} \tag{11.35}$$

The hydraulic regime has a large influence on the determination of the coefficient K_La. Table 11.7 presents a summary of the formulas to be used in the tests under operating conditions. In some of them K_La is explicit, while in others K_La should be obtained by regression analysis with the various values of the pairs DO × t (von Sperling, 1993). Even though regression analysis with the original equation 11.34 is frequently the preferred method to estimate the coefficient K_La, there are some alternative approaches based on the transformation of the basic equation. Example 11.5 presents a process based on the adoption of logarithms on both sides of Equation 11.34.

Example 11.5

Determine the K_La coefficient of the aeration system of a reactor with clean water, which has had the dissolved oxygen previously removed by the addition of sodium sulphite. After the removal of the DO, there was no further oxygen demand. After turning on the aerator, the DO values as a function of time were measured, and are presented in the table below. The saturation concentration of DO in the liquid, as a function of temperature and altitude was estimated as 8.4 mg/L.

t (s)	0	120	240	360	480	600	720	840
DO (mg/L)	0	1.0	1.8	2.5	3.2	3.8	4.3	4.8

Solution:

Applying the logarithm on both sides of Equation 11.34, after rearrangement:

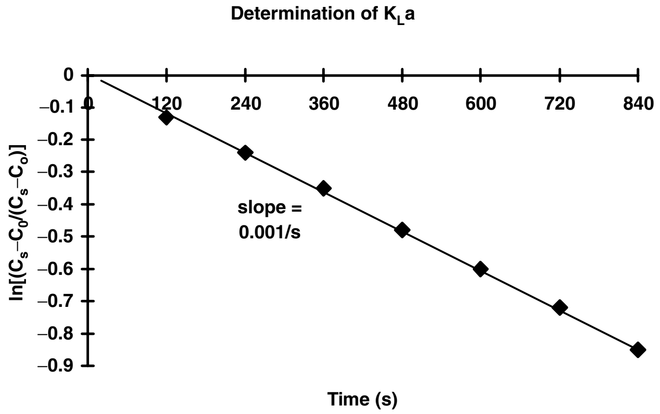
$$\ln\left(\frac{C_s - C}{C_s - C_o}\right) = -K_La.(t - t_o)$$

In a scatter plot on the Y-axis of the various values of $\ln[(C_s - C)/(C_s - C_o)]$ and on the X-axis the values of $(t - t_o)$, K_La corresponds to the slope of the line

***Example 11.5* (*Continued*)**

of best fit. The data necessary for the construction of the graph are calculated below:

t (s)	0	120	240	360	480	600	720	840
$\ln[(C_s - C)/(C_s - C_o)]$	0.00	−0.13	−0.24	−0.35	−0.48	−0.60	−0.72	−0.85



The slope of the line of best fit is 0.001 s^{-1}. This is the value of K_La determined in the experiment.

11.10 GRAVITY AERATION

In some cases, it may be interesting to increase the DO concentration in the effluent from a wastewater treatment plant, aiming at reaching higher concentrations in the water body, at the effluent-river mixing point. A simple way to achieve this is through the process of **gravity aeration**, used in some treatment plants.

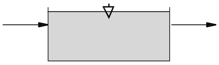
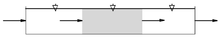
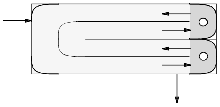
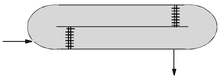
Gravity aeration takes place in weirs or in steps in aeration cascades. Effluents from several wastewater treatment processes may benefit from an increased DO concentration. *However, it should be kept in mind that aeration is a gas transfer process: anaerobic effluents submitted to gravity aeration tend to release* H_2S*, which may cause bad odour and corrosion problems.*

The following text, based on von Sperling (1983b) and Pöpel (1979), describes the principles and application of gravity aeration.

The principle of gravity aeration is the use of the potential energy of the water to create gas–liquid interfaces for an efficient gas transfer. When the water passes over the crest of the weir or cascade, two different gas transfer mechanisms may occur:

- exposure of the water to the surrounding air
- exposure of the air to the water mass

Table 11.7. Formula for the determination of K_La according to various hydraulic regimes

Hydraulic regime	Method	Volume of the reactor	Formula
Complete-mix reactor	Steady state	Total	$K_La = \dfrac{r}{(C_s - C)}$
	Unsteady state	Total	$C = C_\infty - (C_\infty - C_o).e^{-K_La.(t-t_o)}$ (nonlinear regression) $C_\infty = C_s - (r/K_La)$
Plug-flow reactor	Steady state	Complete mixing zone	$K_La = \dfrac{r}{(C_s - C)}$
	Unsteady state	Complete mixing zone	$C = C_\infty - (C_\infty - C_o).e^{-K_La.(t-t_o)}$ (nonlinear regression) $C_\infty = C_s - (r/K_La)$
Carrousel	Steady state (DO increase)	Complete mixing zone	$K_La = \dfrac{Q.(C - C_i) + r.V}{V.(C_s - C)}$
	Steady state (DO consumption)	Total (1 aerator turned on)	$K_La = \dfrac{r}{(C_s - C)}$
	Unsteady state	Total	$C = C_\infty - (C_\infty - C_o).e^{-K_La.(t-t_o)}$ (nonlinear regression) $C_\infty = C_s - (r/K_La)$
Pasveer ditch	Steady state (DO consumption)	Total (1 aerator turned on)	$K_La = \dfrac{r}{(C_s - C)}$
	Unsteady state	Total	$C = C_\infty - (C_\infty - C_o).e^{-K_La.(t-t_o)}$ (nonlinear regression) $C_\infty = C_s - (r/K_La)$

The first mechanism relates to the *exposure of the water to the surrounding air*, which occurs during the free fall. If the fall height H is known, the average air exposure time [$t = (2H/g)^{0.5}$] can be estimated, which allows an evaluation of the gas transfer coefficients. The configuration of the water fall crest influences the aeration, because the subdivision of the flow into several jets increases the air-water contact area, enabling an increased efficiency of the gas transfer operation.

The second mechanism refers to the *exposure of air to the water mass*, exactly the reverse of the first phenomenon. It occurs due to the submergence of the flow into the bulk of the liquid located on the base of the waterfall, causing significant amounts of air to be absorbed. The incorporated air is then dispersed under the form of bubbles in the liquid, leading to an intense gas transfer. The amount of air absorbed in the second mechanism depends primarily on the velocity [$v = (2gH)^{0.5}$] of the jet passing through the surface of the downstream water. Consequently, the gas transfer is substantially determined by the height of the fall, in a much more significant manner than in the first mechanism mentioned.

Besides that, the depth of the receiving water influences the amount of gas transferred: the deeper the jet can submerge into the water mass, the larger the specific surface area and the longer the contact time between the bubbles and the water. For an optimal utilisation of this effect, the depth should be such that the final velocity of the jets prior to reaching the bottom is equal to the upward velocity of the bubbles produced.

In general terms, it is understood that the first mechanism is efficient for gas release, and the second for gas absorption. Thus, for example, hydrogen sulphide has better release conditions during the free fall phase, while oxygen is mostly absorbed after the submergence of the flow into the downstream water. In summary:

- *water surrounded by air*: predominance of gas release
- *air surrounded by water*: predominance of gas absorption

A large part of the oxygen absorption is also caused by the shock of the water jets against obstacles, allowing the subdivision of the falling liquid mass, thus increasing the exposure area. In addition to that, if the water does not fall freely, but attached to the face of the waterfall or steps, the aeration will be significantly reduced.

The effluent (downstream) oxygen concentration can be estimated based on the coefficient of gas transfer, named **efficiency coefficient (K)** in the case of gravity aeration. Knowing the K value for the water fall at issue, the effluent DO concentration can be estimated for different conditions of saturation and influent concentrations:

$$\boxed{C_e = C_o + K.(C_s - C_o)} \tag{11.36}$$

where:

C_o = influent (upstream) oxygen concentration (mg/L)
C_e = effluent (downstream) oxygen concentration (mg/L)
C_s = oxygen saturation concentration (mg/L)
K = efficiency coefficient (dimensionless)

The K coefficient is specific and constant for each aeration system (in this case, each water fall), provided that certain conditions, such as the influent flow, remain constant. In an existing waterfall, the K coefficient can be obtained by rearranging Equation 11.36.

$$K = \frac{C_e - C_o}{C_s - C_o} \tag{11.37}$$

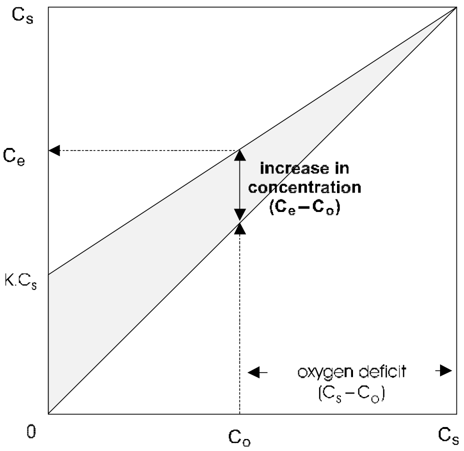


Figure 11.7. Estimated DO gains in gravity aeration

The conceptual graphic representation of Equation 11.36 is shown in Figure 11.7. In this figure, the C_e value is obtained from the K coefficient and the C_o and C_s concentrations. The figure emphasises the large influence of the oxygen deficit ($C_s - C_o$) on the increase of the oxygen concentration. For the case in which $C_o = 0$, the gain in the oxygen concentration will be $C_e - C_o = K.C_s$. Therefore, the K coefficient establishes the fraction of the saturation concentration to be gained in aerating a water completely devoid from oxygen. K is always lower than 1. It is also observed that, the higher (closer to C_s) the influent concentration C_o, the lower the oxygen deficit and, consequently, the lower the increase in the oxygen concentration ($C_e - C_o$). This can be the case of effluents from facultative and maturation ponds containing high DO contents.

The literature presents some empirical formulas for the determination of K according to the fall height (each single step) and other conditions (Table 11.8).

Equation 11.36 applies to *each step* or level of the cascade. For a system consisting of several free falls (e.g. steps), the overall K of the sequence of steps may be determined based on the individual K for each step according to:

$$K = 1 - [(1 - K_1).(1 - K_2) \cdot \cdots \cdot (1 - K_n)] \tag{11.38}$$

where:

K_1 = efficiency coefficient of the first free fall
K_2 = efficiency coefficient of the second free fall
K_n = efficiency coefficient of the nth free fall
K = overall efficiency coefficient of the system

Table 11.8. Formulas to determine the efficiency coefficient K for gravity aeration in weirs and cascades

Author	K coefficient	Coefficients of the equation
Barret, Gameson, and Ogden (apud Pöpel, 1979)	$K = P.(1 + 0.046.T).H$	$P = 0.45$ (clean water) $P = 0.36$ (polluted water) $P = 0.29$ (sewage)
Kroon and Schram (apud von Sperling, 1983b)	$K = R.H$	$R = 0.40$ $R = 0.64$ (in case of weirs with more than 4 jets per linear metre and falls lower than 0.70 m)
Parkhurst and Pomeroy (1972)	$K = 1 - e^{-F.H}$	$F = 0.53\ m^{-1}$ (clean water over weirs) $F = 0.41\ m^{-1}$ (slightly polluted water over weirs) $F = 0.28\ m^{-1}$ (treatment effluents over weirs)

Source: von Sperling (1983b)
H = height of each free fall (m)
T = temperature of the liquid (°C)

In the frequent case in which the steps have the same height, Equation 11.38 is simplified to:

$$\boxed{K = 1 - (1 - K_1)^n} \tag{11.39}$$

where:

n = number of equal free falls in the aeration system

Example 11.6

Estimate the effluent concentration of a step aeration, based on the following data:

- Influent DO concentration to the sequence of steps (effluent from the wastewater treatment plant): $C_o = 3.0$ mg/L
- DO saturation concentration: $C_s = 8.5$ mg/L
- $T = 20$ °C
- Height of each step: $H = 0.30$ m
- Number of steps: $n = 5$

Solution:

a) Determination of the K coefficient for each step

Based on the formulas on Table 11.8:

- Barret, Gameson, and Ogden

With $P = 0.33$ (adopted), $T = 20$ °C and $H = 0.30$ m:
$K = P.(1 + 0.046.T).H = 0.33 \times (1 + 0.046 \times 20) \times 0.30 = 0.19$

***Example 11.6* (*Continued*)**

- Kroon and Schram

With $R = 0.40$ (adopted) and $H = 0.30$ m:
$K = 0.40 \times 0.30 = 0.12$

- Pomeroy

With $F = 0.32$ (adopted) and $H = 0.30$ m:
$K = 1 - e^{-F.H} = 1 - e^{-0.32\times0.30} = 0.09$

Adopt an intermediate value, such as: $K = 0.13$.

b) Determination of the overall K coefficient of the step aeration system

Since the steps are equal, Equation 11.39 is used, with $n = 5$ steps:

$$K = 1 - (1 - K_1)^n = 1 - (1 - 0.13)^5 = 0.50$$

c) Determination of the effluent DO concentration

From Equation (11.36):

$C_e = C_o + K.(C_s - C_o) = 3.0 + 0.50 \times (8.5 - 3.0) = 5.8$ mg/L.

Therefore, the DO has been increased by 2.8 mg/L, and the concentration raised from 3.0 mg/L to 5.8 mg/L.

d) Calculation of the DO gain should the concentration of influent DO be 1.0 mg/L and 5.0 mg/L

For $C_o = 1.0$ mg/L:
$C_e = C_o + K.(C_s - C_o) = 1.0 + 0.50 \times (8.5 - 1.0) = 4.8$ mg/L (gain of 3.8 mg/L)

For $C_o = 5.0$ mg/L:
$C_e = C_o + K.(C_s - C_o) = 5.0 + 0.50 \times (8.5 - 5.0) = 6.8$ mg/L (gain of 1.8 mg/L)

The influence of the influent concentration, that is, of the DO deficit, is evident from these calculations.

e) Comments

The gain of DO can be optimised by trying different combinations of numbers of steps and individual heights of each step, within the total height available to allocate the step aeration system.

Part Two

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PART THREE

Stabilisation ponds

12

Overview of stabilisation ponds

The stabilisation pond systems constitute the simplest form of wastewater treatment. There are several variants of the stabilisation pond systems, with different levels of operational simplicity and land requirements. The following pond systems, whose main objective is the removal of carbonaceous matter, are covered in this part of the book:

- *Facultative ponds*
- *Anaerobic ponds followed by facultative ponds*
- *Facultative aerated lagoons*
- *Complete-mix aerated lagoons followed by sedimentation ponds*

Besides these ponds, *maturation ponds*, which may be included for the removal of pathogenic organisms, are also analysed.

There are still other variants of the pond systems, as listed in Chapter 4. However, in the present part of the book, only the ponds mentioned above are analysed in greater detail.

In general, stabilisation ponds are highly recommended for warm-climate areas and developing countries, due to the following aspects:

- sufficient land availability in a large number of locations
- favourable climate (high temperature and sunlight)
- simple operation
- little or no equipment required

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Table 12.1. Brief description of the main stabilisation pond systems

System	Description
Facultative pond	The soluble and fine particulate BOD is aerobically stabilised by bacteria that grow dispersed in the liquid medium, while the BOD in suspension tends to settle, being converted anaerobically by bacteria at the bottom of the pond. The oxygen required by the aerobic bacteria is supplied by algae through photosynthesis. The land requirements are high.
Anaerobic pond – facultative pond	Around 50 to 70% of the BOD is converted in the anaerobic pond (deeper and with a smaller volume), while the remaining BOD is removed in the facultative pond. The system occupies an area smaller than that of a single facultative pond.
Facultative aerated lagoon	The BOD removal mechanisms are similar to those of a facultative pond. However, oxygen is supplied by mechanical aerators instead of through photosynthesis. The aeration is not sufficient to keep the solids in suspension, and a large part of the sewage solids and biomass settles, being decomposed anaerobically at the bottom.
Complete-mix aerated lagoon – sedimentation pond	The energy introduced per unit volume of the pond is high, which causes the solids (principally the biomass) to remain dispersed in the liquid medium, in complete mixing. The resulting higher biomass concentration in the liquid medium increases the BOD removal efficiency, which allows this pond to have a volume smaller than that of a facultative aerated lagoon. However, the effluent contains high levels of solids (bacteria) that need to be removed before being discharged into the receiving body. The sedimentation pond downstream provides conditions for the removal of these settleable solids. The sludge of the sedimentation pond must be removed every few years.
Maturation ponds	The main objective of maturation ponds is the removal of pathogenic organisms. In maturation ponds prevail environmental conditions which are adverse to these organisms, such as ultraviolet radiation, high pH, high DO, lower temperature (compared with the human intestinal tract), lack of nutrients and predation by other organisms. Maturation ponds are a post-treatment stage for BOD-removal processes, being usually designed as a series of ponds or a single-baffled pond. The coliform removal efficiency is very high.

Table 12.1 presents a brief description of the main pond systems analysed in this part of the book, while Table 12.2 compares some basic characteristics of the systems. Table 4.21 (Chapter 4) presents the balance of advantages and disadvantages of each system. The corresponding flowsheets are presented in Figures 12.1 and 12.2.

It should be noticed that the ponds can work as *post-treatment* for effluents from *anaerobic reactors* (such as *UASB* – Upflow anaerobic sludge blanket). When the removal of pathogenic organisms is the main objective, these post-treatment ponds are also called *polishing ponds* (see Figure 12.3), but they are basically maturation ponds, and their design parameters are very similar to those adopted for maturation ponds. If aerated lagoons are adopted as post-treatment, the detention time can be reduced, as a result of the lower input of organic matter load to the pond.

Table 12.2. Characteristics of the main pond systems

		System of ponds				
General item	Specific item	Facultative	Anaerobic–facultative	Facultative aerated	Compl.-mix aerated – sedim.	Anaerobic – facultative – maturation
Removal Efficiency (%)	BOD	75–85	75–85	75–85	75–85	80–85
	COD	65–80	65–80	65-80	65–80	70–83
	SS	70–80	70–80	70–80	80–87	73–83
	Ammonia	<50	<50	<30	<30	50–65
	Nitrogen	<60	<60	<30	<30	50–65
	Phosphorus	<35	<35	<35	<35	> 50
	Coliforms	90–99	90–99	90–99	90–99	99.9–99.9999
Requirements	Area (m^2/inhab.)	2.0–4.0	1.2–3.0	0.25–0.5	0.2–0.4	3.0–5.0
	Power (W/inhab.)	≈ 0	≈ 0	1.2–2.0	1.8–2.5	≈ 0
Costs (US$/inhab)	Construction	15–30	12–30	20–35	20–35	20–40
	O & M	0.8–1.5	0.8–1.5	2.0–3.5	2.0–3.5	1.0–2.0

Notes: O&M: operation and maintenance
Costs based on Brazilian experience
Other comparative data are presented in Tables 4.9 and 4.10, Chapter 4.

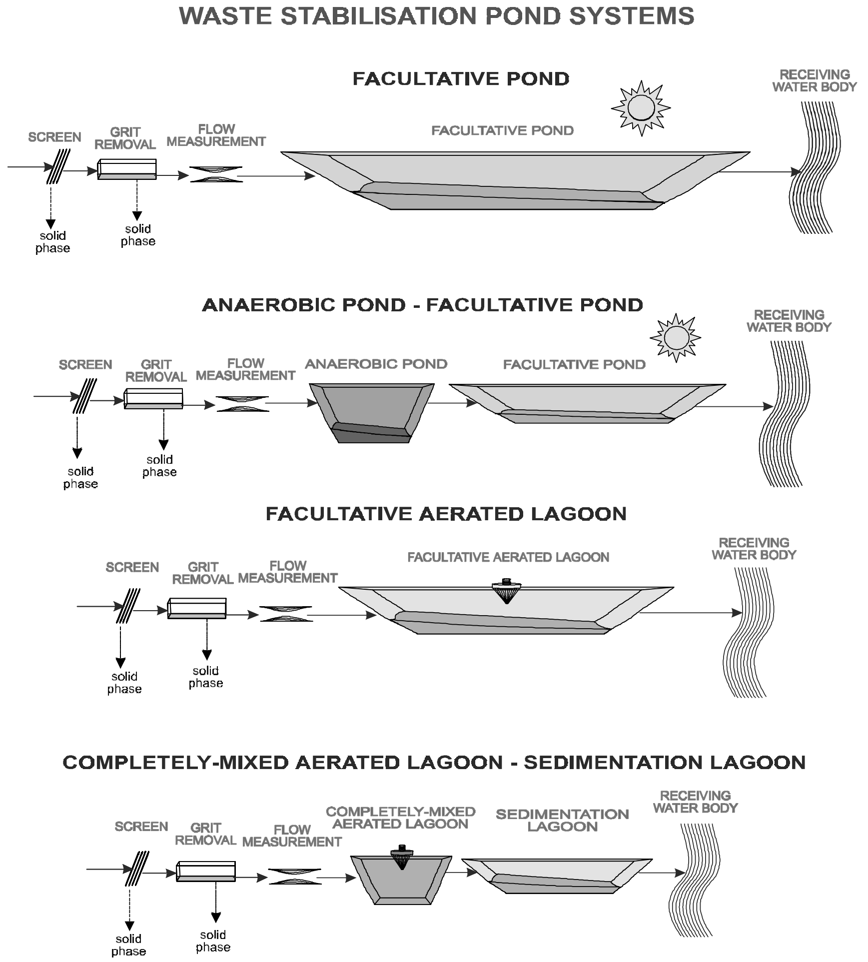

Figure 12.1. Flowsheets of the main stabilisation pond systems applied for the removal of BOD

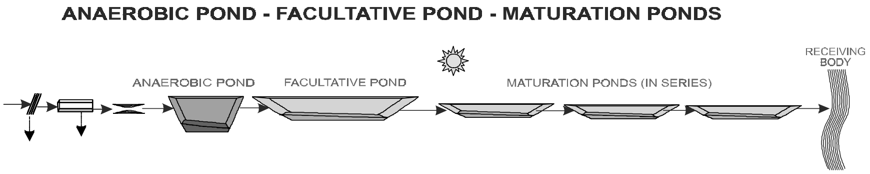

Figure 12.2. Flowsheet of a system of stabilisation ponds followed by maturation ponds in series

Table 12.3. Typical removal efficiencies of pathogenic and indicator organisms in stabilisation pond systems

	Typical removal efficiency (% or log units removed) (*)				
Parameter	Facultative	Anaerobic – facultative	Facultative – maturation	Anaerobic – facultative – maturation	UASB reactor – polishing pond
Coliforms	1–2 log	1–2 log	3–6 log	3–6 log	3–6 log
Pathogenic bacteria	1–2 log	1–2 log	3–6 log	3–6 log	3–6 log
Viruses	$\leq$ 1 log	$\approx$ 1 log	2–4 log	2–4 log	2–4 log
Protozoan cysts	$\approx$ 100%	$\approx$ 100%	100%	100%	100%
Helminth eggs	$\approx$ 100%	$\approx$ 100%	100%	100%	100%

(*) 1 log = 90%; 2 log = 99%; 3 log = 99.9%; 4 log = 99.99%; 6 log = 99.9999%

Table 12.4. Sludge management in stabilisation ponds

Item	Anaerobic	Primary facultative	Secondary facultative	Maturation
Sludge accumulation rate (m^3/inhab.year)	0.02–0.10	0.03–0.09	0.03–0.05	–
Removal interval (years)	< 7	> 15	> 20	> 20
Total solids concentration in the sludge (% TS)	> 10% (c)	> 10% (c)	> 10% (c)	–
VS/TS ratio	< 50%	< 50%	< 50%	–
Coliform concent. in the sludge (FC/gTS)	10^2–10^4	10^2–10^4	10^2–10^4	10^2–10^4
Helminth eggs concent. in the sludge (eggs/gTS)	10^1–10^3	10^1–10^3	10^1–10^3	10^1–10^3
Additional treatment required	Dewat. (a)	Dewat. (a)	Dewat. (a)	–
Usual disposal routes	(b)	(b)	(b)	–

Obs: prior grit removal is essential

(a) Disinfection (usually lime treatment) in the case of agricultural use of the sludge

(b) Final disposal routes similar to those used for the other wastewater treatment processes (agricultural reuse, landfill, others)

(c) When removed by pumping, the concentration can decrease to values of 5 to 7%

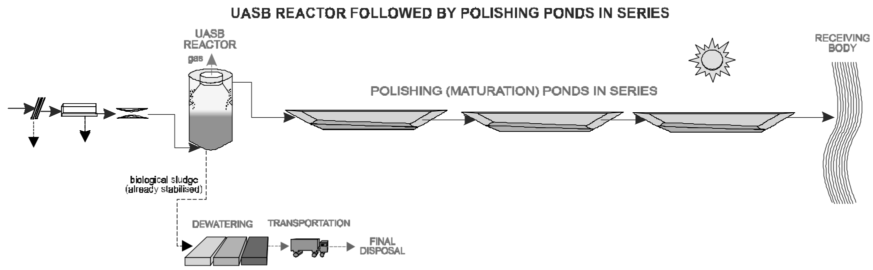

Figure 12.3. Flowsheet of a system of UASB reactor followed by polishing (maturation) ponds in series

Table 12.5. Typical design parameters for stabilisation pond systems

Design parameter	Anaerobic	Facultative	Facultative aerated	Completely mixed aerated	Sedimentation	Maturation
Detention time t (d)	3–6	15–45	5–10	2–4	≈ 2	(*)
Surface loading rate L_s ($kgBOD_5/ha.d$)	–	100–350	–	–	–	–
Volumetric loading rate L_V ($kgBOD_5/m^3.d$)	0.10–0.35	–	–	–	–	–
Depth H (m)	3.0–5.0	1.5–2.0	2.5–4.0	2.5–4.0	3.0–4.0	0.8–1.2
L/B ratio (length/breadth)	1 to 3	2 to 4	2 to 4	1 to 2	–	(**)
BOD removal coefficient K (complete mix) (20 °C) (d^{-1})	–	0.25–0.40	0.6–0.8	1.0–1.5	–	–
Temperature coefficient θ (complete mix)	–	1.05–1.085	1.035	1.035	–	–
BOD removal coefficient (dispersed flow) (20 °C)(d^{-1})	–	0.13–0.17	–	–	–	–
Temperature coefficient θ (dispersed flow)	–	1.035	–	–	–	–
Dispersion number d ($L/B = 1$)	–	0.4–1.3	–	–	–	0.4–1.1
Dispersion number d ($L/B = 2$ to 4)	–	0.1–0.7	–	–	–	0.1–0.5
Dispersion number d ($L/B \geq 5$)	–	0.02–0.3	–	–	–	0.03–0.23
Effluent particulate BOD ($mgBOD_5/mgSS$)	–	0.3–0.4	0.3–0.4	0.3–0.6	–	–
Average O_2 requirements ($kgO_2/kgBOD_5$ remov)	–	–	0.8–1.2	1.1–1.4	–	–
Power level (W/m^3)	–	–	<2.0	≥ 3.0	–	–
Coliform die-off coefficient K_b (complete mix) (20 °C) (d^{-1})	–	0.4–5.0	–	–	–	0.6–1.2 (***)
Temperature coefficient θ (complete mix)	–	1.07	–	–	–	1.07
Coliform die-off coefficient K_b (dispersed flow) (20 °C) (d^{-1})	–	0.2–0.3	–	–	–	0.4–0.7
Temperature coefficient θ (dispersed flow)	–	1.07	–	–	–	1.07

For details of the parameters: see text; L = length (m); B = breadth (m)

(*) The detention time in a maturation pond is a function of the pond shape and the required coliform removal efficiency

(**) L/B ratio including baffles in a single pond >10; L/B ratio in each pond of a series of more than 3 ponds: 1–5

(***) Coefficient K_b (complete mix) for maturation pond: values given are for ponds in series (baffled ponds are not well represented by the complete-mix model)

Regarding the removal of *pathogenic* organisms, a series of ponds *including maturation ponds* is capable of reaching very high removal efficiencies. Typical efficiencies of widely used pond systems for pathogen removal are presented in Table 12.3.

Sludge management in unaerated ponds is summarised in Table 12.4. Details are presented in the respective chapters, including the aerated lagoons. Sludge management is analysed specifically in Chapter 22.

A summary of the main design criteria adopted for the pond systems covered in this part of the book is presented in Table 12.5.

13

Facultative ponds

13.1 INTRODUCTION

Facultative ponds are the simplest variant of the stabilisation ponds systems. Basically, the process consists of the retention of wastewater for a period long enough, so that the natural organic matter stabilisation processes take place. Therefore, the main advantages and disadvantages of facultative ponds are associated to the predominance of natural phenomena.

The advantages are associated with the high operational simplicity and reliability. Natural processes are likely to be reliable: there is no equipment that can be out of order or the need for special operational schemes. However, nature is slow and needs long detention times so that the reactions are completed, which implies large land requirements. The biological activity is largely affected by temperature, mainly under the natural conditions of the ponds. As a result, the stabilisation ponds are more appropriate where the land is cheap, the climate is favourable, and a treatment method that does not require equipment or a special training for the operators is desired (Arceivala, 1981).

The costs of stabilisation ponds are very competitive, as long as the land costs or the need of earth works is not excessive. The construction is simple and involves mainly earth works, and the operational costs are much smaller than in other treatment methods. The efficiency of the system is usually satisfactory, and levels comparable to many secondary treatment systems can be obtained.

Figure 13.1 presents the typical flowsheet of a facultative pond.

A terminology frequently adopted for ponds is related to their position in the series of treatment units:

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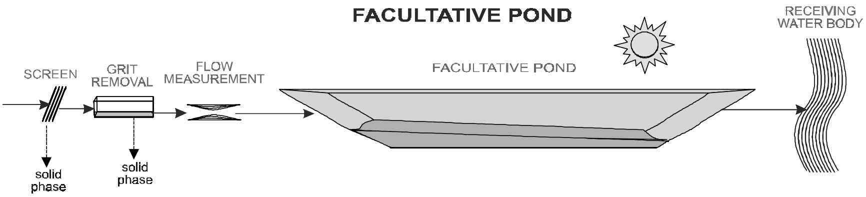


Figure 13.1. Flowsheet of a facultative pond

- *Primary pond*: first pond of the series - facultative pond that receives raw sewage
- *Secondary pond*: second pond of the series - receives effluent from another unit upstream (usually an anaerobic pond)
- *Tertiary, quaternary ponds*, etc.: occupy the third, fourth, etc. position in the series – they are usually maturation ponds

13.2 DESCRIPTION OF THE PROCESS

The influent wastewater enters at one end of the pond and leaves at the opposite end. During this time, which takes several days, a series of mechanisms contribute to the purification of the wastewater. These mechanisms occur in three zones of the ponds, denominated: *anaerobic zone, aerobic zone* and *facultative zone*.

The suspended organic matter (*particulate BOD*) tends to settle, constituting the bottom sludge (**anaerobic zone**). This sludge undergoes a decomposition process by anaerobic microorganisms, being slowly converted into carbon dioxide, methane and others. After a certain period, practically only the inert fraction (non-biodegradable) remains in the bottom layer. The hydrogen sulphide generated does not cause malodour problems, since it is oxidised by chemical and biochemical processes in the upper aerobic layer.

The dissolved organic matter (*soluble BOD*), together with the small suspended organic matter (*finely particulate BOD*) does not settle and remains dispersed in the liquid mass. In the upper layer, an **aerobic zone** is present. In this zone, the organic matter is oxidised by aerobic respiration. Oxygen is required, which is supplied to the medium by the photosynthesis undertaken by algae, and there is a balance between the consumption and production of oxygen and carbon dioxide (see Figure 13.2):

Bacteria → *respiration*:
- Consumption of oxygen
- Production of carbon dioxide

Algae → *photosynthesis*:
- Production of oxygen
- Consumption of carbon dioxide

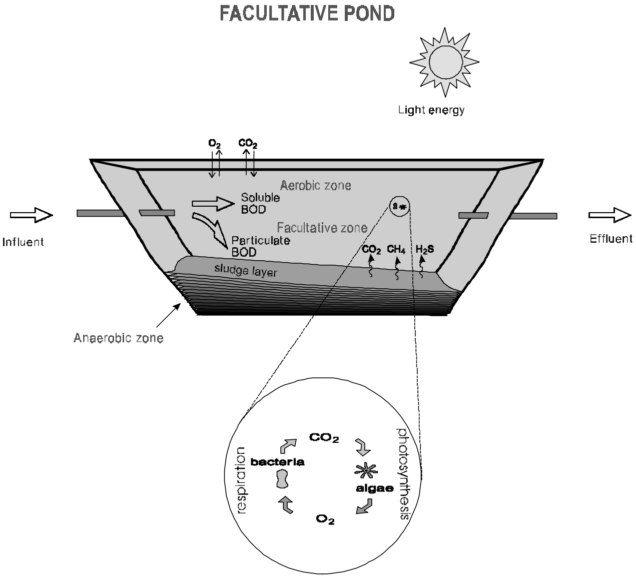


Figure 13.2. Simplified working principle of a facultative pond

It should be highlighted that the reactions of *photosynthesis* (production of organic matter) and *respiration* (oxidation of the organic matter) are similar, but with opposite directions:

- Photosynthesis:
  $CO_2 + H_2O + \text{Energy} \rightarrow \text{organic matter} + O_2$
- Respiration:
  $\text{Organic matter} + O_2 \rightarrow CO_2 + H_2O + \text{Energy}$

For the occurrence of photosynthesis, a source of *light energy* is necessary, in this case, represented by the sun. For this reason, localities with high solar radiation and a low level of cloudiness are highly favourable for facultative ponds.

Since photosynthesis depends on solar energy, it reaches higher levels close to the pond surface. Deeper down in the pond, light penetration is smaller, which causes the predominance of the oxygen consumption (respiration) over its production (photosynthesis), with the occasional absence of dissolved oxygen from a certain depth. Besides, photosynthesis only occurs during the day (sunshine hours), and during the night, the absence of oxygen can prevail. Because of these facts, it is essential that there are several groups of bacteria, responsible for the stabilisation of the organic matter, which can survive and proliferate in the *presence* as well as in the *absence* of oxygen. In the absence of free oxygen, other electron acceptors are used, such as nitrates (anoxic conditions). This zone, where the presence or the absence of oxygen can occur, is called a **facultative zone**. This condition also gives the name to the ponds (facultative ponds).

As commented, the process of facultative ponds is essentially natural and does not need any equipment. For this reason, the stabilisation of the organic matter takes place at slow rates, implying the need of a high detention time in the pond (usually greater than 20 days). Photosynthesis, to be effective, requires a high exposure area for the best use of the solar energy by the algae, justifying the need of large units. Consequently, the total area required by facultative ponds is the largest amongst all the wastewater treatment processes (excluding land disposal systems). On the other hand, the fact that they are a natural process is associated with a larger operational simplicity, which is a factor of fundamental importance in developing countries.

The effluent from a facultative pond has the following main characteristics (CETESB, 1989):

- green colour due to the algae
- high dissolved oxygen concentration
- high suspended solids concentration, although these practically do not settle (the algae practically do not settle in the Imhoff-cone test)

13.3 INFLUENCE OF ALGAE

Algae play a fundamental role in facultative ponds. Their concentration is much higher than that of bacteria, giving the greenish appearance of the liquid at the pond surface. In terms of dry suspended solids, their concentration is usually lower than 200 mg/L, although in terms of numbers they can reach counts in the range of 10^4 to 10^6 organisms per ml (Arceivala, 1981). The presence of algae is usually measured in the form of chlorophyll a, a pigment presented by all plants, and the main parameter for the quantification of the algal biomass (König, 2000). The chlorophyll a concentrations in facultative ponds depend on the applied load and temperature, but are usually located in the range from 500 to 2000 μg/L (Mara et al, 1992).

The main types of algae found in stabilisation ponds are (Mara et al, 1992; Silva Jr. and Sasson, 1993; Jordão and Pessoa, 1995):

- **Green algae** (Chlorophyta) and **pigmented flagellated** (Euglenophyta). These algae give the pond the predominant greenish colour. The main genera are *Chlamydomonas*, *Chlorella* and *Euglena*. *Chlamydomonas* and *Euglena* are usually the first to appear in the pond, tending to be dominant in cold periods, and possessing flagella, which gives them motility (optimisation of their position with relation to the incidence of light and to temperature).
- **Cyanobacteria** (previously called Cyanophyta or blue-green algae). In reality these organisms present characteristics of bacteria and algae, and are classified as bacteria. The cyanobacteria do not have locomotion organelles,

> such as cilia, flagella or pseudopodes, but are capable of moving by sliding. The nutrient requirements are very small: the cyanobacteria can proliferate in any environment that has at least CO_2, N_2, water, some minerals and light. These organisms are typical of conditions with low pH values and low nutrient availability in the wastewater. This environment (not typical in stabilisation ponds) is unfavourable for the green algae, which may also serve as food for other organisms, such as protozoa, leading to the proliferation of the cyanobacteria. *Oscillatoria*, *Phormidium*, *Anacystis* and *Anabaena* are among the main genera that can be mentioned.

Other types that can be found are algae of the phyla Bacyllariophyta and Chrysophyta (König, 2000; Mara et al, 1992). The predominant species vary from place to place, and even with the position in the series of ponds (facultative ponds and maturation ponds).

The algae photosynthesise during the hours of the day that are subject to light radiation. In this period, they produce the organic matter necessary for their survival, converting the light energy into condensed chemical energy in the form of food. During the 24 hours of the day, they respire, oxidising the organic matter produced, and release the energy for growth, reproduction, locomotion and others. The balance between oxygen production (photosynthesis) and consumption (respiration) widely favours the former. In fact, the algae may produce about 15 times more oxygen than they consume (Abdel-Razik, 1991), leading to a positive balance of DO in the system.

Owing to the requirement of light energy, most of the algae are located close to the pond surface, a location of high oxygen production. When deepening down into the pond, the light energy decreases, therefore reducing the algal concentration. In the surface layer, under 50 cm, is the range of higher light intensity, with the rest of the pond being practically dark.

There is a position in the pond depth in which the oxygen production by the algae equals the oxygen consumption by the algae and the decomposing microorganisms. This point is called **oxypause** (see Figure 13.3).

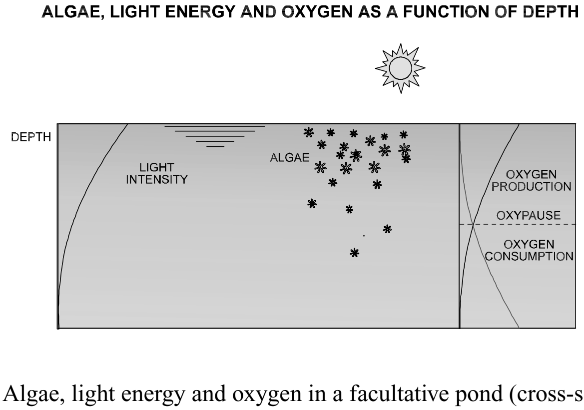


Figure 13.3. Algae, light energy and oxygen in a facultative pond (cross-section)

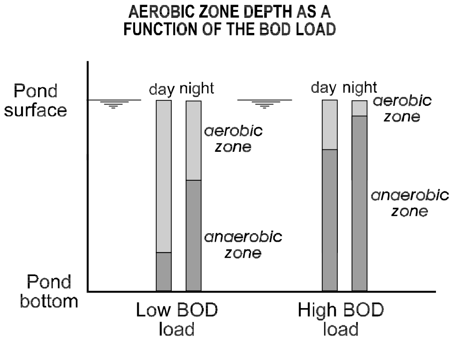


Figure 13.4. Influence of the organic load applied to the pond and the hour of the day on the thickness of the aerobic and anaerobic layers (adapted from Arceivala, 1981)

Above the oxypause, aerobic conditions prevail, while below it, anoxic or anaerobic conditions predominate. The level of the oxypause varies during the 24 hours of the day, as a function of the variability of the photosynthesis during this period. At night, the oxypause level rises in the pond, while during the day it lowers down.

The thickness of the aerobic zone, besides varying along the day, also varies with the loading conditions of the pond. Ponds with a greater BOD load tend to have a larger anaerobic layer, which can practically take up the whole pond depth during the night. Figure 13.4 schematically illustrates the influence of the loading conditions on the thickness of the aerobic layer.

The pH in the pond also varies with the depth and along the day. The pH depends on the photosynthesis and respiration, according to:

- *Photosynthesis:*
  - Consumption of CO_2
  - Bicarbonate ion (HCO_3^-) of the wastewater is converted to OH^-
  - pH rises
- *Respiration:*
  - Production of CO_2
  - Bicarbonate ion (HCO_3^-) of the wastewater is converted to H^+
  - pH decreases

During the day, in the hours of maximum photosynthetic activity, the pH can reach values around 9 or even more. In these conditions of high pH, the following phenomena can occur:

- Conversion of the ammonium ion (NH_4^+) to free ammonia (NH_3), which is toxic, but tends to be released to the atmosphere (nutrient removal)
- Precipitation of the phosphates (nutrient removal)
- Conversion of sulphide (H_2S), which may cause bad odours, to the odourless bisulphide ion (HS^-). At pH levels greater than 9 there is practically no H_2S.

Table 13.1. Influence of the main external environmental factors

Factor	Influence
Solar radiation	• Photosynthesis velocity
Temperature	• Photosynthesis velocity • Bacterial decomposition rate • Gas solubility and transfer • Mixing conditions
Wind	• Mixing conditions • Atmospheric reaeration (*)

(*) Mechanism of lesser importance in the DO balance in facultative ponds

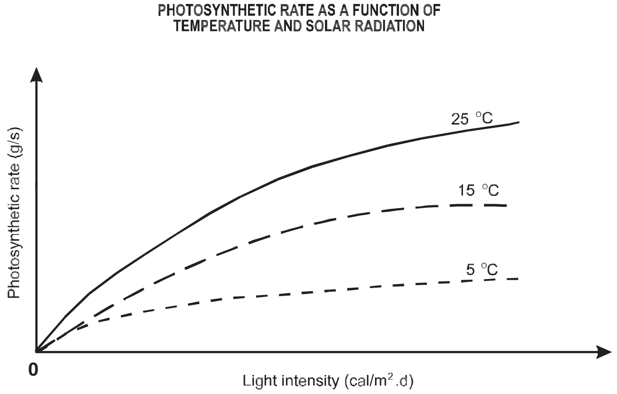


Figure 13.5. Influence of temperature and light radiation in the photosynthetic velocity (adapted from Jordão and Pessôa, 1995)

13.4 INFLUENCE OF ENVIRONMENTAL CONDITIONS

The main environmental conditions in a stabilisation pond are *solar radiation*, *temperature* and *wind* – see Table 13.1 (Jordão and Pessôa, 1995).

The influence of the temperature and solar radiation in the photosynthetic rate is shown schematically in Figure 13.5.

a) Mixing and thermal stratification

Mixing in a stabilisation pond occurs mainly through the following mechanisms: wind and temperature difference. Mixing is important for the performance of the pond due to the following beneficial aspects (Silva and Mara, 1979):

- Minimisation of the occurrence of hydraulic short circuits
- Minimisation of the occurrence of stagnant zones (dead zones)
- Homogenisation of the vertical distribution of BOD, algae and oxygen
- Transport to the photic surface zone of non-motile algae that would tend to settle

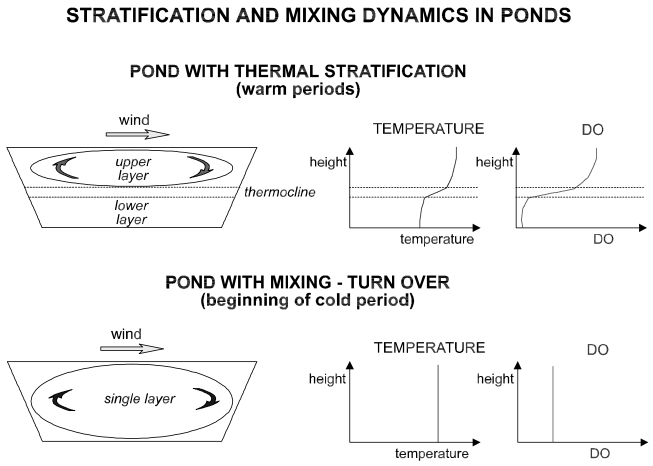


Figure 13.6. Stratification and mixing in a pond

- Transport to the deeper layers of the oxygen produced by photosynthesis in the photic zone

To maximise the influence of the *wind*, the pond should not be surrounded by natural or artificial obstacles that could obstruct the wind access. Additionally, the pond should not have a very irregular shape, which could hinder the homogenisation of the peripheral areas with the main pond body.

The pond is also subject to *thermal stratification*, in which the upper layer (warm) is not mixed with the lower (cold) layer. When deepening down in the pond, there is a point with a great decrease in the temperature, accompanied by high density and viscosity increases. This point is called the *thermocline*. Thus, two distinct layers are formed: the superficial one (lower density) and the bottom one (greater density), which are not mixed (see Figure 13.6).

The behaviour of the algae is influenced by the stratification according to:

- The non-motile algae settle and reach the dark zone of the pond, where they stop producing oxygen, leading, on the other hand, only to its consumption.
- The motile algae tend to escape from the upper surface layer (30 to 50 cm) of high temperature (occasionally 35 °C or more), and form a dense layer of algae, which hinders the penetration of the solar energy.

Because of these aspects, in stratified ponds there may be a low presence of algae in the photic zone, which reduces the oxygen production of the system and consequently its capacity to stabilise the organic matter. In locations with little or no wind at the pond surface, the pond may remain stratified.

The stratification can be interrupted by means of a natural mixing mechanism, denominated *turnover* or *thermal inversion* (see Figure 13.6). In stratified tropical lakes, the thermal inversion can take place in the cold period (winter). Besides this, in shallow lakes, such as stabilisation ponds, the mixing can happen once a day, according to the following sequence (Silva and Mara, 1979):

- *Beginning of the morning, with wind.* Complete mixing. The temperature is uniform throughout the depth.
- *Middle of the morning, with sun, without wind.* Increase of the temperature in the surface layer (above the thermocline). Little variation of the temperature at the bottom (below the thermocline), which is influenced by the ground temperature. Stratification.
- *Beginning of the night, without wind.* The layer above the thermocline loses heat more quickly than the bottom layer. If the temperatures of the layers become similar, mixing occurs.
- *Night, with wind.* The wind aids in the mixing of the layers. The upper layer sinks and the bottom layer rises.

Figure 13.7 shows experimental results (mean values) of temperature in a shallow pilot pond (1.0 m deep, with baffles, length/breadth ratio = 32), located in

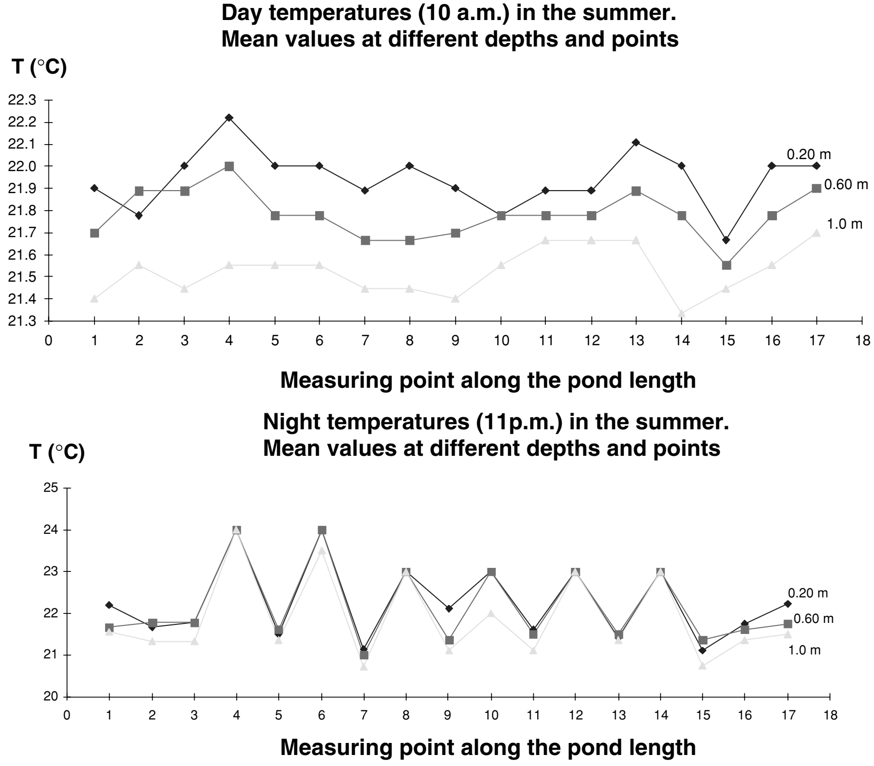


Figure 13.7. Longitudinal profile of the temperature in a pilot baffled pond, at daily and nightly hours. Measurements at the depths of 0.20 m, 0.60 m, and 1.00 m below the water level. Pond depth: 1.00 m.

Southeast of Brazil. The measurements were made at depths of 0.2 m, 0.6 m, and 1.0 m below the water level, and along the longitudinal course of the liquid in the pond. The figure shows summer data taken at 10 a.m., clearly indicating stratification (higher temperatures at the shallower depths, closer to the water level). However, at 11 p.m., also in the summer, the pond becomes totally mixed. The winter data is not presented here, but they indicate total mixing in the morning as well as in the night.

Kellner and Pires (1998, 1999) present a mathematical model for the estimation of thermal stratification in stabilisation ponds. They point out that the stratification leads to a loss of the net volume of the pond, and that the volume of the upper layer may be insufficient for the completion of the desired biochemical reactions.

b) Relationship between the air and the liquid temperature

The *average temperature of the liquid in the coldest month* is usually considered in many designs. Yanez (1993) and Brito et al (2000) present correlation studies between the air and the liquid temperature, in two ponds in Brazil, two in Peru and one in Jordan. The regressions are presented in Figure 13.8. The figure also presents a straight line, calculated by the author, based on the average values of the five equations. The resulting equation is:

$$\boxed{T_{\text{liquid}} = 12.7 + 0.54 \times T_{\text{air}}} \qquad (13.1)$$

Table 13.2 presents the resulting values of the water temperature calculated using Equation 13.1 for different values of the air temperature. The values obtained in the range of 20 to 30 °C are in agreement with the comment from Mara et al (1997) that the temperature of the pond is about 2 to 3 °C warmer than the temperature of the air in the cold period, the inverse occurring in the hot

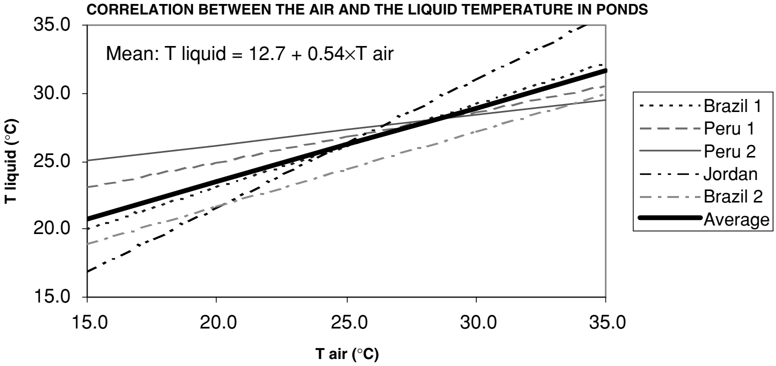


Figure 13.8. Lines of best fit for the regressions between the water and the air temperatures in five ponds. Data from Yanez (1993) and Brito et al (2000). Average line calculated by the author.

Table 13.2. Water temperature in the pond, as a function of the air temperature

Air temperature (°C)	Average liquid temperature (°C)
15	20.8
20	23.5
25	26.2
30	28.9
35	31.6

Estimation of the liquid temperature using Equation 13.1

period. An additional interpretation of the data from Yanez (1993) leads to the conclusion that the temperature in the surface of the pond is 1 to 5 °C higher than the average temperature, with the largest differences occurring in the warm period.

13.5 DESIGN CRITERIA

The main parameters for the design of facultative ponds are:

- Surface organic loading rate
- Depth
- Detention time
- Geometry (length / breadth (L/B) ratio)

Surface organic loading rate. The surface organic loading rate (organic load per unit area) is the main design criterion for facultative ponds. It is based on the need to have a certain exposure area to the sun light in the pond, so that the process of photosynthesis may take place. The objective of guaranteeing photosynthesis and algal growth is to have enough oxygen production to counterbalance the oxygen demand. Thus, the surface loading rate criterion is associated with the need of oxygen for the stabilisation of the organic matter. Therefore, the surface loading rate is related to the activity of *algae* and the balance between oxygen production and consumption.

Depth. The depth has an influence on the physical, biological, and hydrodynamic aspects of the pond. After obtaining the value of the surface area (through the adoption of a value for the surface loading rate) and adopting a value for the depth, the volume of the pond is obtained.

Detention time. The detention time is not a direct design parameter, but a verification parameter (resulting from the determination of the pond volume). The detention time criterion is associated with the time necessary for the microorganisms to stabilise the organic matter in the reactor (pond). Therefore, the detention time is related to the activity of the *bacteria*.

Pond geometry. The length to breadth (L/B) ratio is another important criterion, since it affects the hydraulic regime in the pond, which can be designed to approximate plug-flow or complete-mix conditions.

The design parameters are basically empirical. For the surface loading rate, there are some mathematical models that allow the design of facultative ponds based on conceptual methods, such as algae production as a function of the solar radiation, oxygen production per unit algal mass and others. However, such methods are outside the scope of the present book, where the approach is essentially simplified. Besides this, the empirical methods have been traditionally used, based on experience acquired in several areas of the world.

a) Surface organic loading rate

The area required for the pond is calculated as a function of the surface loading rate L_S. The rate is expressed in terms of the BOD load (L, expressed in $kgBOD_5/d$) that can be treated per unit surface area of the pond (A, expressed in ha).

$$\boxed{A = L/L_S} \tag{13.2}$$

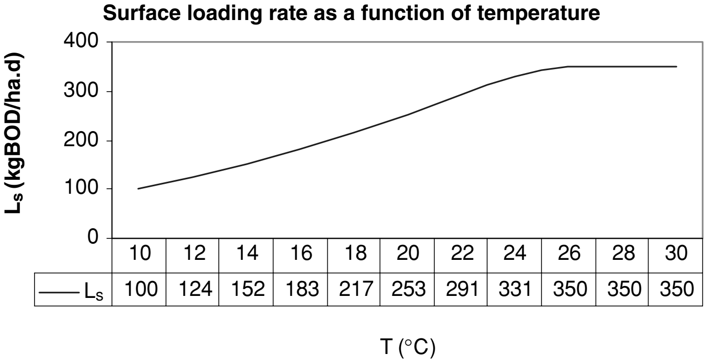
where:

A = area required for the pond (ha)
L = influent total (soluble + particulate) BOD ($kgBOD_5/d$)
L_S = surface loading rate ($kgBOD_5/ha.d$)

The rate to be adopted varies with the local temperature, latitude, solar exposure, altitude and others. Locations with extremely favourable climate and sunlight allow the adoption of very high rates, occasionally greater than 300 $kgBOD_5/ha.d$, which implies smaller surface areas. On the other hand, temperate climate locations require loading rates lower than 100 $kgBOD_5/ha.d$. In *tropical and subtropical-climate regions*, the following rates have been adopted:

- Regions with warm winter and high sunshine: $L_S = 240$ to $350\ kgBOD_5/ha.d$
- Regions with moderate winter and sunshine: $L_S = 120$ to $240\ kgBOD_5/ha.d$
- Regions with cold winter and low sunshine: $L_S = 100$ to $180\ kgBOD_5/ha.d$

There are several empirical equations available on the international literature, correlating the surface loading rate L_S with the temperature T. One of the equations, proposed by Mara (1997), is presented below. According to him, the equation has global applicability. The equation uses the **mean temperature of the air in the coldest month**. The reason for using the mean temperature of the air is that, in the cold period, a safe value is obtained, since the temperature of the water will be slightly higher. The selection of the cold period is because it is the most critical in the operation of the pond, in terms of the velocities of the biochemical reactions. In the design of the facultative ponds in this book, the mean temperature of the liquid in the coldest month is adopted (in order to calculate the BOD removal rates). However, to estimate the surface loading rate, the safe assumption proposed by Mara is adopted (that is, to consider the air temperature the same as the liquid temperature). Section 13.4.b discusses the relationship between the water and the air temperature.



T (°C)	10	12	14	16	18	20	22	24	26	28	30
—— Ls	100	124	152	183	217	253	291	331	350	350	350

Figure 13.9. Values of the surface loading rate as a function of the mean air temperature in the coldest month (according to Equation 13.3, Mara, 1997)

$$\boxed{L_s = 350 \times (1.107 - 0.002 \times T)^{(T-25)}} \tag{13.3}$$

where:

T = mean air temperature in the coldest month (°C)

The application of Equation 13.3 produces the values of L_s presented in Figure 13.9. Even though Equation 13.3 leads to very high values of L_s with high temperatures (above 25 °C), it is recommended that the surface loading rate be limited to a maximum value of 350 kgBOD/ha.d for design purposes.

Naturally, the use of an empirical formula is only for an initial estimate of the surface loading rate. As commented, if there are local experiences, as well as other climatic evidences that suggest the adoption of other values, these specificities should always be taken into consideration when selecting the value of L_s.

There is no absolute maximum value for the surface area, beyond which facultative pond systems become unfeasible. The desirability of adopting more compact systems if large ponds are required depends essentially on the local conditions, topography, geology and land cost. Similarly, the division of a single pond into ponds in parallel depends on topography and the desirability to have more flexibility and improved hydraulics.

b) Depth

As seen, the aerobic zone of the facultative pond depends on the penetration of sun light to give support to the photosynthetic activity. The intensity of light in the water body tends to reduce exponentially with depth. This phenomenon occurs even in distilled water, although at a much lower magnitude. The larger the colour and turbidity of the water and its algae concentration, the faster the light extinguishes. Below a certain depth in the pond, the environment is inappropriate for the growth of algae.

Table 13.3. Aspects related to the pond depth

Depth	Aspect
Shallow	• Shallow ponds, with depths lower than 1.0 m, can be completely aerobic. • The required area is very high, in order to comply with the detention time requirement. • The penetration of light through the depth is practically complete (the light energy tends to extinguish with depth, even in clean waters). • The production of algae is maximised and the pH is usually high (due to photosynthesis), causing the precipitation of phosphates and the stripping of ammonia (removal of nutrients). • Due to the low depth, there can be the development of emergent vegetation, which is a potential shelter for mosquito larvae (ponds with a depth around 0.60 m or less). • Shallow ponds are more affected by ambient temperature variations along the day, and can reach anaerobic conditions in warm periods (increase of the decomposition rate of the organic matter and a larger influence of the resolubilisation of by-products from the anaerobic decomposition of the sludge at the bottom).
Deep	• Ponds with higher depths provide a larger detention time for the stabilisation of the organic matter. • The performance of the pond is more stable and less affected by environmental conditions, producing an effluent with a more uniform quality throughout the year. • There is a larger storage volume for the sludge. • The bottom layer stays in anaerobic conditions, in which the BOD removal rate and the pathogenic death rate are slower. • The anaerobic decomposition obviously does not consume the dissolved oxygen in the medium. Thus, in the calculation of the DO balance, the fraction of the organic matter subject to the anaerobic decomposition can be taken into consideration. Usually, for a question of safety, the total influent BOD is considered to exert the oxygen demand, and for that the photosynthetic production in the upper layer should be sufficient. • The by-products of the anaerobic decomposition are released to the upper layers, still exerting some oxygen demand. The risks of bad smells are reduced, because in the aerobic layer the sulphide generated in the anaerobic decomposition is oxidised chemically and biochemically. • The deeper ponds allow future expansion for the inclusion of aerators, becoming aerated lagoons.

Based on the area and volume criteria, the depth H of the pond is a compromise between the required volume V and the required area A, considering that $H = V/A$. However, other aspects influence the selection of the depth of the pond (Arceivala, 1981), as listed in Table 13.3.

In conclusion, the available knowledge is still limited to optimise the depth of the pond, in order to maximise the number of benefits. The depth range to be adopted in the design of facultative ponds lies between 1.5 to 3.0 m, although the following range is more usual:

$$\boxed{H = 1.5\ \text{m to } 2.0\ \text{m}}$$

c) Detention time

The detention time of the pond is associated with the volume and the design flow:

$$\boxed{t = V/Q} \tag{13.4}$$

where:

t = detention time (d)
V = pond volume (m^3)
Q = average influent flow (m^3/d)

The average flow is the average of the influent flow and the effluent flow. The effluent flow corresponds to the influent flow minus the sinks plus the sources:

$$Q_{average} = (Q_{infl} - Q_{effl})/2 \tag{13.5}$$

$$Q_{effl} = Q_{infl} + Q_{precipitation} - Q_{evaporation} - Q_{infiltration} \tag{13.6}$$

The additional components in Equation 13.6 can usually be ignored. For example, in a location where the average annual precipitation is 1,000 mm/year, the evaporation is 2,000 mm/year, the influent flow is 3,000 m^3/d (1,095,000 m^3/year) and the surface area of the pond is 48,000 m^2 (flow and area of Example 13. 3), one has (ignoring the infiltration):

$$\begin{aligned} Q_{effl} &= (1{,}095{,}000\,m^3 year) + (1.0\,m/year \times 48{,}000\,m^2) \\ &\quad - (2.0\,m/year \times 48{,}000\,m^2) \\ &= 1{,}095{,}000 + 48{,}000 - 96{,}000 = 1{,}047{,}000\,m^3/year \end{aligned}$$

In this case, the annual loss is only 4.4% of the influent flow. However, depending on the circumstances, in certain dry months there may not be rainfall, at the same time that there is a substantial evaporation rate. In these cases, the water balance may be affected, and the loss (or occasional gain, in an opposite situation) can be more significant. Infiltration may also play an important role, especially in ponds with unsealed bottoms (see Chapter 20).

The detention time required for the oxidation of the organic matter varies with the local conditions, especially the temperature. In **primary** facultative ponds treating *domestic sewage*, the resulting detention times usually vary between:

$$\boxed{t = 15 \text{ to } 45 \text{ days}}$$

The lower detention times occur in areas where the liquid temperature is higher, and a reduction in the volume required for the pond is achieved. The required detention time is a function of the kinetics of the BOD removal and the hydraulic regime of the pond (see Section 13.6.1). In locations with concentrated sewage (low per capita sewage flow and a high BOD concentration), the detention time tends to be high.

With highly concentrated industrial wastewaters, the resulting detention time is usually much higher, because the pond area (and, indirectly, volume) is calculated based on organic load, and not on flow (which is comparatively low, for a given BOD load). The decisive factor, in the case of industrial effluents, continues to be the organic loading rate.

The surface loading rate and detention time criteria are complementary, that is, the area and the volume obtained should be coherent. The detention time can be used in one of the following two ways:

- *Adopt t as an explicit design parameter*. After t has been adopted, V is calculated (V = t.Q). Since the area A has been already determined based on the surface loading rate, the depth H can be calculated (H = V/A) and verified whether it is inside the range presented in Item b.
- *Adopt a value for the depth H, according to the criteria of Item b*. Having H and A, the volume V is calculated (V = A.H) and, in consequence, the detention time t (t = V/Q).

With the value of t, the effluent BOD concentration is estimated (see Section 13.6). If the effluent concentration does not satisfy the requirements, the volume, or the detention time, should be increased.

The second approach is more practical, because it adopts objective values for the surface area and depth. Example 13.3 shows the joint interpretation of these two criteria.

d) Geometry of the pond (length / breadth ratio)

As discussed in Section 13.6.1, the hydraulic regime of plug-flow is the most efficient in terms of the removal of constituents that follow first-order kinetics, such as the organic matter and coliforms. However, the complete-mix regime is more suitable when the wastewater is subject to highly variable loads and the presence of toxic compounds, due to the fact that complete-mix reactors provide an immediate dilution of the influent in the liquid mass (see Chapter 8 for further details).

Plug-flow reactors are also subject to a high oxygen demand close to the pond inlet, as a result of the arrival of raw wastewater, without dilution, in the body of the reactor. Anaerobic conditions can occur as a consequence of the localised organic overload (high organic loading rate in the inlet portion of the pond). For this reason, the following statements can be made:

- ***Primary facultative ponds:*** *not usually designed to approach plug-flow reactors* (high length/breadth ratio) with the introduction of baffles, due to the possibility of organic overload close to the pond inlet.
- ***Secondary facultative ponds:*** also not usually designed to approach plug-flow conditions, but there is *more flexibility in the selection of the L/B ratio*.

- ***Maturation ponds:*** most of the organic matter has been already previously removed, and there is less concern with an overload in the initial compartment. This allows the adoption of elongated ponds or baffles, leading to *high L/B ratios*.

The design of ponds can make use of the available site and its topography to obtain the most adequate length/breadth (L/B) ratio. Systems with high L/B tend to plug flow, while ponds with L/B close to 1.0 (square ponds) approach complete-mix conditions. More frequently, the L/B ratio for facultative ponds is situated within the following range (EPA, 1983; Abdel-Razik, 1991):

$$\boxed{\text{Length / breadth (L/B) ratio} = 2 \text{ to } 4}$$

13.6 ESTIMATION OF THE EFFLUENT BOD CONCENTRATION

13.6.1 Influence of the hydraulic regime

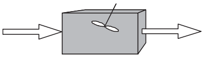
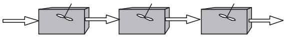
BOD removal follows a *first-order reaction* (in which the reaction rate is directly proportional to the substrate concentration). Under these conditions, the hydraulic regime of the reactor (pond) influences the efficiency of the system.

Although the kinetics of BOD removal are the same in the different hydraulic regimes, the effluent BOD concentration varies. According to the first-order kinetics, the BOD removal rate is higher the greater is the BOD concentration in the medium. This aspect has a great implication in the performance of the reactor, as seen below:

- **Plug-flow reactors**. In reactors in which there is a high BOD concentration (for example, close to the inlet), the removal rate is higher at this point. This is the case, for instance, of predominantly longitudinal reactors, such as the plug-flow reactors (the concentration close to the reactor inlet is different from the effluent concentration).
- **Complete-mix reactors**. Reactors that allow an immediate dispersion of the pollutant as a result of the homogenisation of the entire tank cause the influent concentration to rapidly equal the low effluent concentration. The low concentrations prevailing in the reactor lead to a lower BOD removal efficiency. This is the case of predominantly square complete-mix reactors (the concentration in the reactor, close to the inlet, is equal to the concentration at the outlet).

These two types of *idealised reactors* characterise an envelope, inside which all the existing reactors are placed in practice. Table 13.4 presents a description of the hydraulic models used in the representation of stabilisation ponds (see also Table 8.1).

Table 13.4. Characteristics of the hydraulic models more frequently used in the design and performance evaluation of stabilisation ponds

Hydraulic model	Reactor scheme	Characteristics
Plug flow		The fluid particles enter the tank continuously in one end, pass through the reactor, and are then discharged at the other end, in the same sequence in which they entered the reactor. The fluid particles move as a plug, without any longitudinal mixing. The particles maintain their identity and stay in the tank for a period equal to the theoretical hydraulic detention time. This type of flow is reproduced in long tanks with a large length-to-breadth ratio, in which longitudinal dispersion is minimal. Plug-flow reactors are idealised reactors, since complete absence of longitudinal dispersion is difficult to obtain in practice.
Complete mix		The particles that enter the tank are immediately dispersed in all the reactor body. The influent and effluent flows are continuous. The fluid particles leave the tank in proportion to their statistical population. Complete mixing can be obtained in tanks in which the contents are continuously and uniformly distributed. Complete-mix reactors are also known as CSTR or CFSTR (continuous-flow stirred tank reactors). Complete-mix reactors are idealised reactors, since total and identical dispersion is difficult to obtain in practice.
Complete-mix reactor in series		Complete-mix reactors in series are used to model the hydraulic regime of ponds in series or the regime that exists between the idealised plug flow and complete mix. If the series is composed of only one reactor, the system reproduces a complete-mix reactor. If the system has an infinite number of reactors in series, plug flow is reproduced. Influent and effluent flows are continuous. Reactors in series are also commonly applied to maturation ponds.
Dispersed flow		Dispersed or arbitrary flow is obtained in any reactor with an intermediate degree of mixing between the two idealised extremes of plug flow and complete mix. In reality, most reactors present dispersed-flow conditions. However, because of the greater difficulty in their modelling, the flow pattern is frequently represented by one of the two idealised hydraulic models. The influent and effluent flows are continuous.

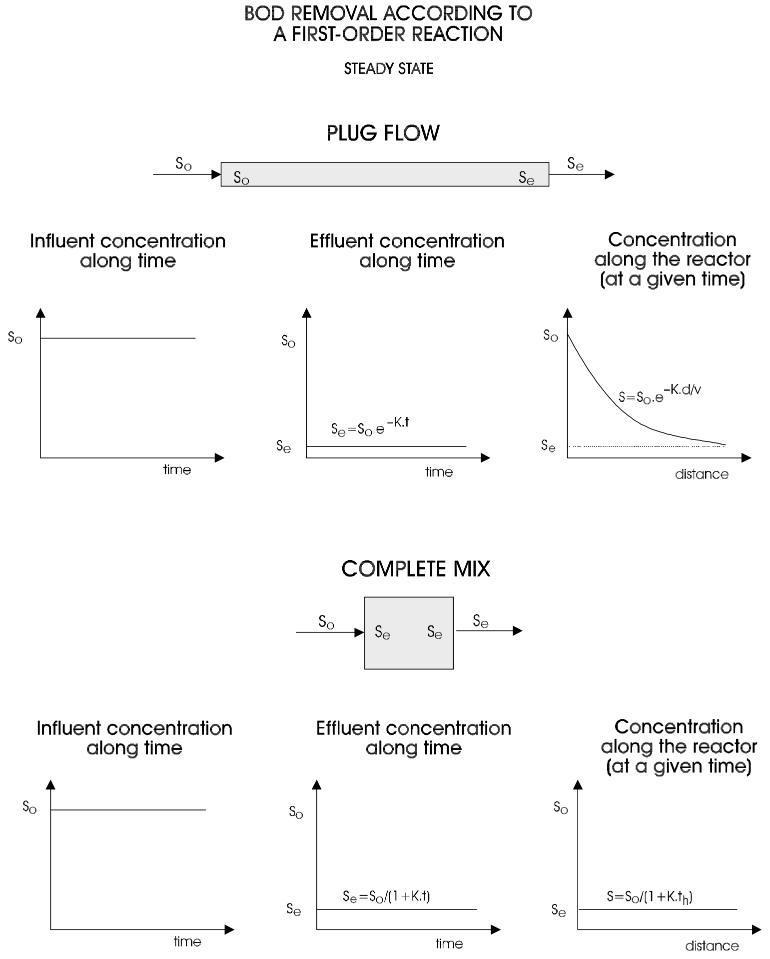


Figure 13.10. BOD removal according to first-order kinetics in plug-flow and complete-mix reactors (S_o = influent total BOD concentration; S = concentration of soluble BOD at a certain distance or time; S_e = effluent soluble BOD concentration; t = operating time; d = horizontal distance along the reactor; v = horizontal velocity; t_h = hydraulic detention time).

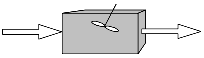
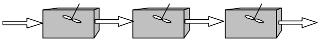
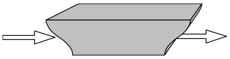
The efficiency in the removal of pollutants that are modelled according to first-order reactions (e.g. BOD and coliforms) follows the order presented below:

– *plug flow pond*	Greater efficiency
– *series of complete-mix ponds*	⇩
– *single complete-mix pond*	Lower efficiency

The dispersed-flow regime is not listed above because it can represent well reactors that approach both plug-flow and complete-mix conditions.

Figure 13.10 illustrates the behaviour of the BOD concentration in ponds according to the idealised plug-flow and complete-mix regimes, assuming a

Table 13.5. Formulas for the calculation of the effluent soluble BOD concentration (S)

Hydraulic regime	Scheme	Formula for the soluble effluent BOD_5 concentration
Plug flow		$S = S_o e^{-K.t}$
Complete mix (single cell)		$S = \frac{S_o}{1 + K.t}$
Complete mix (equal cells in series)		$S = \frac{S_o}{(1 + K\,t/n)^n}$
Dispersed flow		$S = S_o \cdot \frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}}$ $a = \sqrt{1 + 4K.t.d}$

S_o = total influent BOD concentration (mg/L)	t = total detention time in the system (d)
S = soluble effluent BOD concentration (mg/L)	n = number of ponds in series (–)
K = BOD removal coefficient (d^{-1})	d = dispersion number (dimensionless)

first-order removal reaction. Further details of this important topic are found in Chapter 8.

Table 13.5 presents the formulas for the determination of the soluble effluent BOD concentration for the various hydraulic regimes.

13.6.2 Soluble and particulate effluent BOD

It should be noticed that, in Table 13.5, **S** is the ***soluble* effluent BOD**. The **influent BOD S_o** is considered to be the ***total* BOD** (soluble + particulate), because the organic suspended solids, responsible for the particulate BOD, are converted into soluble organic matter, through the action of enzymes released into the medium by the bacteria themselves. Therefore, the bacteria assimilate the original soluble BOD of the wastewater (rapid assimilation) and the particulate BOD (after conversion to soluble BOD). Hence, in principle, the total BOD (soluble + particulate) would be available for the bacteria.

The **total effluent BOD** is also associated with two components:

- *soluble BOD*: mostly remaining BOD from the influent wastewater after treatment
- *particulate BOD*: BOD caused by the suspended solids in the effluent

The suspended solids in the effluent of facultative ponds are predominantly algae that may or may not exert some oxygen demand in the receiving water body, depending on their survival conditions. The following comments can be made (Arceivala, 1981; Abdel-Razik, 1991; Mara et al, 1997):

- If the algae die, the stabilisation of the organic fraction of their cellular mass will consume oxygen.

- If the algae are consumed by zooplankton and enter the food web, this can be advantageous for having a more productive environment, useful, for instance, for fish culture.
- If the algae continue to multiply themselves in the receiving water, they can lead to the beneficial effect of oxygen production. The algae undertake photosynthesis as well as respiration, but the amount of oxygen produced by photosynthesis during the sunny hours of the day is much greater than that consumed for respiration during the 24 hours of the day.
- If the effluent is used for irrigation, the algae can also be beneficial. Cyanobacteria contribute to the fixation of nitrogen, and other algae, when dead, slowly release nutrients used by the plants. Besides that, they increase the organic matter in the soil, enhancing its water retention capacity. However, excessive concentrations of algae can affect the soil porosity.

According to Mara (1995), the suspended solids from facultative ponds are about 60 to 90% algae. Each 1 mg of algae generates a BOD_5 around 0.45 mg. Consequently, 1 mg/L of suspended solids in the effluent is capable of generating a BOD_5 (in the BOD test, and not necessarily in the receiving body) in the range of $0.6 \times 0.45 \approx 0.3$ mg/L to $0.9 \times 0.45 \approx 0.4$ mg/L:

$$\boxed{1 \text{ mg SS/L} = 0.3 \text{ to } 0.4 \text{ mgBOD}_5\text{/L}}$$

Monitoring of some ponds in Brazil also leads to the following relationship, expressed in terms of COD:

$$\boxed{1 \text{ mg SS/L} = 1.0 \text{ to } 1.5 \text{ mgCOD/L}}$$

Owing to the uncertainty regarding these aspects, a practical approach can be the one of not considering the BOD from the algae (or from the suspended solids) in the effluent from facultative ponds. As a result, the BOD of the effluent from facultative ponds can be considered as being just the **soluble BOD**. In fact, the European Community established the following standards for the effluents from stabilisation ponds (Council of the European Communities, 1991):

- Soluble (filtered) $BOD_5 \leq 25$ mg/L
- Soluble (filtered COD) ≤ 125 mg/L
- Suspended solids ≤ 150 mg/L

The legislation from most countries makes no distinction between the BOD forms, and considers for the discharge standards the values of total BOD. The SS concentration in the effluent from facultative ponds usually complies with the European Community standards, although there can be occasional periods with values greater than those specified.

Unfortunately, there is no mathematical model that gives a reliable prediction of the suspended solids concentration in the effluent from a facultative pond, because of their great temporal variability as a function of the environmental conditions. For design purposes, the estimation of the particulate BOD may be based on effluent SS in the following range:

$$\boxed{\text{SS effluent} = 60 \text{ to } 100\,\text{mg/L}}$$

13.6.3 BOD removal according to the complete-mix model

Since the length / breadth (L/B) ratio usually employed in primary facultative ponds is in the order from 2 to 4, the hydraulic regime that occurs in fact is the dispersed flow (see Section 13.6.4). However, in the design of facultative ponds the complete-mix model (for one or more ponds) has been more frequently adopted due to the following reasons:

- The calculations with the complete-mix model are simpler.
- Facultative ponds are not especially elongated, and deviations from a complete-mix reactor are not substantial.
- Most of the BOD removal coefficients available in literature are for the complete-mix model.
- There is no need to determine the dispersion number of the pond

The value of the BOD removal coefficient (K) was obtained by several researchers at different existing ponds as a function of the influent and effluent BOD concentrations and the detention time. The value of K is always calculated as a function of the assumed hydraulic model. As a result, the values of K reported in the literature are associated with the hydraulic regime, and this fact needs to be taken into account when selecting the value to be adopted in the design of a new pond. As commented, most of the authors assume the complete-mix regime, but this hypothesis is not always explicit when presenting the values of K. When obtaining the value of K based on experimental data, the temperature, flow and the main geometric relationships of the pond (depth, length and breadth) must always be reported, besides the hydraulic model assumed in the calculations. Another point to remember is that, in the estimation of the K values, the BOD values to be considered are: (a) influent BOD: total BOD; (b) effluent BOD: soluble BOD.

For the most frequent case of the design according to the complete-mix model, the following range of K values may be used for design (Silva and Mara, 1979; Arceivala, 1981; EPA, 1983; von Sperling, 2001):

Pond	K value (20 °C)
Primary ponds (receiving raw wastewater)	0.30 to 0.40 d^{-1}
Secondary ponds (receiving effluent from a previous pond or reactor)	0.25 to 0.32 d^{-1}

Von Sperling (2001), analysing the BOD removal in 10 primary and secondary stabilisation ponds in Brazil, found the following mean values: primary ponds: $K = 0.40\ d^{-1}$ (4 data); secondary ponds: $K = 0.27\ d^{-1}$ (6 data); all the ponds: $K = 0.32\ d^{-1}$ (10 data). The value of K equal to $0.25\ d^{-1}$ (for COD) was found by the author and co-workers in a facultative pond treating effluent from a UASB reactor.

It is natural that the BOD removal coefficient is higher in primary facultative ponds, since the raw wastewater contains more easily biodegradable organic matter. On the other hand, the effluent from anaerobic ponds or anaerobic reactors has a more slowly biodegradable organic matter, since the more easily degradable fraction has been already removed in them. Consequently, the secondary facultative ponds, maturation ponds or polishing ponds should have lower K values.

For different temperatures, the value of K can be corrected using the following equation:

$$\boxed{K_T = K_{20}.\theta^{(T-20)}} \tag{13.7}$$

where:

K_T = BOD removal coefficient at a temperature T (d^{-1})
K_{20} = BOD removal coefficient at a temperature of 20°C (d^{-1})
T = liquid temperature (°C)
θ = temperature coefficient (–)

It should be noted that different values of θ are proposed in the literature. For $K = 0.35\ d^{-1}$, mentioned by EPA (1983), the temperature coefficient is $\boldsymbol{\theta = 1.085}$. For $K = 0.30\ d^{-1}$, mentioned by Silva and Mara (1979), the reported value is $\boldsymbol{\theta = 1.05}$.

When designing ponds or wastewater treatment plants, one should always keep in mind that the uncertainty in the design is not just in the coefficients of the model, but also in all the input data, starting from the design population and inflow. The design should always have this uncertainty in perspective, in order not to exaggerate in the sophistication in obtaining some coefficients, and forgetting to analyse the reliability of other data, which are possibly more influential (von Sperling, 1995a).

Example 13.1 illustrates the determination of the effluent soluble BOD concentration and the calculation of the resulting removal efficiency, for a given detention time and an adopted K value.

Example 13.1

Calculate the effluent soluble BOD concentration (S) in the following facultative ponds systems: (a) one plug-flow cell; (b) two complete-mix cells in series; (c) one complete-mix cell. Data:

- influent total BOD: $S_o = 300\ mg/L$
- BOD removal coefficient: $K = 0.30\ d^{-1}$ (adopted, for all the systems)

***Example 13.1* (*Continued*)**

- total detention time: t = 30 days
- liquid temperature: 20 °C

Solution:

Using the formulas from Table 13.5:

Hydraulic model	Formula		Soluble BOD (S) (mg/L)	Efficiency E (%)
Ideal plug flow (1 cell)	$S = S_o e^{-K.t}$	$S = 300.e^{-0,30\times 30}$	≤ 1	99.99
Ideal complete mix (2 cells)	$S = \frac{S_o}{(1+K\frac{t}{n})^n}$	$S = \frac{300}{(1+0.30\times\frac{30}{2})^2}$	10	97
Ideal complete mix (1 cell)	$S = \frac{S_o}{1+K.t}$	$S = \frac{300}{1+0.30\times 30}$	30	90

Efficiency: $E = (S_o - S).100/S_o$

Comments:

- Greatest efficiency is obtained with the plug-flow reactor
- Cells in series are more efficient than a single cell
- The results are obtained assuming that the ponds behave as ideal reactors, and that the value of K is the same, independent of the hydraulic regime
- For primary facultative ponds, the plug-flow model is not adequate, since the geometry of the ponds is not of a very elongated rectangle, in order to avoid organic overloading close to the inlet zone of the pond
- The calculated efficiencies are based on the soluble BOD in the effluent, and do not take into account the particulate BOD, also present in the ponds effluent.

13.6.4 BOD removal according to the dispersed-flow model

In reality, the hydraulic regime in a stabilisation pond does not exactly follow the ideal complete-mix or plug-flow models, but an intermediate model. The complete-mix and plug-flow models constitute an envelope, inside which all the reactors in reality are located. The **complete-mix** model represents one extreme (*infinite longitudinal dispersion*), while the **plug-flow** model represents the other extreme (*no longitudinal dispersion*). Inside these extremes are located the reactors modelled according to the **dispersed flow**, comprising all the ponds found in practice. For this reason, the knowledge of the dispersed-flow model is important, since it can be used as a better approximation for the design of stabilisation ponds.

However, modelling of a pond according to the dispersed flow model is more complicated, due to the need of two parameters (BOD removal coefficient and dispersion number), unlike the previous models, in which the knowledge of only the BOD removal coefficient is needed.

a) BOD removal coefficient K

The value of the BOD removal coefficient (K) can be obtained through one of the following empirical relations, obtained in studies of ponds modelled according to the dispersed flow regime:

- Arceivala (1981), after some simplifications by the author:

$$\boxed{K = 0.132 \times (\log_{10} L_s) - 0.146} \tag{13.8}$$

- Vidal (1983), after some simplifications by the author:

$$\boxed{K = 0.091 + 2.05 \times 10^{-4}.L_s} \tag{13.9}$$

It should be highlighted that the temperature coefficient (θ) for Arceivala's equation is **1.035**, differently from the coefficients expressed in Item 13.6.3. With relation to Vidal's equation, the temperature correction was not expressed in the usual Arrhenius form, but through analysis of the original formula, a value of θ lower than 1.035 is obtained.

Table 13.6 presents the values of K according to Arceivala and Vidal for different surface loading rates (for a liquid temperature of 20 °C and inside of the validity range of the equations). It can be observed that the values of K obtained by the two formulas are very similar. Experimental data obtained by the author and co-workers in facultative ponds acting as post-treatment for the effluent of UASB reactors showed good agreement with the removal coefficients K obtained with both equations.

Table 13.6. Values of the BOD removal coefficient (K, in d^{-1}) as a function of the surface loading rate, for the dispersed flow model (20 °C)

	L_s (kgBOD$_5$/ha.d)				
Equation	120	140	160	180	200
Arceivala (1981)	0.128	0.137	0.145	0.152	0.158
Vidal (1983)	0.116	0.120	0.124	0.128	0.132

b) Dispersion Number d

The other parameter to be determined is the Dispersion Number (d), which is expressed by Equation 13.10.

$$\boxed{d = D/U.L = D.t/L^2} \tag{13.10}$$

where:

d = Dispersion Number (−)

D = longitudinal dispersion coefficient (m^2/d)

U = mean longitudinal velocity along the reactor (m/d)
L = longitudinal length along the reactor (m)

When **d** tends to *infinity*, the reactor tends to the **complete-mix** regime. On the other hand, when **d** tends to *zero*, the reactor tends to the **plug-flow** regime.

The dispersion coefficient D is needed for the calculation of **d**. In existing reactors, D can be obtained experimentally by means of tests with tracers. For the design of new ponds, **d** is of course unknown, and its future value should be estimated according to some criterion. The literature presents some empirical relationships that can be used for this preliminary estimation:

- Polprasert and Batharai (1983):

$$d = \frac{0.184.t.\nu.(B + 2.H)^{0.489}.B^{1.511}}{(L.H)^{1.489}} \qquad (13.11)$$

- Agunwamba et al (1992), original formula simplified by the author:

$$d = 0.102.\left(\frac{3.(B + 2.H).t.\nu}{4.L.B.H}\right)^{-0.410}.\left(\frac{H}{L}\right).\left(\frac{H}{B}\right)^{-(0.981 + 1.385.H/B)} \qquad (13.12)$$

- Yanez (1993)

$$d = \frac{(L/B)}{-0.261 + 0.254.(L/B) + 1.014.(L/B)^2} \qquad (13.13)$$

- Von Sperling (1999)

$$d = \frac{1}{(L/B)} \qquad (13.14)$$

where:
L = length of the pond (m)
B = breadth of the pond (m)
H = depth of the pond (m)
t = detention time (d)
ν = kinematic viscosity of the water (m^2/d)

The kinematic viscosity of the water is a function of the temperature (see Table 13.7). Based on the data from Table 13.7, von Sperling (1999) proposed a correlation for the kinematic viscosity of the water as a function of the temperature (Equation 13.15).

$$\nu = 0.325.T^{-0.450} \qquad (13.15)$$

(for $T = 10$ to $30\,^{\circ}C$, $R^2 = 0.986$)

Table 13.7. Relation between the kinematic viscosity and the temperature of the water

Water temperature (°C)	Kinematic viscosity (m^2/d)
10	0.113
15	0.098
20	0.087
25	0.077
30	0.069

Source: Metcalf & Eddy (1991)

Table 13.8. Ranges of values of the Dispersion Number d, obtained through the use of the Agunwamba et al (1992), Yanez (1993) and von Sperling (1999) equations

Model	Length (m)	Depth (m)	$L/B = 1$	$L/B = 2$ to 4	$L/B = 5$ to 10
Agunwamba (Eq.13.12)	$L \leq 100$	1.5	0.4–0.7	0.1–0.4	0.03–0.17
		2.5	0.5–0.9	0.1–0.5	0.02–0.22
	$L > 100$	1.5	0.6–1.1	0.2–0.5	0.07–0.23
		2.5	0.7–1.3	0.2–0.7	0.10–0.30
Yanez (Eq.13.13)	–	–	1.0	0.24–0.46	0.1–0.2
von Sperling (Eq. 13.14)	–	–	1.0	0.25–0.5	0.1–0.2

Limits for the utilisation of Agunwamba's equation in this table: $t = 20$ to 40 d; $L \leq 300$ m; $T = 20\,°C$

In each column, for each range of L/B ratios, the smallest value of d corresponds to the largest L/B value

It should be highlighted that the dispersion number d can vary with time, in the same pond, as a result of the variation of environmental conditions, which affect the hydrodynamics of the pond. Kellner and Pires (1998) emphasise the limitations associated to the estimation of the dispersion in the pond, which should always be present in the interpretation of operational results.

However, in terms of design, a practical approach is needed, leading to the use of the empirical formulas. Equation 13.12 (Agunwamba et al, 1992) was reported to give a better fit to the experimental data than Equation 13.11 (Polprasert and Agarwalla, 1994). Table 13.8 presents ranges of average values of d obtained using Equations 13.12, 13.13 and 13.14. The equations of Agunwamba and Yanez lead to similar results, for ponds with lengths greater than 100 m. The equation of von Sperling is essentially a simplification of the Yanez equation, leading to practically the same values.

An additional comparison between the four Dispersion Number estimation methods was done by von Sperling (2003). A series of 1000 randomly generated independent sets of physical data was used to compare the values of d resulting from the four methods. In each one of the 1000 groups, the input data varied randomly, covering most of the situations found in practice. The ranges of variation

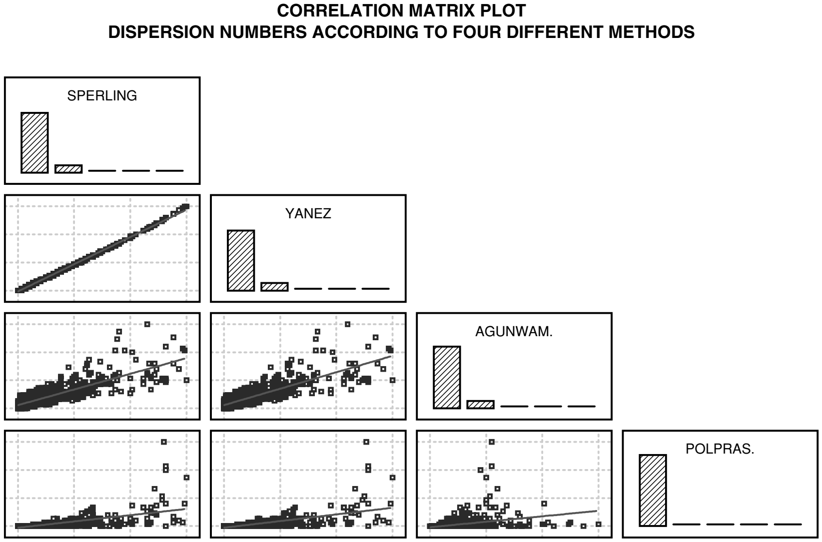


Figure 13.11. Scatter-plot of the 1000 values of d, generated according to the von Sperling, Yanez, Agunwamba et al and Polprasert and Batharai models

were: (a) length/breadth ratio: $L/B = 1$ to 16; (b) length of the pond: $L = 20$ to 300 m; (c) depth of the pond: $H = 1.0$ to 3.0 m; (d) hydraulic detention time: $t =$ 3 to 40 d; (e) liquid temperature: $T = 15$ to 25 °C.

Figure 13.11 shows the scatter-plot of the 1000 results of the Dispersion Number d obtained, according to the four methods. From the figure, it is clearly observed that: (a) the von Sperling and Yanez models lead to practically the same results, in all the d range values; (b) the Agunwamba model produces results close to the von Sperling and Yanez models, especially for lower d values; (c) the Polprasert model generates values that are very different from the three other models, especially in the upper half of the d values (according to von Sperling and Yanez), and in all the d values range (according to the Agunwamba model).

c) Relationship between removal coefficients for different hydraulic regimes

With respect to the removal coefficient K, in principle, its value should be the same for the complete-mix regime as well as for the plug-flow regime. However, in existing ponds, in most cases the value of K is estimated assuming the complete-mix model, knowing the BOD concentrations at the inlet (S_o) and outlet (S) and the detention time (t). Through rearrangement of the equation for S for the complete-mix model (Table 13.5), the value of K can be obtained. In this case, the value of K is overestimated, because in reality, the hydraulic regime is not the ideal complete mix, but the dispersed flow. Even for a square pond, the dispersion number d is equal to 1.0 (according to Yanez and von Sperling – Equations 13.13

Table 13.9. Ratio between the removal coefficients K obtained in the complete-mix regime and in the plug-flow regime, for different values of K.t (dispersed flow) and of the dispersion number d

K.t (dispersed flow)	K (complete mix) / K (dispersed flow)			
	d = 1.0 *L/B ≈ 1*	d = 0.5 *L/B ≈ 2*	d = 0.2 *L/B ≈ 4*	d = 0.1 *L/B ≈ 10*
0	1.00	1.00	1.00	1.00
1	1.14	1.23	1.40	1.52
2	1.29	1.52	1.95	2.32
3	1.46	1.83	2.68	3.55
4	1.64	2.21	3.66	5.39
5	1.83	2.65	4.95	8.18
6	2.04	3.15	6.62	12.28
7	2.27	3.73	8.81	18.21
8	2.53	4.39	11.60	26.81
9	2.79	5.14	15.16	39.11
10	3.08	6.01	19.66	56.50

and 13.14), which is far away from the higher values that characterise ideal complete mix.

Table 13.9 presents the correspondence between the values of K calculated according to the two hydraulic regimes (complete mix and dispersed flow), for different values of d (or L/B ratio) and the dimensionless pair K.t (for dispersed flow). For example, in a pond with an L/B ratio ≈ 2 (d = 0.5), detention time t = 27 d, K (dispersed flow) = **0.15 d^{-1}**, one has: K.t = 27 × 0.15 ≈4. The ratio K (complete mix) / K (dispersed flow) is, according to Table 13.9, for K.t = 4 and d = 0.5, equal to 2.21. This means that, if the coefficient K were determined in this pond assuming complete mix, a value of K = 2.21 × 0.15 = **0.33 d^{-1}** would be obtained. The values of K mentioned in the literature for the complete-mix regime are between 0.25 and 0.40 d^{-1} (see Section 13.6.3), that is, close to the value of 0.33 d^{-1} obtained in this example. However, if the pond had other geometric relationships and other detention times, the conversion of the coefficients could lead to very different values.

d) Removal efficiency

With the values of d and K (dispersed flow), the efficiency of the pond in the removal of BOD can be estimated, using the formulas presented in Table 13.5 for dispersed-flow reactors. In these equations, when d = 0, the formula produces results practically equal to those of the plug-flow equation. Similarly, when d = ∞ or, in practical terms, very high, the results are very close to those of the complete-mix equation. In the dispersed-flow formula, the second term of the denominator can be ignored, because it is usually very small. Example 13.2 illustrates the calculation for a pond of conventional dimensional relations.

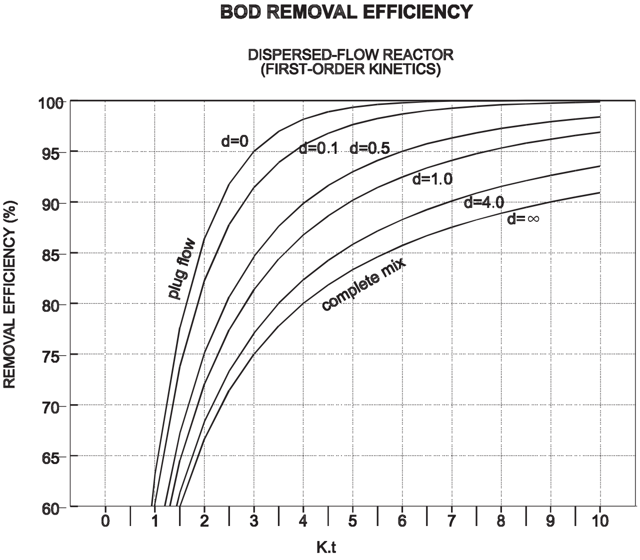


Figure 13.12. Removal efficiency of a compound following a first-order reaction (e.g.: BOD), for the main hydraulic models

To visualise these concepts, Figure 13.12 plots the dimensionless product K.t versus the BOD removal efficiency, based on a rearrangement of the classical graph of Thirumurty (1969).

Example 13.2

Calculate the effluent soluble BOD concentration (S) according to the dispersed-flow model, for a pond with the following data:

- Influent total BOD: $S_o = 300$ mg/L
- BOD removal coefficient for the dispersed-flow model: $K = 0.15\ d^{-1}$ (adopted, see Table 13.5)
- Detention time: $t = 30$ d
- Ratio length/breadth: $L/B = 2$

Solution:

a) Estimation of the dispersion number d

Considering that the L/B ratio is equal to 2, the dispersion number d is between 0.4 and 0.5, according to the formulas of Agunwamba et al, Yanez and von Sperling (see equations 13.12 to 13.14). Adopt, in the present example, the value of 0.4.

***Example 13.2* (*Continued*)**

b) Calculation of the effluent concentration S

According to the formulas presented in Table 13.5 for the dispersed-flow model:

$$a = \sqrt{1 + 4K.t.d} = \sqrt{1 + 4 \times 0.15 \times 30 \times 0.4} = 2.86$$

$$S = S_o.\frac{4ae^{1/2d}}{(1+a)^2e^{a/2d} - (1-a)^2e^{-a/2d}}$$

$$= 300.\frac{4 \times 2.86.e^{1/(2\times0.4)}}{(1+2.86)^2.e^{2.86/(2\times0.4)} - (1-2.86)^2.e^{-2.86/(2\times0.4)}} = 22\,\text{mg/L}$$

It can be observed that this value is in-between those obtained in Example 13.1 for one complete-mix cell (S = 10 mg/L) and two complete-mix cells in series (S = 30 mg/L).

The same considerations made above regarding the conversion of the coefficients K (dispersed flow) to K (complete mix) could have been made.

c) Calculation of the BOD removal efficiency

$$E = 100.(S_o - S)/S_o = 100 \times (300 - 22)/300 = 93\%$$

The same value could have been obtained through Figure 13.12, for $d = 0.4$ and $K.t = 0.15 \times 30 = 4.5$.

The calculated efficiencies are based on the soluble BOD in the effluent, and do not take into account the particulate BOD, also present in the pond effluent.

13.7 POND ARRANGEMENTS

The system of facultative ponds can be designed to have more than one pond, leading to a higher operational flexibility. When analysing the division of a pond into a larger number of units, the following aspects should be taken into consideration:

- **Cells in series**. In principle, a system of ponds in series, with a certain total detention time, has a greater efficiency than a single pond, with the same total detention time. The implication is that, for the same effluent quality, a smaller area can be occupied with a system of ponds in series. However, organic overloading in the first facultative pond in the series should be considered (see below).
- **Cells in parallel**. A system of ponds in parallel has approximately the same efficiency as a single pond (some difference may occur because of different dispersion numbers between the single pond and each pond in parallel). However, the system has more flexibility and guarantee, in case there is the need to interrupt the flow to a pond, owing to some problem or occasional maintenance (although this should be rare). As a consequence, the operation of the system will not be interrupted.

- **Organic overload in the first cell**. If the ponds are in series, it should be taken into account that the first cell may be overloaded, because it receives the entire influent load, with the possibility of having anaerobic conditions. The design should evaluate the oxygen balance in the first cell (production and consumption), or verify that the surface loading rate is not excessive in the first cell. To minimise this situation, cells of different sizes can be adopted, with the first unit having the largest area. However, the subsequent units could be considered to be more maturation ponds than facultative ponds as such. This overloading aspect is very important in primary ponds (that receive raw sewage), and *frequently restricts the utilisation of facultative ponds in series*. Ponds in series are more used for the removal of pathogens (maturation ponds), in which there should be no problems with organic overloading in the first cell.
- **Internal divisions**. The subdivision of a single pond into a larger number of ponds implies the need of intermediate embankments.
- **Plug flow**. Theoretically, an infinite number of cells in series corresponds to a plug flow, which would be the most efficient system for the removal of BOD. Thus, instead of having a high number of ponds in series, a single pond with a predominantly longitudinal pathway can be adopted, which can be obtained through a series of U-curves or baffles. In this case, the mentioned aspects of organic overloading close to the inlet zone should be taken into consideration. The plug flow is more used for the polishing of the effluent, such as in maturation ponds, in which there is no concern with organic overload in the inlet zone. For facultative ponds, Yanez (1993) suggests a maximum length/breadth ratio of 8:1. However, it is believed that lower ratios, of the order of 2 to 4 can be safer, from the point of view of organic overloading.

13.8 SLUDGE ACCUMULATION

The sludge accumulated in the bottom of the pond is a result of the suspended solids from the influent, including sand, plus settled microorganisms (bacteria and algae). The organic fraction of the sludge is digested anaerobically, being transformed into gases. Hence, the accumulated volume is lower than the settled volume.

The average sludge accumulation rate in facultative ponds is in the order of only 0.03 to 0.08 m^3/inhab.year (Arceivala, 1981). Silva (1993) and Gonçalves (1999) present average sludge thickness values around 1 to 3 cm/year. Because of this low accumulation rate, the occupation of the pond volume is very slow. Unless the pond receives a very high load, the sludge will accumulate for several years without the need for its removal.

From the accumulated sludge, only a small fraction is represented by grit. In spite of this, it can be necessary to remove the accumulated grit, since it tends to concentrate close to the inlet and in the first cell of a system in series. This emphasises the need for good preliminary treatment of the wastewater.

Table 13.10. Connection between the colour of the pond and operational characteristics

Pond colour	Interpretation
Dark green and partially transparent	• Unimportant presence of other microorganisms in the effluent • High pH and DO values • Pond in good conditions
Yellow green or excessively clear	• Growth of rotifers, protozoa or crustaceans, which feed on the algae and can cause their destruction in few days • If the conditions persist, there will be a decrease in DO and an occasional bad smell
Greyish	• Overload of organic matter and/or short detention time • Incomplete fermentation in the sludge layer • The pond should be put out of operation
Milky green	• The pond is in a self-flocculation process as a result of high pH and temperature • Precipitation of manganese and calcium hydroxides, sweeping the algae and other microorganisms
Blue greenish	• Excessive proliferation of cyanobacteria • The bloom of certain species forms a scum that decomposes easily, leading to the release of bad smells, reduction of light penetration and, as a consequence, reduction of oxygen production
Brownish red	• Overload of organic matter • Presence of photosynthetic sulphide-oxidising bacteria (they require light and sulphides, use CO_2 as an electron acceptor, do not produce oxygen and do not help in BOD removal)

Source: Arceivala (1981); CETESB (1989)

The anaerobic digestion of the bottom sludge can generate soluble non-stabilised by-products, which, when reintroduced into the upper liquid mass, are responsible for a new BOD load. This happens at a higher rate in the warmer periods. Thus, the summer months cannot necessarily be the best performance months of the pond (Abdel-Razik, 1991). The impact of this phenomenon will be larger or smaller, depending on the magnitude of the reintroduced BOD load, compared to the influent BOD load.

13.9 OPERATIONAL CHARACTERISTICS

The interpretation of the predominant colour in the pond can reveal important operational conditions (see Table 13.10). Some of these aspects are reviewed in Section 21, relative to maintenance and operation.

13.10 POLISHING OF POND EFFLUENTS

There are several possibilities for improving the quality of pond effluents, mainly aiming at the removal of suspended solids (algae). Some of the technologies are: (a) intermittent sand filters, (b) rock filters, (c) microsieves, (d) ponds with floating macrophytes, (e) land application, (f) wetlands, (g) coagulation and clarification processes, (h) flotation and (i) aerated biofilters (EPA, 1983; WPCF, 1990; Mara et al, 1992; Oliveira and Gonçalves, 1995; Gonçalves et al, 2000; Crites and Tchobanoglous, 2000).

The inclusion of any of these processes should naturally find a justification from the point of view of the needs of the receiving body (and not only as a safeguard in terms of compliance to discharge standards), since they imply an elevation of the treatment costs and complexity.

The post-treatment systems are more applicable for the improvement of the effluent from already existing ponds. Possibly, in new projects, if a high quality effluent in terms of BOD/COD and nutrients is required, other more efficient treatment systems should be adopted from the beginning, instead of the combination of facultative ponds with post-treatment. Some processes for the removal of algae are discussed below.

Rock filters. Rock filters consist of submerged stone porous beds, in which the algae settle, as the water flows through the bed. The algae are decomposed, releasing nutrients that are used by the bacteria growing on the surface of the filter. Besides the removal of algae, nitrification can also occur. The performance depends on the loading rate, temperature and size and shape of the stones. Loading rates are in the order of 1.0 m^3 of effluent per m^3 of rock medium per day. The stones have dimensions of about 50 to 200 mm – larger values reduce the surface exposure area, while smaller values can lead to clogging. The height of the bed is around 1.5 to 2.0 m. The pond effluent should be introduced below the surface layer to avoid odour problems. The unit can be located inside the pond. The costs are low and the operation simple, being associated with the periodic removal of the accumulated humus (Mara et al, 1992). The main disadvantages are associated with the possible generation of bad odours and the fact that the net life and the cleaning procedures are not yet totally established (WPCF, 1990; Crites and Tchobanoglous, 2000).

Intermittent sand filters. Intermittent sand filters are somewhat similar to the slow filters, operated in an intermittent way. The effluent is disposed periodically on the surface of the filter bed. The suspended solids and the organic matter are retained in the first 5 to 8 cm. After clogging, the surface sand layer is removed. The bed layer has a thickness of about 0.5 to 1.0 m, with a sand of effective size between 0.2 and 0.3 mm. The hydraulic loading rate is within 0.2 to 0.6 $m^3/m^2.d$, with the lower values associated with effluents with SS levels greater than 50 mg/L and cold periods (Crites and Tchobanoglous, 2000).

Floating macrophytes. The use of ponds with water hyacinths (*Eichhornia crassipes*) has been the object of considerable controversy. Water hyacinths are autotrophs (plants), meaning that they do not use the organic matter from the sewage. However, their root system allows the development of a biomass capable of stabilising part of the organic matter, besides adsorbing other pollutants, such as metals. The root system also contributes to a larger sedimentation of the suspended solids. Although there is no consensus on this subject, most people involved directly with the operation of these ponds comment that the problems outweigh the benefits.

Water hyacinths grow very fast, and it is necessary to have an infrastructure for their removal compatible with their growth rate, in order to avoid dead plants sinking to the bottom of the pond, where they undergo anaerobic conversion and allow the resolubilisation of the removed pollutants.

A macrophyte of simpler handling, owing to its smaller size, is the duckweed (*Lemna* sp.). The duckweed develops on the surface of the pond, decreasing the light penetration, which reduces the algal growth rate and leads to a more clarified effluent. Ponds with duckweed should be located at the end of the pond series, since their efficiency is lower than a maturation pond in the removal of coliforms, but they generate an effluent with lower SS levels. The duckweeds can be collected and serve as food for fish in other ponds.

Physical–chemical removal. The removal of SS by coagulation/flocculation can be done in a simple way locating the units inside the pond. Gonçalves et al (2000) inserted a mixing tank (t = 1 min), a granular flocculation unit (t = 7 min) and a laminar settling tank (hydraulic loading rate = 70 $m^3/m^2.d$) inside a facultative pond. The coagulant that produced better results was ferric chloride, with a dosage of 80 mg/L. Good removals of SS (73%), COD (58%) and phosphorus (83%) were reached. The sludge was recirculated to the anaerobic pond and no alteration in the performance of the anaerobic pond was observed.

Example 13.3

Design a treatment system composed of primary facultative ponds based on the following data:

- Population = 20,000 inhabitants
- Influent flow: $Q = 3{,}000\ m^3/d$
- Influent BOD: $S_o = 350$ mg/L
- Temperature: T = 23 °C (mean liquid temperature in the coldest month)

Solution:

a) Calculation of the influent BOD_5 load

$$\text{load} = \text{concentration} \times \text{flow} = \frac{350\,g/m^3.3000\,m^3/d}{1000\,g/kg} = 1{,}050\,kg/d$$

b) Adoption of the surface loading rate

$$L_s = 220\,kgBOD_5/ha.d\ (\text{adopted} - \text{see Section 13.5.a})$$

c) Calculation of the required area

$$A = \frac{L}{L_s} = \frac{1{,}050\,kg/d}{220\,kg/ha.d} = 4.8\,ha = 48{,}000\,m^2$$

d) Adoption of a value for the pond depth

$$H = 1.80\,m\ (\text{adopted})$$

Example 13.3 (_Continued_)

e) Calculation of the resulting volume

$$V = A.H = 48{,}000\,m^2 \times 1.80\,m = 86{,}400\,m^3$$

f) Calculation of the resulting detention time

$$t = \frac{V}{Q} = \frac{86{,}400\,m^3}{3{,}000\,m^3/d} = 28.8\ d$$

g) Adoption of a value for the BOD removal coefficient (K)

- Complete-mix regime, at 20 °C (see Section 13.6.3):

$$K = 0.35 d^{-1}$$

- Correction for the temperature of 23 °C:

Adopting a value for the temperature coefficient $\theta = 1.05$:

$$K_T = K_{20}.\theta^{(T-20)} = 0.35 \times 1.05^{(23-20)} = 0.41\ d^{-1}$$

h) Estimation of the effluent soluble BOD

Using the complete-mix model (considering a not predominantly longitudinal cell):

$$S = \frac{S_o}{1 + K.t} = \frac{350}{1 + 0.41 \times 28.8} = 27\,mg/L$$

Note: if the dispersed-flow model had been adopted, with the dimensions L, B and H determined in item m below, together with equations from Section 13.6 (Table 13.5, Equations 13.12, 13.13 or 13.14, $K = 0.15\ d^{-1}$ for 20 °C, $\theta = 1.035$), this would have lead to:

- $d = 0.35$ (according to Eq. 13.12) , $d = 0.37$ (according to Eq. 13.13) or $d = 0.40$ (according to Eq. 13.14)
- $S = 23$ mg/L

i) Estimation of the effluent particulate BOD

Assuming an effluent SS concentration equal to 80 mg/L, and considering that each 1 mgSS/L implies a BOD_5 of around 0.35 mg/L (see Section 13.6.2):

Particulate $BOD_5 = 0.35\ mgBOD_5/mgSS \times 80\ mgSS/L = 28\ mgBOD5/L$

It should be remembered that the particulate BOD is detected in the BOD test, but it may not be exerted in the receiving body, depending on the survival conditions of the algae.

Example 13.3 (*Continued*)

j) Total effluent BOD

Total effluent BOD = Soluble BOD + Particulate BOD
Total effluent BOD = 27 + 28 = 55 mg/L

l) Calculation of the BOD removal efficiency

$$E = \frac{S_o - S}{S_o}.100 = \frac{350 - 55}{350}.100 = 84\%$$

m) Dimensions of the pond

The dimensions of the pond are a function of the local area and topography. For the purposes of this example, unspecific values will be adopted.

If two ponds in parallel and a length/breadth (L/B) ratio equal to 2.5 in each pond are adopted, one has:

Area of 1 pond = 48,000/2 = 24,000 m^2

$A = L.B = [(L/B).B].B = [2.5.B].B = 2.5.B^2$

$24{,}000\ m^2 = 2.5.B^2 \rightarrow B = [A/\ (L/B)]^{0.5} = (24{,}000/2.5)^{0.5} = 98.0\ m$
$L = (L/B) \times B = 2.5.B = 2.5 \times 98.0\ m = 245.0\ m$

- Length: L = 245.0 m
- Breadth: B = 98.0 m

n) Total area required for the whole system

The total area required for the ponds, including the embankments, urbanisation, internal roads, laboratory, parking and others, is about 25% to 33% greater than the net area calculated at mid-depth (Arceivala, 1981). Hence:

$$A_{total} = 1.3.A_{net} = 1.3 \times 48{,}000\,m^2 \cong 62{,}400\,m^2 (6.2\,ha)$$

$$\text{Per capita land requirements} = \frac{62{,}400\ m^2}{20{,}000\ hab} = 3.1\ m^2\ /inhab.$$

o) Sludge accumulation

$$\text{Accumulation per year} = 0.05m^3/inhab. \times 20{,}000\ inhab. = 1{,}000\ m^3/year$$

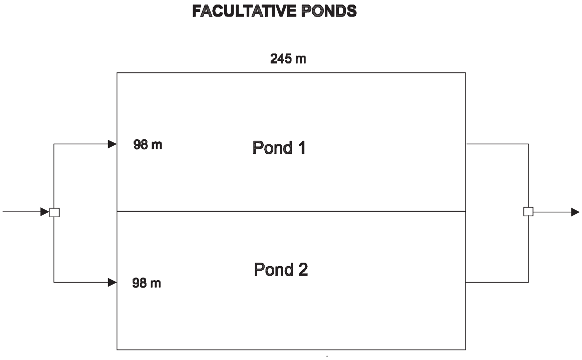
Thickness in 1 year:

$$\text{Thickness} = \frac{1{,}000\ m^3/year \times 1\ year}{48{,}000\,m^2} = 0.021\,m/year = 2.1\,cm/year$$

Example 13.3 **(*Continued*)**

Thickness in 20 years of operation:
Thickness: 2.1 cm/year × 20 years = 42 cm in 20 years
After 20 years of operation, the sludge occupies only 23% (= 0.42 m/1.80 m) of the liquid depth of the pond.

p) Layout of the system



14

System of anaerobic ponds followed by facultative ponds

14.1 INTRODUCTION

Anaerobic ponds constitute an alternative form of treatment, in which the existence of *strictly anaerobic* conditions is essential. This is reached through the application of a high BOD load per unit of volume of the pond, which causes the oxygen consumption rate to be several times greater than the oxygen production rate. In the oxygen balance, the production by photosynthesis and atmospheric reaeration are, in this case, negligible.

Anaerobic ponds have been used for the treatment of domestic sewage and organic industrial wastewaters, with high BOD concentrations, such as slaughterhouses, piggery wastes, dairies, beverage industries, etc.

The conversion of organic matter under anaerobic conditions is slow, owing to the slow growth rate of anaerobic bacteria. This results from the fact that the anaerobic reactions generate less energy than the aerobic reactions for the stabilisation of organic matter. The temperature of the medium has a great influence in the biomass reproduction and substrate conversion rates, which makes warm-climate regions to be favourable for the utilisation of this type of pond.

Anaerobic ponds are usually deep, of the order of 3 m to 5 m. The depth is important, in order to reduce the possibility of the penetration of the oxygen produced in the surface to the other layers. Because these ponds are deeper, the land requirements are correspondingly small.

Biological Wastewater Treatment in Warm Climate Regions by Marcos von Sperling and Carlos Augusto de Lemos Chernicharo. ISBN: 1 84339 002 7. Published by IWA Publishing, London, UK.

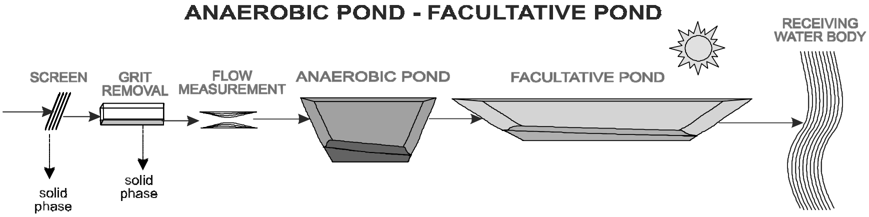


Figure 14.1. System of anaerobic ponds followed by facultative ponds

Anaerobic ponds do not require any special equipment and have a practically negligible energy consumption (for a possible pumping of the raw sewage or the recirculation of the final effluent).

The BOD removal efficiency in anaerobic ponds is usually of the order of 50% to 70%. The effluent BOD is still high and implies the need of a post-treatment unit. The most widely used post-treatment units are facultative ponds, composing the system of anaerobic ponds followed by facultative ponds (Fig. 14.1).

The removal of BOD in the anaerobic pond provides a substantial saving in the area required for the facultative pond, making the total land requirement (anaerobic + facultative ponds) to be around 45% to 70% of the requirement for a primary facultative pond (receiving raw wastewater).

The existence of an anaerobic stage in an open reactor is always a matter of concern, owing to the possibility of the generation of bad odours. If the system is well balanced, the generation of bad smell should not be important, but occasional operational problems can lead to the release of hydrogen sulphide (H_2S), responsible for obnoxious odours. If the sulphate concentration in the influent is lower than 300 mg/L, the production of sulphide should not be problematic (in anaerobic conditions, sulphate is reduced to sulphide). Additionally, if the pH in the pond is close to neutrality, most of the sulphide will be present in the form of the bisulphide ion (HS^-), which is odourless (Mara et al, 1997). Wastewaters with low pH values (industrial effluents or wastewater originated from a water that is soft, with low alkalinity, high acidity or without pH correction) may induce odour problems. As a result of the points above, the anaerobic-facultative ponds system should be located far away from houses (during all the operational life of the ponds).

14.2 DESCRIPTION OF THE PROCESS

In a simplified way, the anaerobic conversion takes place in two stages (for more details, see Chapter 9):

- liquefaction and formation of acids (through the acid-forming bacteria, or acidogenic bacteria)
- formation of methane (through the methane-forming organisms, or methanogenic archaea)

In the first phase, there is no BOD removal, just the conversion of the organic matter to other forms (simpler molecules and then acids). It is in the second stage that BOD is removed, with the organic matter (acids produced in the first stage) being converted mainly to methane and carbon dioxide. The carbon is removed from the liquid medium by the fact that the methane (CH_4) escapes to the atmosphere.

The methane-forming organisms are very sensitive to the environmental conditions. If their reproduction rate is reduced, there will be the accumulation of the acids formed in the first stage, with the following consequences: (a) interruption of the BOD removal process and (b) generation of bad odours, because the acids are very fetid.

Therefore, it is essential that the appropriate balance between the two communities is guaranteed, ensuring the completion of both stages. For the adequate development of the methane-forming archaea, the following conditions should be met:

- absence of dissolved oxygen (methane-forming archaea are *strict anaerobes* and do not survive in the presence of dissolved oxygen)
- adequate temperature of the liquid (above 15 °C)
- adequate pH (close to or above 7)

The anaerobic activity affects the nature of the solids, in such a way that, in the facultative pond, the solids are less prone to fermentation and flotation, besides decomposing more easily.

14.3 DESIGN CRITERIA FOR ANAEROBIC PONDS

The main design parameters for anaerobic ponds are:

- *Volumetric organic loading rate*
- *Detention time*
- *Depth*
- *Geometry (length / breadth ratio)*

The criterion of the *volumetric organic loading rate* is the most important, and is established as a function of the need of a certain pond volume for the conversion of the applied BOD load. The criterion of the *detention time* is based on the time necessary for the reproduction of the anaerobic bacteria.

a) Volumetric organic loading rate

The volumetric loading rate L_V, the main design parameter for anaerobic ponds, is a function of the temperature. Warmer locations allow a larger loading rate (smaller pond volume). The consideration of the volumetric load is important, because industrial wastewaters can vary widely in the relationship between flow and BOD concentration (load = flow × concentration). Therefore, only the detention time criterion is insufficient.

Table 14.1. Permissible volumetric loading rates for the design of anaerobic ponds as a function of temperature

Mean air temperature in the coldest month – T (°C)	Permissible volumetric loading rate L_V (kgBOD/m^3.d)
10 to 20	$0.02T - 0.10$
20 to 25	$0.01T + 0.10$
> 25	0.35

Source: adapted from Mara (1997)

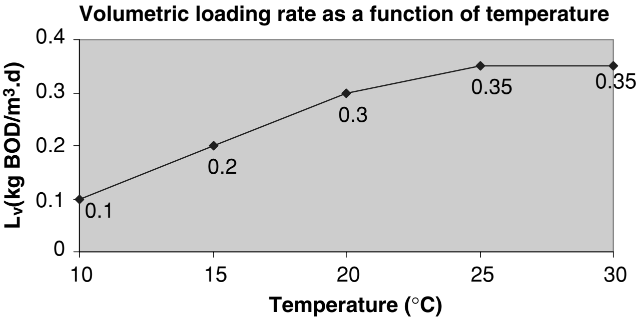


Figure 14.2. Relation between the permissible volumetric loading rates in anaerobic ponds and the temperature, according to the criteria of Mara (Table 14.1)

Values of volumetric loading rates usually adopted are within the following range:

$$\boxed{L_v = 0.1 \text{ to } 0.3 \text{ kgBOD}_5/\text{m}^3.\text{d}}$$

The upper limit aims at avoiding organic overloading in the anaerobic pond. The lower limit is to avoid that the pond receives a very low organic load, which could give conditions that, under some circumstances, the pond could behave as a facultative pond. This would be harmful to the strictly anaerobic methanogenic archaea.

Mara (1997) proposes the relation between the volumetric loading rates and the temperature presented in Table 14.1 and in Figure 14.2. The values represent maximum permissible rates, and the designer may decide to incorporate more safety by the adoption of lower values of the loading rate.

The volume required is given by:

$$\boxed{V = L/L_v} \tag{14.1}$$

where:

V = volume required for the pond (m^3)

L = total (soluble + particulate) influent BOD load (kgBOD$_5$/d)

L_v = volumetric loading rate (kgBOD$_5$/m^3.d)

For domestic sewage, the final volume to be adopted for the anaerobic pond is a compromise between the two criteria (detention time and volumetric rate), aiming, as much as possible, to satisfy both. For industrial effluents, the defining criterion is the volumetric loading rate.

In situations in which there is a great variation of the influent load, for example, between the beginning and the end of the design horizon, it is important to verify compliance with the design criteria from the start of the operation. If the initial influent load is low, it may be advisable to divide the implementation into two or more anaerobic ponds in parallel, with only one or some ponds being implemented in the first stage. This assists in guaranteeing that the ponds work under really anaerobic conditions, avoiding very low loading rates.

b) Detention time

For *domestic sewage*, the hydraulic detention time is usually within the following range:

$$\boxed{t = 3.0\ \text{d to } 6.0\ \text{d}}$$

In conventional anaerobic ponds (in which the inlet pipe is above the sludge layer), if the detention time is lower than 3.0 days, the methane-forming organisms may be washed out of the reactor. In these conditions, the maintenance of a stable bacterial population would not be possible. Apart from the efficiency of the anaerobic pond being reduced, the more serious aspect of imbalance between the acid-forming and methane-forming stages would occur. The consequences would be the accumulation of acids in the liquid, with the generation of bad odours, as a result of the small population of methane-forming organisms to continue the conversion of acids.

However, there is a recent tendency of decreasing the detention times in anaerobic ponds to around **2 days** and, possibly, 1 day. For this, it is necessary to increase the retention time of the biomass and to allow an intimate biomass–wastewater contact. These conditions can be obtained with the distribution of the influent in the bottom of the pond, at several points, aiming at approaching the working principle of an upflow anaerobic sludge blanket reactor. When entering the pond, the influent sewage has direct contact with the anaerobic biomass, optimising the important aspect of the organic matter – biomass contact. Traditional anaerobic ponds that presented operational problems showed an improvement in the performance and a reduction of odour generation with the simple change of the inlet pipe to the bottom of the pond.

With detention times greater than 6 days, the anaerobic pond can behave occasionally as a facultative pond. This is undesirable, because the presence of oxygen is fatal for the methane-forming organisms. *Anaerobic ponds must work as strict anaerobic ponds and cannot alternate between anaerobic, facultative and aerobic conditions*.

After calculating the volume based on the volumetric loading rate (L_v), the resulting detention time is obtained by:

$$\boxed{t = V/Q} \tag{14.2}$$

where:

t = detention time (d)
V = volume of the pond (m^3)
Q = average influent flow (m^3/d)

c) Depth

The depth of anaerobic ponds is high, in order to guarantee the predominance of anaerobic conditions, avoiding the pond to work as a facultative pond. In fact, the deeper the pond, the better. However, deep excavations tend to be more expensive. Values usually adopted are in the range of:

$$\boxed{H = 3.5 \text{ m to } 5.0 \text{ m}}$$

When there is no previous grit removal, the anaerobic pond could have an additional depth of at least 0.5 m, close to the inlet and extending to at least 25% of the area of the pond. However, it is believed that the inclusion of grit chamber units is beneficial, because they minimise problems of grit accumulation close to the inlet pipe and due to their simplicity.

d) Geometry (length / breadth ratio)

Anaerobic ponds are square or slightly rectangular, with typical length/breadth (L/B) ratios of:

$$\boxed{\text{Length / breadth ratio (L/B)} = 1 \text{ to } 3}$$

14.4 ESTIMATION OF THE EFFLUENT BOD CONCENTRATION FROM THE ANAEROBIC POND

There are still no conceptual mathematical models in widespread use that allow an estimation of the effluent BOD concentration from anaerobic ponds. For this reason, these ponds have been designed mainly according to empirical criteria. Mara (1997) proposed the BOD removal efficiencies as a function of the temperature presented in Table 14.2 and illustrated in Figure 14.3.

Table 14.2. BOD removal efficiencies in anaerobic ponds as a function of the temperature

Mean air temperature of the coldest month - T (°C)	BOD removal efficiency E (%)
10 to 25	2T + 20
> 25	70

Source: Mara (1997)

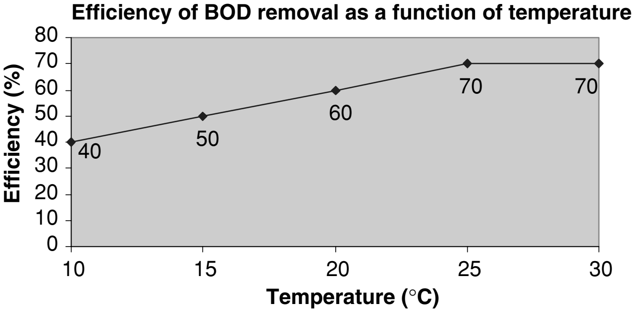


Figure 14.3. Relationship between the BOD removal efficiency in anaerobic ponds and the temperature, according to Mara's criterion (Table 14.2)

Once the removal efficiency (E) has been estimated, the effluent concentration (BOD_{effl}) of the anaerobic pond is calculated using the formulas:

$$\boxed{E = (S_o - BOD_{effl}).100/S_o} \tag{14.3}$$

or

$$\boxed{BOD_{effl} = (1 - E/100).S_o} \tag{14.4}$$

where:

S_o = influent total BOD concentration (mg/L)
BOD_{effl} = effluent total BOD concentration (mg/L)

In this empirical approach, the effluent BOD considered is the total BOD, different from the calculations of facultative ponds, in which the effluent BOD is split in terms of soluble BOD and particulate BOD.

14.5 DESIGN OF FACULTATIVE PONDS FOLLOWING ANAEROBIC PONDS

Secondary facultative ponds can be designed following the same surface loading rates described in Chapter 13. The resulting detention time will be now smaller, owing to the previous removal of the BOD in the anaerobic pond.

For the design according to the surface loading rate, the BOD concentration and load at the *influent to the facultative pond* are the *effluent from the anaerobic pond*. There are some evidences to suggest that the surface loading rate in secondary facultative ponds could be somewhat higher than those adopted for primary ponds. However, for design purposes, it is better to consider both as being equal for safety reasons (Mara et al, 1992).

In secondary facultative ponds there is more flexibility with regards to the geometry of the pond, which could have higher L/B ratios, since the overloading problems in the inlet zone should be smaller due to the previous removal of a large part of the BOD in the anaerobic pond.

The estimation of the effluent BOD concentration from the facultative pond can be done according to the methodology described in Section 13.6. The removal coefficient K will be in this case lower than in primary facultative ponds, due to the previous removal of the more easily degradable organic matter in the anaerobic pond. The remainder of the organic matter is harder to degrade, implying slower conversion rates. In Section 13.6.3, the following values of K have been suggested for secondary facultative ponds, using the complete-mix model:

$$\boxed{K = 0.25 \text{ to } 0.32 \text{ d}^{-1}}$$

(20 °C, secondary facultative ponds, complete-mix model)

14.6 SLUDGE ACCUMULATION IN ANAEROBIC PONDS

The considerations here are similar to those made in the case of the facultative ponds (Section 13.8). The accumulation rate is in the order of 0.03 to 0.10 m^3/inhab.year (Mendonça, 1990; Gonçalves, 2000), and the lower range is more usual in warm-climate areas. Other data available for accumulation rates are 2 to 8 cm/year (Silva, 1993; CETESB, 1989; Gonçalves, 2000). These values of yearly increases in the thickness of the sludge layer correspond to accumulation rates lower than 0.03 m^3/inhab.year.

The aspects of sludge management in anaerobic ponds are different from facultative ponds. In the latter, the system can operate for several years, eventually during all of the design period, without needing to remove sludge (provided there is a good grit removal in the preliminary treatment). However, because of the smaller volume of the anaerobic ponds, the sludge accumulation manifests itself more rapidly, bringing about the need of an appropriate planning related to the sludge management (see Chapter 22). The anaerobic ponds should be cleaned according to one of the following strategies:

- when the sludge layer reaches approximately 1/3 of the liquid depth
- annual removal of a certain volume, in a pre-determined month, to include the cleaning stage in a systematic way in the operational strategy of the pond

If the removal is not by emptying and drying inside the pond, the whole sludge mass should not be removed, since this would lead to a total loss of the biomass, requiring the anaerobic pond to start up again.

Example 14.1

Design an anaerobic – facultative pond system using the same data from Example 13.3:

- Population = 20,000 inhabitants
- Influent flow = 3,000 m^3/d
- Influent BOD: $S_o = 350$ mg/L
- Temperature: $T = 23$ °C (mean liquid temperature in the coldest month)

Solution:

a) Influent BOD load

From Example 13.3:

$$L = 1{,}050\,\text{kgBOD}_5/\text{d}$$

Design of the anaerobic pond

b) Adoption of a value for the volumetric loading rate L_v

$$L_v = 0.15\,\text{kgBOD/m}^3.\text{d}$$

This is a conservative value (see Section 14.3.a). However, higher values would lead to a smaller pond volume and, as a result, to low detention times (see section d below).

c) Calculation of the required volume

$$\text{volume} = \frac{\text{load}}{\text{volumetric load}} \longrightarrow V = \frac{L}{L_v} = \frac{1{,}050\ \text{kgBOD/d}}{0.15\ \text{kgBOD/m}^3.\text{d}} = 7{,}000\,\text{m}^3$$

d) Verification of the detention time

$$t = \frac{V}{Q} = \frac{7{,}000\ \text{m}^3}{3{,}000\ \text{m}^3/\text{d}} = 2.3\ \text{d} \quad \text{OK!}$$

Ponds with such a low detention time should have the inlet at the bottom, in contact with the settled sludge.

Example 14.1 (Continued)

e) Determination of the required area and dimensions

Depth H = 4.5 m (adopted)

$$\text{area} = \frac{\text{volume}}{\text{depth}} \longrightarrow A = \frac{V}{H} = \frac{7{,}000\,\text{m}^3}{4.5\,\text{m}} = 1{,}556\,\text{m}^2$$

Adopt 2 ponds
Area of each pond: $1{,}556\ \text{m}^2/2 = 778\,\text{m}^2$
Possible dimensions of each pond: $34\ \text{m} \times 23\,\text{m}$

f) Concentration of effluent BOD

BOD removal efficiency: E = 60% (see Section 14.4)

$$\text{BOD}_{\text{effl}} = (1 - E/100).S_o = (1 - 60/100) \times 350 = 0.4 \times 350 = 140\,\text{mg/L}$$

The effluent from the anaerobic pond is the influent to the facultative pond.

g) Sludge accumulation in the anaerobic pond

Adopting an accumulation rate of 0.04 m^3/inhab.year (see Section 14.6):

$$\text{Annual accumulation} = 0.04\ \text{m}^3/\text{inhab.year} \times 20{,}000\ \text{inhab} = 800\ \text{m}^3/\text{year}$$

Thickness of the sludge layer in 1 year:

$$\text{thickness} = \frac{\text{Annual accumulation} \times \text{time}}{\text{pond area}} = \frac{800\ \text{m}^3/\text{year} \times 1\ \text{year}}{1{,}556\ \text{m}^2}$$
$$= 0.51\ \text{m/year} = 51\text{cm/year}$$

This annual accumulation rate, expressed in cm/year, is greater than the values mentioned in Section 14.6, probably because the pond in the present example is deep and has a small detention time (smaller surface area for the sludge to spread itself).
Time to reach 1/3 of the pond depth:

$$\text{time} = \frac{H/3}{\text{yearly thickness}} = \frac{4.5\ \text{m} / 3}{0.51\ \text{m} / \text{year}} = 2.9\ \text{year}$$

The sludge volume accumulated during this period corresponds to 1/3 of the net pond volume, that is, $7{,}000\ \text{m}^3/3 = 2{,}333\ \text{m}^3$ of sludge.

The sludge should be removed approximately every 3 years (volume of 2,333 m^3) or, annually (removal of 800 m^3).

Design of the facultative pond

h) Influent load to the facultative pond

The effluent load from the anaerobic pond is the influent load to the facultative pond. With the removal efficiency of 60% in the anaerobic pond, the

Example 14.1 (*Continued*)

influent load to the facultative pond is:

$$L = \frac{(100 - E).L_o}{100} = \frac{(100 - 60) \times 1{,}050}{100} = 420\,\text{kg BOD/d}$$

i) Adoption of the surface loading rate

$L_s = 220$ kgBOD/ha.d (equal to the value adopted in Example 13.3)

j) Required area

$$A = \frac{L}{L_s} = \frac{420\ \text{kgBOD/d}}{220\ \text{kgBOD/ha.d}} = 1.9\,\text{ha}\ (19{,}000\,\text{m}^2)$$

Adopt two ponds
Area of each pond: $19{,}000\ \text{m}^2/2 = 9{,}500\ \text{m}^2$
Possible dimensions of each pond: L = 155 m and B = 62 m (L/B ratio = 2.5)

k) Adoption of a value for the depth

H = 1.80 m (adopted)

l) Calculation of the resulting volume

$V = A.H = 19{,}000\,\text{m}^2 \times 1.80\,\text{m} = 34{,}200\,\text{m}^3$

m) Calculation of the resulting detention time

$$t = \frac{V}{Q} = \frac{34{,}200\,\text{m}^3}{3{,}000\,\text{m}^3/\text{d}} = 11.4\,\text{d}$$

n) Adoption of a value for the BOD removal coefficient (K)

- Complete-mix regime, at 20°C: $K = 0.27\ \text{d}^{-1}$ (adopted – see Section 14.5)
- Correction for the temperature of 23°C:

$K_T = K_{20}.\theta^{(T-20)} = 0.25 \times 1.05^{(23-20)} = 0.31\ \text{d}^{-1}$

o) Estimation of the effluent soluble BOD

Using the complete-mix model, since the pond is not predominantly longitudinal (length/breadth ratio of 2.5):

$$S = \frac{S_o}{1 + K.t} = \frac{140}{1 + 0.31 \times 11.4} = 31\ \text{mg/L}$$

p) Estimation of the effluent particulate BOD

Assuming an effluent SS concentration equal to 80 mg/L, and considering that each 1 mgSS/L leads to a BOD_5 of around 0.35 mg/L (see Section 13.6.2):

$$\text{particulate BOD}_5 = 0.35\,\text{mgBOD}_5/\text{mgSS} \times 80\,\text{mgBOD}_5/\text{L} = 28\,\text{mgBOD}_5/\text{L}$$

Example 14.1 (*Continued*)

It should be remembered that the particulate BOD is detected in the BOD test, but it may not be exerted in the receiving body, depending on the survival conditions of the algae.

q) Total effluent BOD

total effluent BOD = soluble BOD + particulate BOD
Total effluent BOD = 31 + 28 = 59 mg/L

r) Calculation of the total BOD removal efficiency of the anaerobic–facultative pond system

$$E = \frac{(S_o - BOD_{effl})}{S_o}.100 = \frac{350 - 59}{350} \times 100 = 83\%$$

s) Total net area (anaerobic + facultative pond)

$$\text{Total net area} = 0.16\ \text{ha} + 1.9\ \text{ha} = 2.1\ \text{ha}$$

t) Total area required

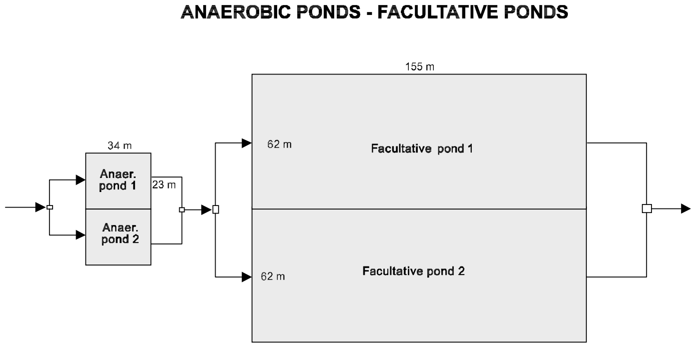
The total area is in the order of 25% to 33% greater than the required net area. Thus, the total area occupied by the system of ponds and auxiliary structures is approximately:

$$\text{Total area} = 1.3 \times 2.1 = 2.7\ \text{ha}$$

With primary facultative ponds (Example 13.3), the total area required is 6.2 ha. Therefore, there is a substantial economy of area (56%). The total detention time in the present example is 13.7 d (= 2.3 + 11.4), much lower than that for a primary facultative pond (28.8 m).

It should be remembered that these land requirements are applicable to the current example, which is associated to a relatively high temperature of the liquid, which allows high loading rates and removal efficiencies. In applications in colder places, the required area will be naturally larger.

u) Layout of the system



15

Facultative aerated lagoons

15.1 INTRODUCTION

Facultative aerated lagoons (Figure 15.1) are used when it is desired to have a predominantly aerobic system, more compact than facultative ponds or anaerobic-facultative ponds. The main difference with relation to the conventional facultative pond regards the form of oxygen supply. While in facultative ponds the oxygen is obtained from algal photosynthesis, in the case of facultative aerated lagoons the oxygen is supplied by *aerators.*

Because of the introduction of mechanisation, aerated lagoons are less simple in terms of maintenance and operation, compared with conventional facultative ponds. The reduction of the land requirements is therefore obtained with a certain increase in the operational level, besides the introduction of energy consumption.

Overloaded conventional facultative ponds without area for expansion can be converted to facultative aerated lagoons by the inclusion of aerators. However, it is interesting to foresee this possibility in the design period itself, as part of the staging of the plant, so that a pond depth compatible with the future aeration equipment is selected and that concrete protecting plates can be placed at the bottom of the pond, underneath the future aerators.

15.2 DESCRIPTION OF THE PROCESS

The pond is denominated facultative because the level of energy introduced by the aerators is sufficient only for oxygenation, but not to maintain the solids (biomass and raw sewage suspended solids) dispersed in the liquid mass. Consequently, the

Biological Wastewater Treatment in Warm Climate Regions by Marcos von Sperling and Carlos Augusto de Lemos Chernicharo. ISBN: 1 84339 002 7. Published by IWA Publishing, London, UK.

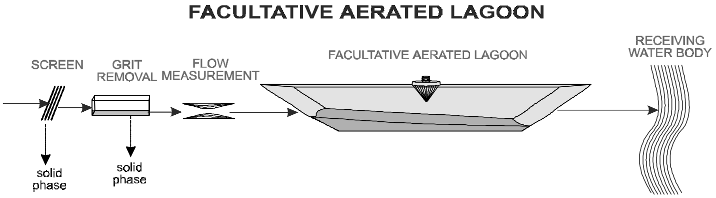


Figure 15.1. Facultative aerated lagoon system

solids tend to settle and to form a bottom sludge layer, which is decomposed anaerobically. Only the soluble BOD and the BOD represented by finely particulated solids remain in the liquid mass, undergoing aerobic decomposition. Therefore, in terms of the distribution of the heterotrophic biomass, the pond behaves as a conventional facultative pond.

The mechanical aerators more commonly used in the aerated lagoons are high-speed vertical-shaft units (see Chapter 11). A greater introduction of oxygen is obtained compared to the conventional facultative ponds, allowing a faster decomposition of the organic matter. Consequently, the hydraulic detention time in the pond can be smaller (of the order of 5 to 10 days), that is, the land requirement is lower.

15.3 DESIGN CRITERIA

The design of facultative aerated lagoons is similar to that of facultative ponds with respect to the kinetics of BOD removal. There are no requirements in terms of surface area (surface loading rates), due to the fact that the process is independent from photosynthesis. Some design criteria are specific for the aeration system, and are described in Sections 15.5 and 15.6.

The following criteria should be considered:

- *detention time*
- *depth*

a) Detention time

The detention time should be adopted in order to allow a satisfactory removal of BOD, according to the kinetics described in Section 15.4a. Usually, the values adopted vary in the following range:

$$\boxed{t = 5 \text{ to } 10\ \text{d}}$$

b) Depth

The depth of the pond should be selected in order to satisfy the following criteria:

- compatibility with the aeration system
- need of an aerobic layer of approximately 2 m to oxidise the gases from the anaerobic decomposition of the bottom sludge

Usually, the depth varies in the range of:

$$\boxed{H = 2.5 \text{ to } 4.0 \text{ m}}$$

15.4 ESTIMATION OF THE EFFLUENT BOD CONCENTRATION

The estimation of the effluent BOD concentration follows a similar procedure to that used for the facultative ponds (Section 13.6). The influence of the hydraulic regime of the pond should also be taken into consideration, although the formulas corresponding to the complete-mix regime are adopted in most designs. The formulas for the estimation of the dispersion number d, presented in Section 13.6, should not be adopted here, since they are specific for unaerated facultative ponds.

Similarly to the facultative ponds, the effluent from facultative aerated lagoons is constituted of *dissolved organic matter* (**soluble BOD**) and *suspended organic matter* (**particulate BOD**). However, the latter is *not* anymore associated predominantly to algae.

$$\boxed{BOD_{tot} = BOD_{sol} + BOD_{part}} \tag{15.1}$$

where:

BOD_{tot} = total BOD_5 of the effluent (mg/L)
BOD_{sol} = soluble BOD_5 of the effluent (mg/L)
BOD_{part} = particulate (suspended) BOD_5 of the effluent (mg/L)

The *suspended* organic matter is represented mainly by the *bacteria* responsible for the stabilisation of the organic matter. In spite of the fact that facultative aerated lagoons allow the sedimentation of solids, not all of them settle. A large part of the bacterial protoplasm is constituted of organic matter, which exerts an oxygen demand on the receiving body and in the BOD test. In the case of unaerated facultative ponds, the solids in the effluent consist mainly of algae, which may even lead to the production of oxygen in the receiving body. For this reason, the BOD of the effluent from *unaerated facultative ponds* is considered in the European legislation as being mainly the soluble BOD. However, in the case of *aerated lagoons* (and all other wastewater treatment processes, with the exception of unaerated facultative ponds), the BOD related to the organic fraction of the suspended solids (particulate BOD) should be considered.

a) Soluble effluent BOD

The estimation of the soluble effluent BOD is accomplished using the same formulas presented for facultative ponds, which are a function of the hydraulic regime assumed for the reactor (Section 13.6).

The value of the BOD removal coefficient K is higher in the case of facultative aerated lagoons. Typical values for the complete-mix regime are in the range of (Arceivala, 1981):

$$\boxed{K = 0.6 \text{ to } 0.8\ d^{-1}}$$

This value is for the liquid temperature of 20 °C. For other temperatures, Equation 13.3 can be used, with $\theta = 1.035$.

With relation to the value of S_o to be used in the equations in Table 13.4 and for the design of the aeration system, the following aspects should be taken into consideration. Facultative aerated lagoons allow the sedimentation of the particulate organic matter of the raw sewage, which undergoes *anaerobic* decomposition in the bottom sludge. The influent BOD value (S_o) available for *aerobic* stabilisation is, therefore, lower than the total value in the raw sewage. The value of S_o to be adopted in the calculations depends on the anaerobic activity, which is a function of the temperature in the liquid. Consequently, the following two conditions can happen regarding the organic matter in the bottom sludge (Arceivala, 1981):

- *Anaerobic decay with hydrolysis and acidification, but without methanogenesis*

  S_o = 100% of total influent BOD

  Climate: cold
  Comment: there are regions with cold periods in which the methanogenic stage (responsible for the removal of BOD) does not fully occurs, implying the release of intermediate by-products of the digestion, which exert an oxygen demand in the aerobic layer. Hence, the BOD to require aerobic stabilisation can be considered as being equal to S_o.
- *Anaerobic decay with hydrolysis, acidification, and methanogenesis*

  S_o = 40% to 70% of total influent BOD

  Climate: warm
  Comment: under conditions in which the liquid temperature is sufficiently high (>15 °C), the anaerobic conversion is complete, including all the stages. Because a fraction of the BOD is stabilised anaerobically in the bottom, the value of S_o considered for the estimation of the effluent BOD and the oxygen requirements is only a portion of the influent BOD (around 40 to 70%).

However, for *design* purposes, the BOD load to be aerobically stabilised can be considered, for safety reasons, as being equal to the total influent load (S_o = BOD of the influent):

$$\boxed{\textit{Design}\text{: } S_o = 100\% \text{ of total (soluble + particulate) } BOD_5 \text{ of the influent}}$$

b) Particulate effluent BOD

To calculate the particulate effluent BOD from a facultative aerated lagoon it is necessary to estimate the concentration of suspended solids in the effluent from the pond, since the particulate BOD is caused exactly by the suspended solids.

The amount of solids that stay in suspension in the liquid medium is a function of the turbulence level introduced by the aerators. This turbulence level, or mixing capacity, is evaluated through the concept of the **power level**. The power level represents the *energy introduced by the aerators per unit volume of the reactor*, being obtained from the formula (see also Section 11.6.3):

$$\phi = P/V \tag{15.2}$$

where:

ϕ = power level (W/m^3)
P = power for aeration (W)
V = reactor volume (m^3)

The greater the power level, the greater the quantity of suspended solids that can remain dispersed in the liquid medium (Table 15.1). The values presented in the table are only estimates, since the mixing intensity also depends on the number and distribution of aerators (in the case of mechanical aeration) and on the size and geometry of the pond.

Table 15.1. Suspended solids concentrations that can be maintained dispersed in the liquid as a function of the power level

Power level (W/m^3)	SS (mg/L)
0.75	50
1.75	175
2.75	300

Source: Eckenfelder (1979)

Facultative aerated lagoons work with low power levels, since one of their objectives is exactly to facilitate the sedimentation of the solids. The values of the power level of facultative aerated lagoons are in the range of:

$$\boxed{\text{Power level: } \phi = 0.75 \text{ to } 1.50\ \text{W/m}^3}$$

As a result, the SS concentrations in the lagoon effluent would be in the range of 50 to 140 mg/L. However, the outlet zone of the lagoon may be left without aerators, in order to improve the settling conditions and, therefore, the effluent quality. Usual SS values in the final effluent may then be in the approximate range of:

$$\boxed{\text{SS effluent: 50 to 100 mg/L}}$$

Once the SS concentration in the effluent has been estimated, the calculation of the expected value for the effluent particulate BOD can be undertaken, using the following relationship:

$$\boxed{\text{BOD}_{\text{part}} = 0.3 \text{ to } 0.4 \text{ mgBOD}_5/\text{mgSS}}$$

Thus, each 1 mg/L of SS produces a particulate BOD between 0.3 and 0.4 mg/L. Knowing the total concentration of effluent SS, the particulate effluent BOD is readily estimated.

15.5 OXYGEN REQUIREMENTS

The amount of oxygen to be supplied by the aerators for the aerobic stabilisation of the organic matter should usually be equal to the total ultimate influent BOD. The ultimate BOD (BOD_u) corresponds to the total oxygen demand exerted for the complete stabilisation of the organic matter (see Chapter 3). In typical domestic sewage, BOD_u is reached at the end of a long period, in the order of 20 days. BOD_u is therefore higher than the BOD_5, since the latter is exerted only until the fifth day. The ratio BOD_u/BOD_5 is frequently adopted in the range of 1.2 and 1.5.

The considerations made in Section 15.4a are also valid here. Thus, the oxygen demand can be admitted, for *design purposes*, as being due to all the influent BOD, without any reduction related to the anaerobic conversion in the bottom.

In the computation of the oxygen requirements, the following items can be *discounted*:

- Fraction of non-stabilised BOD (S) leaving with the effluent. This is due to the fact that the efficiency of the system in the removal of BOD is lower than 100%. See Section 15.4a. for the estimation of S.
- Fraction of BOD (oxygen consumption) not exerted by the solids leaving with the effluent. This corresponds to the particulate BOD (converted to ultimate BOD), covered in Section 15.4.

Considering these aspects, the amount of oxygen to be supplied can be adopted as:

$$\boxed{\text{OR} = \text{a.Q.}\ (\text{S}_\text{o} - \text{S})/1000} \tag{15.3}$$

where:

OR = oxygen requirement (kgO_2/d)
a = coefficient, varying from 0.8 to 1.2 $kgO_2/kgBOD_5$
Q = influent flow (m^3/d)
S_o = total (*soluble* + *particulate*) influent BOD_5 concentration (g/m^3)
S = *soluble* effluent BOD_5 concentration (g/m^3)
1000 = conversion from kg to g (g/kg)

15.6 AERATION SYSTEM

The aerators more frequently used for aerated lagoons are the *mechanical vertical-shaft high-speed floating aerators.* Aerators with aspirating devices have also been used. See Chapter 11 for further information on aeration systems.

Both systems require simple maintenance. Installation is also simple, without the need of walkways and supporting columns. If needed, the position of the aerators in the pond can be easily changed. The floating units also adapt themselves to the water level variations in the lagoon, which can be controlled by the outlet weir.

The following aspects should be taken into consideration:

- The aerators should be distributed homogeneously in the aerated zone of the lagoon.
- If the lagoons are predominantly rectangular, a larger number of aerators or more powerful aerators can be placed close to the inlet zone, where the oxygen demand is higher.
- Adjacent aerators should have opposite rotation directions, that is, one should be clockwise and the other anti-clockwise.
- If lower effluent SS concentrations are desired, the final area of the lagoon can be without aerators, in order to provide better settling conditions.
- In small ponds, there should be a minimum of two aerators.
- The manufacturers' data should be consulted with relation to the recommended lagoon depth, influence zone of each aerator, oxygenation efficiency, etc.

There are two types of the area of influence of a mechanical aerator (Figure 15.2):

- *Mixing zone*. Area in which mixing of the liquid is guaranteed, allowing the maintenance of solids in suspension. Area with a smaller diameter, having the aerator in the centre.
- *Oxygenation zone*. Area in which the diffusion of oxygen in the liquid is guaranteed, but not the mixing. Area with a larger diameter, encircling the mixing zone.

Table 15.2 presents approximate values for the operating ranges of mechanical aerators as a function of their power. As can be observed, the area of influence of each aerator for oxygenation is much higher than that for mixing.

15.7 POWER REQUIREMENTS

The required power is calculated based on the oxygen requirements (OR), determined in Section 15.5. The parameter that converts oxygen consumption into power is the *oxygenation efficiency (OE.)*, which is expressed in the units of kgO_2/kWh (see Section 11.6.1).

The manufacturers' data are usually expressed in *standard conditions*, to allow a common base for the comparison of the efficiencies. The standard conditions are for *20 °C, absence of dissolved oxygen, no salinity, sea level, clean water*. See Chapter 11 for further information regarding aeration.

Table 15.2. Usual operation ranges of high-speed aerators

Power (HP)	Normal operating depth (m)	Influence diameter (m)		Diameter of the anti erosion plate
		Oxygenation	Mixing	
5–10	2.0–3.6	45–50	14–16	2.6–3.4
15–25	3.0–4.3	60–80	19–24	3.4–4.8
30–50	3.8–5.2	85–100	27–32	4.8–6.0

Notes:

- Usual powers of aerators: 1; 2; 3; 5; 7.5; 10; 15; 20; 25; 30; 40 and 50 HP.
- There are high-speed aerators with greater powers, but they tend to be, overall, less efficient.
- The table presents the influence diameter (and not the radius)
- Anti-erosion plate: situated in the pond bottom, underneath the aerator
- Source: table made based on data presented by Crespo (1995)

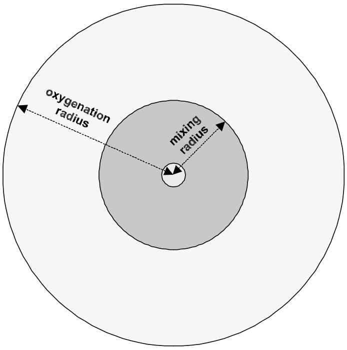


Figure 15.2. Mixing radius and oxygenation radius in a mechanical aerator

At *standard conditions*, the oxygenation efficiency of the aerators is within the range presented below. However, the manufacturers' data should always be consulted.

$$\boxed{\text{OE}_{\text{standard}} = 1.2 \text{ to } 2.0\ \text{kgO}_2/\text{kWh}}$$

Under real (field) operating conditions in the treatment plant, the oxygenation efficiency is smaller, being in the following range:

$$\boxed{\text{OE}_{\text{field}} = 0.55 \text{ to } 0.65\ \text{OE}_{\text{standard}}}$$

The power requirements are finally given by the following formula:

$$\boxed{P = \frac{OR}{24 \times \text{OE}_{\text{field}}}} \tag{15.4}$$

where:

P = power required (kW)

24 = conversion from days to hours (24 h/d)

The power of each aerator is then specified based on the manufacturers' data (or Table 15.2). For this, kW needs to be converted into HP (multiply kW by 1.34 to obtain HP).

15.8 SLUDGE ACCUMULATION

The sludge accumulation rate is in the order of 0.03 to 0.08 m^3/inhab.year (Arceivala, 1981). The sludge should be removed when the layer reaches a thickness that can be affected by the aerators, or when the net pond volume is substantially reduced (usually when the sludge reaches 1/3 of the pond depth). The inclusion of grit removal upstream of aerated lagoons is very important.

Example 15.1

Design a facultative aerated lagoon system using the same input data from Example 13.3:

- Population = 20,000 inhabitants
- Influent flow: $Q = 3{,}000\ m^3/d$
- Influent BOD: $S_o = 350$ mg/L
- Temperature: T = 23 °C (liquid)

Solution:

a) Detention time

t = 8 d (adopted)

b) Effluent soluble BOD

Assuming the complete-mix model and adopting the coefficient $K = 0.7\ d^{-1}$ for 20 °C, corrected for $0.8\ d^{-1}$ for 23°C:

$$\text{Soluble BOD}_5\text{: } S = \frac{S_o}{1 + K.t} = \frac{350}{1 + 0.8 \times 8} = 47\ \text{mg/L}$$

Lower values of S will be obtained if settling and anaerobic digestion of the influent particulate BOD are considered.

c) Estimation of the effluent particulate BOD

Assuming that the effluent contains 80 mg/L of suspended solids, the concentration of effluent particulate BOD_5 will be approximately:

$$\text{Particulate BOD}_5 = 0.35\ \text{mgBOD}_5/\text{mgSS} \times 80\ \text{mgSS/L} = 28\ \text{mgBOD}_5/\text{L}$$

Example 15.1 (Continued)

d) Total effluent BOD

Total BOD = soluble BOD + particulate BOD = 47 + 28 = 75 mg/L

To reduce the effluent BOD concentration, the detention time could be increased. However, this may not be economical, owing to the need of large volume increases for a small reduction in S. The configuration of the pond could also be changed, approaching a plug-flow reactor. Besides that, the settling conditions in the outlet zone could be improved by the exclusion of some aerators (already done in this example).

The efficiency of the system in the removal of BOD is:

$$E = \frac{S_o - S}{S_o} = \frac{350 - 75}{350} \times 100 = 79\%$$

e) Required volume

$$V = t.Q = 8\ d \times 3000\ m^3/d = 24{,}000\ m^3$$

f) Required area

Adopting a depth H = 3.5 m:

$$A = \frac{V}{H} = \frac{24{,}000\ m^3}{3.5\ m} = 6{,}900\ m^2\ (0,69\,ha)$$

g) Oxygen requirements

$$RO = a.Q.(S_o - S) = \frac{1.0 \times 3000\ m^3/d \times (350 - 47)\ g/m^3}{1000\ g/kg}$$

$$= 909\ kgO_2/d = 38\ kgO_2/h$$

h) Power requirements

Adopt high-speed floating aerators. The oxygenation efficiency in standard conditions is adopted as:

$$OE_{standard} = 1.8\ kgO_2/kWh$$

The Oxygenation Efficiency in the field can be adopted as around 60% of the standard OE. Thus:

$$OE_{field} = 0.60 \times 1.8\ kgO_2/kWh = 1.1\ kgO_2/kWh$$

Example 15.1 (Continued)

The required power is:

$$P = \frac{OR}{OE} = \frac{38\ kgO_2/h}{1.1\ kgO_2/kWh} = 34\ kW \cong 45\ HP$$

i) Aerators

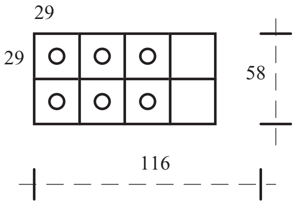
Adopt 6 aerators, each of 7.5 HP.
Therefore, the total installed power is 6 × 7.5 HP = 45 HP (34 kW)

j) Pond dimensions

Adopt two ponds in parallel. With two ponds, there is a larger flexibility during the occasional periods of sludge removal (one pond being cleaned and one pond in operation).

Considering an square area of influence for each aerator, and leaving the final zone without aerators, the pond can have the following dimensions:

Two ponds, each with L = 116 m and B = 29 m (8 squares with dimensions 29 m × 29 m)



According to Table 15.2, for a power of 7.5 HP for each aerator, the area of influence of 29 m × 29 m is inside the oxygenation zone (as desired), but is outside the mixing zone (also desirable, for a facultative aerated lagoon).

k) Verification of the power level

The average power level in the whole lagoon is:

$$\phi = \frac{P}{V} = \frac{34{,}000\ W}{24{,}000\ m^3} = 1.4\ W/m^3$$

This power level is expected to maintain solids in suspension. The estimation of 80 mg/l is reasonable (see Table 15.1), considering that there will be some settlement on the unaerated zone of the lagoon. The power level in the aerated zone only is larger, since the volume of the aerated zone is 75% of the total pond volume (3/4 of the pond length have aerators and $^1/_4$ is without aerators – see item j).

Example 15.1 (Continued)

l) Sludge accumulation

Annual accumulation = 0.05 m^3/inhab.year × 20,000 inhab = 1,000 m^3/year

Thickness in 1 year:

$$\text{Thickness} = \frac{1{,}000\ \text{m}^3/\text{year}\,.\,1\ \text{year}}{6{,}900\ \text{m}^2} = 0.14\ \text{m/year}$$

Thickness in 7 years of operation:

Thickness: 0.14 m/year × 7 years = 1.0 m in 7 years

After 7 years of operation, there will be an accumulation of sludge in the order of 1.0 m, which will reduce the net pond depth from 3.5 m to 2.5 m (reduction around 30%). Cleaning will probably be necessary after this period.

m) Total area required

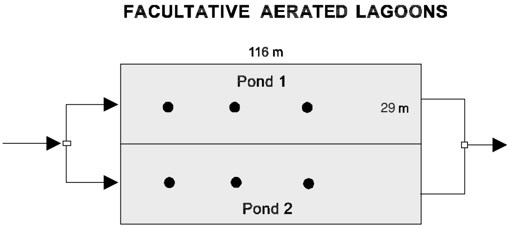
The required net area is 0.69 ha. The total area required for all the components of the treatment plant is approximately 30% greater than this value. Thus, the total area will be 1.30 × 0.69 = 0.90 ha.

The per capita land requirement is:

$$\text{Per capita land requirement} = \frac{\text{total area}}{\text{population}} = \frac{9{,}000\ \text{m}^2}{20{,}000\ \text{inhab}} = 0.45\ \text{m}^2/\text{inhab.}$$

This value is approximately 12% of the value required for a system with facultative ponds only (see Example 13.3).

n) Arrangement of the system



16

Complete-mix aerated lagoons followed by sedimentation ponds

16.1 INTRODUCTION

Complete-mix aerated lagoons are essentially aerobic. The aerators serve not only to *guarantee the oxygenation* of the medium but also to *maintain the suspended solids (biomass) dispersed in the liquid medium.* The typical detention time of a complete-mix aerated lagoon is in the order of 2 to 4 days.

The quality of the effluent from a complete-mix aerated lagoon is not adequate for direct discharge, owing to the high levels of suspended solids. For this reason, these lagoons are usually followed by other ponds, where settling and stabilisation of the settled solids can take place. These ponds are denominated *sedimentation ponds*. Figure 16.1 presents the flowsheet of the system.

The detention times in the sedimentation ponds are low, in the order of 2 days. This time is enough for an efficient removal of the suspended solids produced in the aerated lagoon. However, it does not contribute to an additional biochemical removal of BOD, as a result of the low biomass concentration maintained in suspension in the liquid medium (the biomass tends to settle). Besides this, the sludge accumulation capacity is relatively reduced, implying the need of its removal every 1 to 5 years (there are systems with continuous sludge removal, using pumps coupled to rafts).

The land requirements for this system are the *smallest within the pond systems.* The energy requirements are similar to the other aerated lagoon systems.

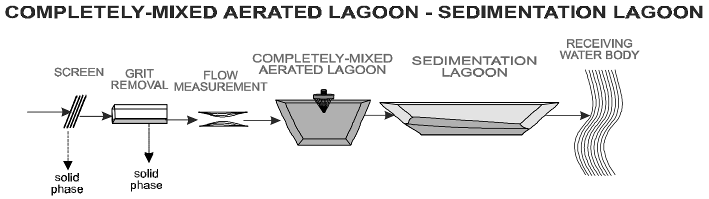


Figure 16.1. System of complete-mix aerated lagoon followed by sedimentation pond

16.2 DESCRIPTION OF THE PROCESS

In the aerated lagoon, the level of energy introduced by the aerators creates a turbulence that, besides guaranteeing the oxygenation, still allows all the solids to be maintained dispersed in the liquid medium. Therefore, the denomination *complete-mix lagoon* is due to the high degree of energy per unit of volume, responsible for the total mixing of the constituents in the whole pond. These lagoons are also called *flow-through lagoons*, in the sense that the liquid and the solids all flow together in the pond (without solids retention), resulting from the high mixing level. Another designation is *CSTR lagoons* (completely-stirred tank reactor).

Amongst the solids maintained in suspension and in complete mixing are included, besides the organic matter from the raw sewage, also the bacteria (biomass). Consequently, there is a larger concentration of bacteria in the liquid medium, together with a greater organic matter - biomass contact. Thus, the efficiency of the aerobic pond increases, also allowing a reduction in its volume.

The aerated lagoon acts in a similar way to the aeration tanks of the activated sludge process. The main difference is the absence of the recirculation of solids, an essential characteristic of the activated sludge process. Owing to the absence of the recirculation, the concentration of the biomass only reaches a certain value, which is dictated by the availability of the influent substrate (BOD load). The concentration of biological suspended solids in the aerated lagoon is in the order of 20 to 30 times less than in the reactor of activated sludge systems, which justifies the high efficiency of the latter.

However, in spite of the good efficiency of the aerated lagoons in the removal of the organic matter originally present in the wastewater, the quality of their effluent is not satisfactory for direct discharge into the receiving body. The biomass stays in suspension in the whole pond volume, and therefore leaves with the effluent from the aerated lagoon. This biomass is also organic matter, although of a different nature from the BOD of the raw sewage. If this organic matter generated in the lagoon is discharged into the receiving body, it will also exert an oxygen demand, causing the deterioration of the water quality.

Therefore, a downstream unit is needed, in which the suspended solids (predominantly the biomass) can settle. In the present case, this unit is represented by a *sedimentation pond* (in the activated sludge process, it is the secondary sedimentation tank). The effluent from the sedimentation pond leaves with a lower solids level and can be discharged directly into the receiving body.

16.3 DESIGN CRITERIA FOR THE COMPLETE-MIX AERATED LAGOONS

The main design criterion is the *detention time.* The design follows several principles of biological wastewater treatment described in chapters 9 and 31 (activated sludge).

a) Detention time

In complete-mix aerated lagoons, there is the following relationship between the detention times of the liquid and the biomass:

$$\boxed{\text{hydraulic detention time} = \text{solids retention time}}$$

or

$$\boxed{t = \theta_c}$$

The *hydraulic detention time (t) is* the average residence time of the liquid molecules in the reactor. The solids retention time, or *sludge age* (θ_c) is the average residence time of the bacterial cells in the reactor.

In the case of complete-mix aerated lagoons, due to the non-existence of sludge recirculation or any form of solids retention, the molecules of the liquid and the bacterial cells remain the same time in the reactor ($t = \theta c$). This important aspect has hydraulic and process implications. In the activated sludge system, the sludge age is the main design parameter. However, in complete-mix aerated lagoons, the hydraulic detention time (= sludge age) constitutes the main parameter.

In complete-mix aerated lagoons, the detention time varies in the range of:

$$\boxed{t = 2 \text{ to } 4 \text{ d}}$$

If more than one cell in series is adopted, the detention time in each one can be close to 2 days. The advantage of having detention times around 2 days is the reduction in the growth of algae, which could be washed out of the lagoon without being able to develop.

b) Depth

The depth of the pond should be selected in order to satisfy the requirements of the aeration equipment, in terms of mixing and oxygenation.

Usually, depth values are in the range of:

$$\boxed{H = 2.5 \text{ to } 4.0 \text{ m}}$$

16.4 ESTIMATION OF THE EFFLUENT BOD CONCENTRATION FROM THE AERATED LAGOON

For the estimation of the effluent BOD concentration from the aerated lagoon, models similar to those employed for the activated sludge process can be adopted. In this chapter, a simplified version based on first-order reactions is presented. In these conditions, the estimation of the effluent concentration follows a procedure similar to that used for the facultative aerated lagoons (Section 15.4).

The influence of the hydraulic regime of the pond can also be taken into consideration. However, the complete-mix model is usually adopted, since it offers a good approximation to the hydraulic behaviour of this type of aerated lagoon.

Also in this case the effluent from the aerated lagoons is composed of *dissolved organic matter* (**soluble BOD**) and *suspended organic matter* (**particulate BOD**) (see Section 15.4):

$$\boxed{BOD_{tot} = BOD_{sol} + BOD_{part}} \tag{16.1}$$

a) Soluble effluent BOD

The estimation of the effluent soluble BOD from the aerated lagoon can be done using the same formulas presented for facultative ponds and facultative aerated lagoons, which are a function of the hydraulic regime adopted for the reactor. As commented, the *complete-mix* model can be assumed.

The value of the removal coefficient K is, in the case of complete-mix aerated lagoons, even higher than in the other pond systems. This is due to the larger biomass concentration in the pond. Typical values of K are in the range of (Arceivala, 1981):

$$\boxed{K = 1.0 \text{ to } 1.5\ d^{-1}}$$

However, this value of K incorporates the influence of the concentration of the volatile suspended solids (VSS or X_v), which represent the biomass. The coefficient K can be dismembered into two fractions, so that:

$$\boxed{K = K'.X_v} \tag{16.2}$$

where:

K' = BOD removal coefficient $(mg/l)^{-1}(d)^{-1}$. The value of K′ is in the range of 0.01 to 0.03 $(mg/l)^{-1}(d)^{-1}$ (Arceivala, 1981)

X_v = concentration of volatile suspended solids (mg/L)

According with Equation 16.2, the larger the biomass concentration (X_v), the larger the coefficient K (K′ is constant) and, consequently, the larger the BOD removal efficiency.

The effluent soluble BOD concentration from the aerated lagoon is given by:

$$\boxed{S = \frac{S_o}{1 + K'.X_v.t}} \tag{16.3}$$

As in the other systems, S_o represents the influent total (soluble + particulate) BOD, while S represents the effluent soluble BOD.

It is interesting to point out that, within certain limits, S is independent from the influent concentration S_o. If S_o increases, the biomass concentration (X_v) increases proportionally, due to the larger food availability. If S_o decreases, X_v decreases, and S remains constant. This comment is for steady-state conditions (for design purposes), because fast variations of S_o (typical in operation) are not immediately accompanied by the increase of X_v.

The values of K and K′ are for a liquid temperature of 20 °C. For other temperatures, Equation 2.3 can be used, with the coefficient θ equal to 1.035.

The concentration of the biomass (X_v) is a result of the gross growth (positive factor) and the bacterial decay (negative factor). The formula for the calculation of X_v is:

$$\boxed{X_v = \frac{Y.(S_o - S)}{1 + K_d.t}} \tag{16.4}$$

where:

Y = yield coefficient (mgX_v/mgBOD_5), representing the amount of biomass (mg X_v) that is produced per unit substrate used (mg BOD_5).

K_d = bacterial decay coefficient or endogenous respiration coefficient (d^{-1}), representing the decay rate of the biomass during endogenous metabolism.

Typical values of these coefficients (Metcalf & Eddy, 1991) are presented in Table 16.1.

The value of the coefficient K_d, in this case, is slightly different from the value of K_d adopted in the chapters relating to activated sludge. In the equations for the activated sludge process a correction is adopted for the biodegradable fraction of VSS, which alter the value of K_d. For simplicity, in the case of aerated lagoons, the formulas are used without the biodegradable fraction concept.

Table 16.1. Kinetic and stoichiometric coefficient values

Coefficient	Unit	Range	Typical value
Y	mgVSS/mgBOD_5	0.4–0.8	0.6
K_d	d^{-1}	0.03–0.08	0.06

b) Particulate effluent BOD

To calculate the effluent particulate BOD from the complete-mix aerated lagoon, it is necessary to estimate the concentration of suspended solids in the effluent from the pond, since this BOD is caused by the suspended solids.

The concentration of volatile suspended solids in the effluent of the aerated lagoon is given by Equation 16.4.

The particulate BOD can be estimated based on the following relationship with the volatile suspended solids:

$$\boxed{\mathrm{BOD_{part}} = 0.4 \text{ to } 0.8\ \mathrm{mgBOD_5/mgVSS}}$$

In aerated lagoons, the relationship between the volatile suspended solids (VSS or X_v) and the total suspended solids (SS or X) is in the order of:

$$\boxed{X_v/X = 0.7 \text{ to } 0.8}$$

Thus, the particulate BOD can also be estimated as a function of the total suspended solids in the effluent, aggregating the last two relationships:

$$\boxed{\mathrm{BOD_{part}} = 0.3 \text{ to } 0.6\ \mathrm{mgBOD_5/mgSS}}$$

The particulate BOD in the final effluent is a function of the effluent SS from the sedimentation pond. There are no widely accepted models that allow the estimation of this effluent concentration. For design purposes, an SS removal efficiency around 80 to 85% can be admitted.

16.5 OXYGEN REQUIREMENTS IN THE AERATED LAGOON

The amount of oxygen to be supplied by the aerators for the aerobic stabilisation of the organic matter should usually be equal to the total ultimate BOD ($\mathrm{BOD_u}$) removed (see Section 15.5). The ratio $\mathrm{BOD_u/BOD_5}$ in the raw wastewater is in the order of 1.2 to 1.5.

In the computation of the total oxygen demand, the consumption not exerted by the volatile suspended solids that leave the system with the effluent can be discounted, similarly to what is done in the activated sludge system calculations. The oxygen requirements can then be calculated by:

$$\boxed{\mathrm{OR} = \frac{a.Q.(S_o - S)}{1000}} \tag{16.5}$$

where:

OR = oxygen requirement ($\mathrm{kgO_2/d}$)

a = coefficient of oxygen consumption (1.1 to 1.4 $\mathrm{kgO_2/kgBOD_5}$ removed)

Q = influent flow (m^3/d)
S_o = influent total (*soluble* + *particulate*) BOD concentration (g/m^3)
S = effluent *soluble* BOD concentration (g/m^3)
1000 = conversion of g to kg (g/kg)

16.6 POWER REQUIREMENTS IN THE AERATED LAGOON

In order to guarantee the mixing energy required for maintaining the suspended solids dispersed in the liquid medium, the *mixing requirements* should be fulfilled. The definition of the power for the aerators is then dictated by the concept of the *power level*.

As seen in Section 15.4b, the power level represents the energy introduced by the aerators per unit reactor volume, and is obtained by:

$$\boxed{\phi = P/V} \tag{16.6}$$

where:

ϕ = power level (W/m^3)
P = power for aeration (W)
V = reactor volume (m^3)

To ensure complete dispersion of the suspended solids in the aerated lagoon, the power level should be:

$$\boxed{\phi \geq 3.0\ W/m^3}$$

The required power (P) for *mixing* can be calculated through Equation 16.6, by adopting a value for ϕ and knowing V.

The required power for *oxygenation* may be determined using the concepts of Oxygen Requirement (OR) and Oxygenation Efficiency (OE – see Section 15.4).

The installed power must comply with both requirements.

16.7 DESIGN OF THE SEDIMENTATION POND

For the design of the sedimentation pond, the following required volumes should be estimated: (a) volume for clarification (sedimentation) and (b) volume for the storage and digestion of the sludge (Alem Sobrinho and Rodrigues, undated):

Volume required for clarification:

- Detention time: $t \geq 1$ d
- Depth: $H \geq 1.5$ m

Total volume of the pond:

- Detention time (end of the planning horizon): $t \leq 2.0$ d (to avoid algal growth)
- Depth: $H \geq 3.0$ m (to allow an aerobic layer above the sludge)

The sludge accumulation can be calculated assuming the following data:

- VSS/SS ratio in the influent solids to the settling pond: 0.70 to 0.80 (70 to 80% of the SS are volatile – see Section 16.4b)
- Volatile solids reduction rate in the sludge: $K_v = 0.5\ \text{year}^{-1}$ (50% removal per year) (Arceivala, 1981)

The following equation, modified from Arceivala (1981), allows the estimation of the accumulated sludge volume after a period of t years, as a function of the decay rate of the volatile solids and the accumulation rate of the fixed solids and assuming a density of the sludge close to 1.0:

$$\boxed{V_t = \frac{\dfrac{M_V}{K_v}.(1 - e^{-K_v.t}) + t.M_F}{1000.(\text{dry solids fraction})}} \qquad (16.7)$$

where:

V_t = volume of sludge accumulated after a period of t years (m^3)

M_v = mass of volatile suspended solids retained in the pond per unit time (kg VSS/year)

M_F = mass of fixed suspended solids retained in the pond per unit time (kg SS_F/year)

K_v = decay coefficient of the volatile suspended solids in the sludge in anaerobic conditions (year^{-1}). K_v varies from 0.4 to 0.6 year^{-1}, with an average value of 0.5 year^{-1}

t = time (year)

dry solids = fraction of dry solids in the sludge = 1 – water content fraction in the sludge

Example 16.1

Design a complete-mix aerated lagoon followed by a sedimentation pond, using the same data from the previous examples:

- Population = 20,000 inhabitants
- Influent flow: $Q = 3{,}000\ m^3/d$
- Influent BOD: $S_o = 350$ mg/L
- Temperature: T = 23 °C (liquid)

Example 16.1 (*Continued*)

Solution:

Aerated lagoon

a) Adoption of the detention time

t = 3 d (adopted)

b) Required volume

$V = t.Q = 3\ d \times 3,000\ m^3/d = 9,000\ m^3$

c) Required area

Adopting a depth H = 3.5 m:

$$A = \frac{V}{H} = \frac{9{,}000\ m^3}{3.5\ m} = 2{,}570\ m^2$$

The dimensions of the pond can be:

$$50\ m \times 50\ m (0.25\ ha)$$

d) Estimation of the concentration of volatile suspended solids (VSS) in the aerated lagoon

Kinetic coefficients (see Table 16.1):

- $Y = 0.6$ (adopted)
- $K_d = 0.06$ (adopted)

Estimation of the effluent soluble BOD concentration (S):

$$S = 50\ mg/L\ (initial\ estimate)$$

$$X_v = \frac{Y.(S_o - S)}{1 + K_d.t} = \frac{0.6 \times (350 - 50)}{1 + 0.06 \times 3} = 153\ mg/L$$

e) Estimation of the effluent soluble BOD

Assuming the complete-mix regime, and adopting the coefficient $K' = 0.017\ (mg/L)^{-1}(d)^{-1}$, which corresponds to $0.015\ (mg/L)^{-1}(d)^{-1}$ for 20 °C, after correction for 23 °C:

$$\text{Soluble BOD}_5\text{: } S = \frac{S_o}{1 + K'.X_v.t} = \frac{350}{1 + 0.017 \times 153 \times 3} = 40\ mg/L$$

Example 16.1 (_Continued_)

In item d above, the initial estimate of S = 50 mg/L can be corrected to S = 40 mg/L, and the VSS concentration recalculated until a satisfactory convergence. However, in this case, the differences will be small (for S = 40 mg/L → $X_v = 158$ mg/L).

This value of the soluble BOD is for the effluent from the aerated lagoon, as well as for the final effluent (since the removal of soluble BOD is neglected in the sedimentation pond).

f) Estimation of the effluent particulate BOD

Considering that the effluent from the aerated lagoon contains 153 mg/L of volatile suspended solids, the effluent particulate BOD from the aerated lagoon will be:

$$BOD_5 \text{ part} = 0.6 \text{ mgBOD}_5/\text{mgVSS} \times 153 \text{ mgVSS/L} = 92 \text{ mgBOD}_5/\text{L}$$

This value is high for direct release into the receiving body, which justifies the need of the sedimentation pond downstream. Assuming that the sedimentation pond presents an efficiency of 85% in the removal of these volatile suspended solids, the VSS concentration in the final effluent from the system will be:

$$VSS_e = \frac{(100 - E)}{100}.VSS_o = \frac{(100 - 85)}{100}.153 = 23 \text{ mg/L}$$

Thus, the particulate BOD in the final effluent will be:

$$BOD_5 \text{ part} = 0.6 \text{ mgBOD}_5/\text{mgVSS} \times 23 \text{ mgVSS/l} = 14 \text{ mgBOD}_5/\text{L}$$

g) Effluent total BOD

Total BOD = soluble BOD + particulate BOD = 40 + 14 = 54 mg/L

The efficiency of the system in the removal of BOD is:

$$E = \frac{S_o - S}{S_o} = \frac{350 - 54}{350}.100 = 85\%$$

h) Oxygen requirements

The oxygen requirements are around 1.1 to 1.4 of the removed BOD_5 load. Adopting the value of 1.2 $kgO_2/kgBOD_{rem}$:

$$RO = a.Q.(S_o - S) = \frac{1.2 \times 3000\,m^3/d.\ (350 - 40)\ g/m^3}{1000\,g/kg} = 1116\ kgO_2/d$$

$$= 47\ kgO_2/h$$

Example 16.1 (*Continued*)

i) Energy requirements

Adopt high-speed floating mechanical aerators. The Oxygenation Efficiency OE, in standard conditions, is in the order of:

$$\mathrm{OE} = 1.8\ \mathrm{kgO_2/kWh}$$

The oxygenation efficiency in the field can be adopted as around 60% of the standard OE. Thus:

$$\mathrm{OE_{field}} = 0.60 \times 1.8\ \mathrm{kgO_2/kWh} = 1.1\ \mathrm{kgO_2/kWh}$$

The required power is:

$$\mathrm{P} = \frac{\mathrm{OR}}{\mathrm{OE}} = \frac{47\ \mathrm{kgO_2/h}}{1.1\ \mathrm{kgO_2/kWh}} = 43\ \mathrm{kW} = 57\ \mathrm{HP}$$

j) Aerators

Adopt four aerators, each of 15 HP.
Therefore the total installed power is 4 × 15 HP = 60 HP (45 kW)

Each aerator will be responsible for an area of influence of 25 m × 25 m (the dimensions of the pond are 50 m × 50 m).

According with Table 15.2, for the power of 15 HP, the influence area is inside the oxygenation zone and close to the mixing zone. The depth of the pond is also satisfactory.

k) Verification of the power level

$$\phi = \frac{\mathrm{P}}{\mathrm{V}} = \frac{45{,}000\ \mathrm{W}}{9{,}000\ \mathrm{m^3}} = 5.0\ \mathrm{W/m^3}$$

This power level is enough to maintain all the solids in suspension (Table 15.1). Besides, it is greater than the value of 3.0 $\mathrm{W/m^3}$ suggested as the minimum for complete-mix aerated lagoons.

Sedimentation pond

l) Design of the sedimentation pond

- Clarification zone (reserved for the liquid):
  Detention time: t = 1.0 d (adopted)
  Volume: $\mathrm{V_{clarif}} = \mathrm{t.Q} = 1.0\ \mathrm{d} \times 3{,}000\ \mathrm{m^3/d} = 3{,}000\ \mathrm{m^3}$
  Depth: $\mathrm{H_{clarif}} = 1.5\ \mathrm{m}$ (adopted)
  Required area:

$$\mathrm{A} = \frac{\mathrm{V}}{\mathrm{H}} = \frac{3{,}000\ \mathrm{m^3}}{1.5\ \mathrm{m}} = 2{,}000\ \mathrm{m^2}(0.20\ \mathrm{ha})$$

Example 16.1 (Continued)

- Sludge zone (reserved for the storage and digestion of the sludge):
  Add an additional depth of 1.5 m.
- Total dimensions and values (clarification and sludge zones):
  Total area: 2,000 m^2
  Depth: 1.5 m + 1.5 m = 3.0 m
  Total volume: 2000 m^2 × 3.0 m = 6,000 m^3
  Number of ponds: 2
  Dimensions of each pond: 40 m × 25 m × 3.0 m
  Detention time in a still clean pond:

$$t = \frac{V}{Q} = \frac{6{,}000}{3{,}000} = 2.0\,\text{d}$$

m) Sludge accumulation

The load of influent solids to the settling pond is composed of volatile suspended solids VSS (determined in Section d) and fixed suspended solids SS_F. Assume a ratio of 0.75 for VSS/SS (see Section 16.4.b). Hence, the ratio SS_F/VSS will be:

$$SS_F/VSS = (1 - 0.75)/0.75 = 1/3$$

The influent solids loads to the pond per year are:

Volatile solids: VSS = 3,000 m^3/d × 0.153 kgVSS/m^3 × 365 d/year = 167,535 kgVSS/year
Fixed solids: SS_F: =3,000 m^3/d × (0.153/3) kgSS_F/m^3 × 365 d/year = 55,845 kgSS_F/year

Assuming a removal of 85% of the solids in the settling pond, the loads of volatile and fixed suspended solids that will be added to the sludge layer in the pond are:

$M_v = 0.85 \times 167{,}535 = 142{,}405$ kgVSS/year
$M_F = 0.85 \times 55{,}845 = 47{,}468$ kgSS_F/year

Adopting Equation 16.7 for the estimation of the sludge accumulation after a period of t years, and assuming a fraction of dry solids in the sludge of 8% (water content = 92%):

$$V_t = \frac{\frac{M_V}{K_v}.(1 - e^{-K_v.t}) + t.M_F}{1000.(\text{dry solids content})} = \frac{\frac{142{,}405}{0.5}.(1 - e^{-0.5\times t}) + t \times 47{,}468}{1000 \times 0.08}$$

Example 16.1 (*Continued*)

For different values of t, the sludge accumulation is:

Time (years)	Accumulated volume (m^3)	Ratio $V_{sludge}/V_{pond} = H_{sludge}/H_{pond}$	Sludge height (m)
0.5	1082	0.18	0.54
1.0	1991	0.33	0.99
1.5	2765	0.46	1.38
2.0	3433	0.57	1.71
2.5	4020	0.67	2.01
3.0	4542	0.76	2.28
3.5	5015	0.84	2.52

Column 2: equation above

Column 3: (column 2)/6,000 m^3, where 6,000 m^3 is the volume of the sedimentation pond

Column 4: (column 3) × 3.0 m, where 3.0 m is the total height of the sedimentation pond

It is observed that after a period of around 1.7 years of operation, the volume reserved for sludge accumulation (corresponding to the height of 1.5 m) is totally used. Therefore, the removal of the sludge from the pond is necessary before this period.

After 1.5 years, the volume of accumulated sludge corresponds to the following accumulation rate per inhabitant per year:

$(2{,}765\ m^3/\ 1.5\ \text{years})\ /\ 20{,}000\ \text{inhab.} = 0.09\ m^3/\text{inhab.year}$

n) Total area required (aerated lagoon + sedimentation pond)

Total area = 0.25 + 0.20 = 0.45 ha

The total area required for all the components of the works is approximately 30% higher than this value. Thus, the total area will be 1.30 × 0.45 ha = 0.59 ha (<0.90 ha, area required for the facultative aerated lagoon - Example 15.1).

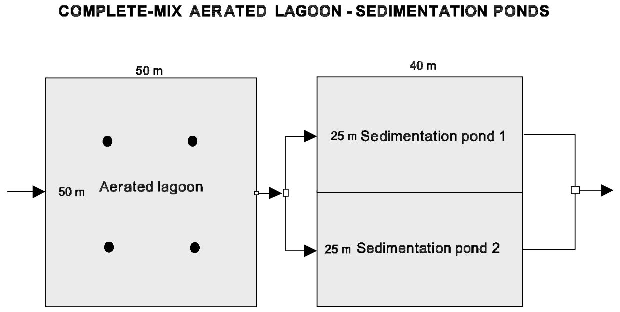
The per capita area requirement is:

$$\text{Per capita land requirement} = \frac{\text{total area}}{\text{population}} = \frac{5,900\,\text{m}^2}{20,000\,\text{inhab.}} = 0.30\,\text{m}^2/\text{inhab.}$$

This requirement is twelve times less than that for a primary facultative pond (Example 13.3).

Example 16.1 (*Continued*)

o) *Arrangement of the system*



17

Removal of pathogenic organisms

17.1 INTRODUCTION

The removal of pathogenic organisms is one of the most important objectives of stabilisation ponds. The organisms to be removed include bacteria, viruses, protozoan cysts and helminth eggs. A certain removal occurs in the anaerobic, facultative and aerated ponds. However, most of the removal takes place in the **maturation ponds**, which are especially designed for this purpose. Table 12.3 in Chapter 12 presents a summary of the removal efficiencies of the pathogens of interest in the main stabilisation pond systems.

Maturation ponds lead to a polishing of the effluent from any of the stabilisation pond systems previously described or, in broader terms, from any wastewater treatment system. Figure 17.1 shows the flowsheet of a system of anaerobic-facultative ponds followed by a series of maturation ponds. The main objective of maturation ponds is the **removal of pathogens**, and not an additional BOD removal. Maturation ponds constitute an economic alternative to the disinfection of the effluent by more conventional methods, such as chlorination.

17.2 PROCESS DESCRIPTION

The ideal environment for pathogenic organisms is the human intestinal tract. Outside it, in the sewerage system, sewage treatment plant or in the receiving water body, the pathogenic organisms tend to die. Several factors contribute to the

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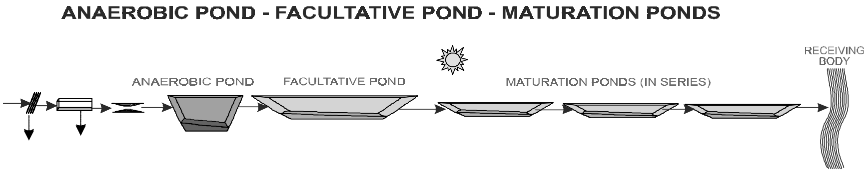


Figure 17.1. Typical flowsheet of a system of stabilisation ponds followed by maturation ponds in series.

removal of the pathogenic organisms:

- *bacteria* and *viruses*: temperature, solar radiation, pH, food shortage, predator organisms, toxic compounds
- *protozoan cysts* and *helminth eggs*: sedimentation

Maturation ponds are designed in order to provide an optimal utilisation of these mechanisms, especially for the removal of *bacteria* and *viruses*, which can be represented by the *coliforms* as indicators. Some of these mechanisms are more effective with smaller pond depths, which justifies the fact that the maturation ponds are shallower, compared with other types of ponds. Among the mechanisms associated to the low *depth* of the pond, the following can be mentioned (van Haandel et Lettinga, 1994; van Buuren et al, 1995; Cavalcanti et al, 2001):

- *High penetration of the solar radiation* (ultraviolet radiation)
- *High pH* (due to high photosynthetic activity)
- *High DO concentration* (favouring the aerobic community, which is more efficient in the removal of coliforms, besides increasing the removal rate due to other mechanisms, such as photooxidation)

The *maturation ponds* should reach high *coliform* removal efficiencies (E > **99.9** or **99.99**%), so that the effluent can comply with most uses of the water in the receiving water body, or for direct uses, such as irrigation (see Section 17.4). In order to maximise the coliform removal efficiency, the maturation ponds are designed with one of the following two configurations: (a) three or four ponds in series or (b) a single pond with baffles. These aspects will be detailed in this chapter.

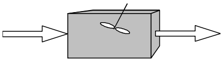
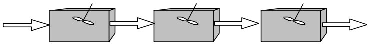
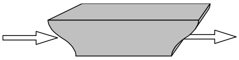
Regarding the other organisms of public health importance, which are not well represented by coliforms as indicators, the ponds usually reach complete (**100%**) removal of *protozoan cysts* and *helminth eggs* (Arceivala, 1981). The major removal mechanism is sedimentation.

17.3 ESTIMATION OF THE EFFLUENT COLIFORM CONCENTRATION

17.3.1 Influence of the hydraulic regime

The decay of the pathogenic organisms (bacteria and viruses), as well as of the indicators of faecal contamination (coliforms), follows *first-order kinetics* (similarly to

Table 17.1. Formulas for the calculation of the effluent coliform concentration (N) from ponds

Hydraulic regime	Scheme	Formula for the effluent coliform concentration (N)
Plug flow		$N = N_o e^{-K_b.t}$
Complete mix (1 cell)		$N = \frac{N_o}{1 + K_b.t}$
Complete mix (equal cells in series)		$N = \frac{N_o}{(1 + K_b.t/n)^n}$
Dispersed flow		$N = N_o.\frac{4ae^{1/2d}}{(1+a)^2e^{a/2d} - (1-a)^2e^{-a/2d}}$ $a = \sqrt{1 + 4K_b.t.d}$

N_o = coliform concentration in the influent(org/100mL)
N = coliform concentration in the effluent(org/100mL)
K_b = bacterial die – off coefficient (d^{-1})
t = detention time (d)
n = number of ponds in series (–)
d = dispersion number (dimensionless)

the BOD stabilisation in the pond systems, which also follows first-order kinetics). According with the first-order reactions, the *die-off rate of pathogens is proportional to the pathogen concentration at any time.* Hence, the greater the pathogen concentration, the larger the die-off rate. A similar comment is valid for the coliforms.

Therefore, the same considerations made in Section 13.6 are valid here. The hydraulic regime of the ponds has a great influence in the coliform removal efficiency. The decreasing order of efficiency is:

– *plug-flow pond*	greater efficiency
– *complete-mix ponds in series*	⇩
– *single complete-mix pond*	lower efficiency

Table 17.1 presents the formulas used for the determination of the coliform count in the effluent from ponds, as a function of the different hydraulic regimes.

17.3.2 Idealised hydraulic regimes

In order to obtain the extremely high coliform removal efficiencies that are usually required, the adoption of cells in series or a reactor approaching plug flow (theoretically equivalent to an infinite number of cells) is necessary. Table 17.2 presents the theoretical relative reactor volumes required, as a function of the number of cells, so that the same efficiency is reached. All the values are expressed as a function of the dimensionless product $K_b.t$. Thus, for a certain value of K_b, different total detention times are given, or, in other words, the total relative volume required. If the value of K_b is known, the table can be used for the direct calculation of the

Table 17.2. Theoretical relative volumes necessary to reach a certain removal efficiency, as a function of the number of complete-mix ponds in series

Number of ponds in series	Relative volume (dimensionless product $K_b.t$)			
	E = 90%	E = 99%	E = 99.9%	E = 99.99%
1	9.0	99	999	9999
2	4.3	18	61	198
3	3.5	11	27	62
4	3.1	8.6	18	36
5	2.9	7.6	15	27
∞ (plug flow)	2.3	4.6	6.9	9.2

total volume required (calculation of t, followed by the calculation of V, knowing that V = t.Q).

The interpretation of Table 17.2 leads to the following comments:

- with only one ideal complete-mix pond, extremely high volumes are necessary to reach satisfactory coliforms removal (for E = 99.99%, the necessary volume is approximately 1.000 times greater than for an ideal plug-flow reactor)
- with ponds in series, a substantial reduction of volume occurs only with a system comprised of more than 3 cells
- the ideal plug-flow reactor requires small volumes in comparison to the other systems
- these comments are valid assuming the ponds to be ideal reactors (what does not strictly occurs, in practice – plug-flow conditions are seldom achieved in practice)

Figure 17.2 illustrates the efficiencies and the number of logarithmic units removed, for different values of the dimensionless pair $K_b.t$ and the number of ideal complete-mix cells in series. An efficiency of E = 90% corresponds to the removal of one logarithmic unit; E = 99% → 2 log units; E = 99.9% → 3 log units; E = 99.99% → 4 log units; E = 99.999% → 5 log units, and so on, according to the formula:

$$\boxed{\text{log units removed} = -\log_{10}[(100 - E)/100]} \qquad (17.1)$$

In the figure, the highest efficiency of the ideal plug-flow reactor is again seen. Removal efficiencies above 99.9% without excessively large detention times can only be reached with a number of cells in series greater than four or preferably with a plug-flow regime.

However, it should be commented that *plug flow is an idealised hydraulic regime. In practice, it can be only approached (but not reached) through the adoption of*

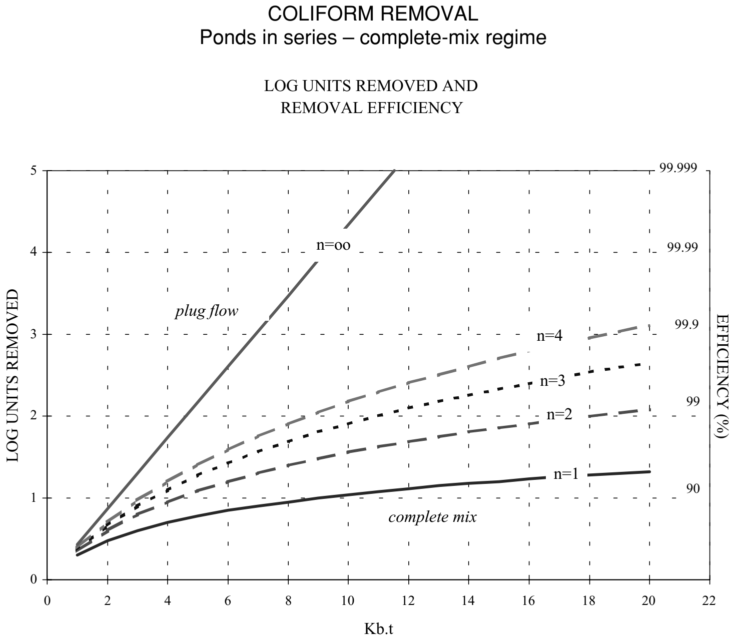


Figure 17.2. Coliform removal efficiencies, for different values of K_b.t and number of cells in series, assuming the complete-mix hydraulic regime

a low dispersion, induced by baffles. Zero dispersion (as assumed in the plug flow regime) is hardly achievable in a pond.

17.3.3 The dispersed-flow hydraulic regime

In reality, the behaviour of ponds follows the dispersed-flow hydraulic regime, and not the idealised regimes of complete mix and plug flow. Figure 17.3 presents the graph of the values of the efficiency E and the number of logarithmic units removed as a function of the dimensionless pair K_b.t and the dispersion number d. The determination of the dispersion number d was discussed in Section 13.6. It should be borne in mind that the coefficient K_b in the dispersed-flow regime is usually different from the value adopted for the complete-mix regime (see Sections 17.3.4 and 13.6.4).

In the case of a single pond, the figure shows clearly the importance of having a pond with a low dispersion number, tending to the plug-flow regime, in order to increase the removal efficiency. To obtain efficiencies greater than 99.9% (3-log removal) without excessive detention times, a dispersion number lower than 0.3, or preferably 0.1, is needed. These dispersion numbers are only obtained in ponds that have a length/breadth (L/B) ratio greater than 5 or 10 (see Table 13.7).

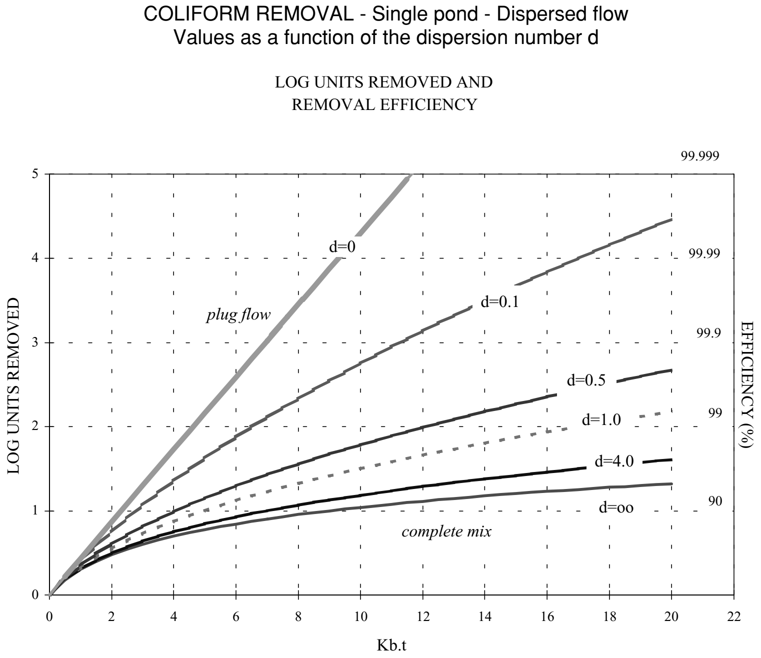


Figure 17.3. Coliform removal efficiency and number of log units removed in a single pond, for different values of K_b.t and d, assuming the dispersed-flow hydraulic regime

Figure 17.4 presents the number of logarithmic units removed and the removal efficiency in maturation ponds, expressed as a function of the length / breadth (L/B) ratio. In this figure, the relationship between the L/B ratio and the dispersion number d was calculated using the equation d = 1/ (L/B) (Equation 13.14).

The calculation of the L/B ratio in a pond with internal divisions (baffles) can be approximated by:

- *divisions parallel to the breadth B:*

$$\boxed{L/B = \frac{B}{L}(n+1)^2} \tag{17.2}$$

- *divisions parallel to the length L:*

$$\boxed{L/B = \frac{L}{B}(n+1)^2} \tag{17.3}$$

where:

L/B = resultant internal length/breadth ratio in the pond
L = length of the pond (m)
B = breadth of the pond (m)
n = number of internal divisions

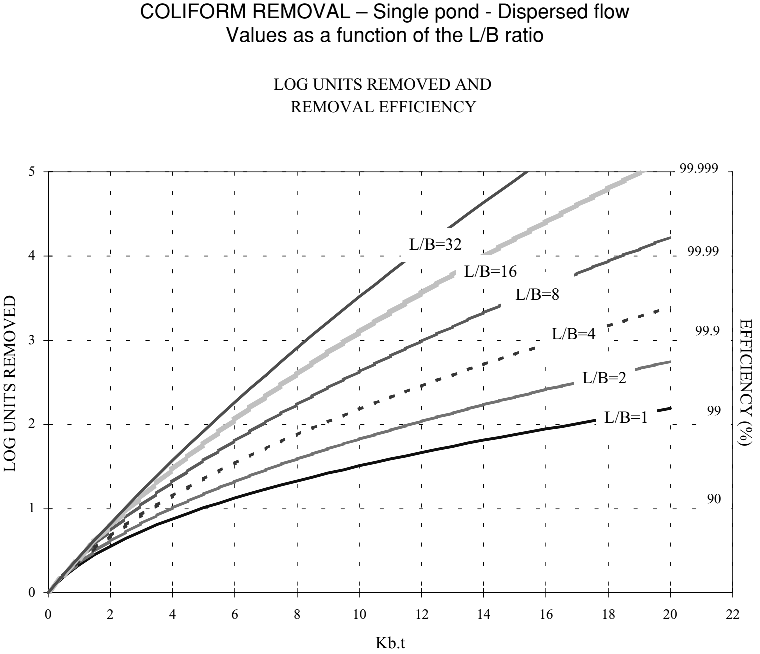


Figure 17.4. Coliform removal efficiency and number of log units removed for different values of K_b.t and L/B ratio, assuming dispersed flow. The relationship between L/B and d was calculated according to d = 1/ (L/B) (Equation 13.14).

17.3.4 The coliform die-off coefficient K_b according to the dispersed-flow regime

The coliform die-off coefficient (K_b) has a great influence on the estimation of the effluent coliform concentration. The literature presents a great scatter of reported coefficients, together with the additional complication that the different values of K_b have been obtained assuming different hydraulic regimes (not always reported). Besides that, there are other influencing factors, such as DO concentration, pH, solar radiation, BOD loads and the physical configuration of the pond.

The depth exerts a great influence in K_b: shallower ponds have higher K_b values because of the following points: (a) higher photosynthetic activity throughout the pond depth, leading to high pH and DO values; (b) higher penetration of the UV radiation throughout the pond depth (Catunda et al, 1994; van Haandel and Lettinga, 1994; von Sperling, 1999). However, the combined effect of the shallower ponds should be analysed: K_b is larger, but the detention time t is smaller (for a given surface area). The impact on the product K_b.t can be evaluated through the formulas presented for the different hydraulic regimes.

In ponds located in warm-climate regions and with a tendency to stratification, the anaerobic layer at the bottom plays a negative role. The bacterial die-off in anaerobic conditions is lower than in aerobic conditions. Therefore, in a facultative pond, the coliform removal efficiency in the summer may be lower than in a mild winter, in which there is a larger predominance of the aerobic conditions (Arceivala, 1981).

In a review of the international literature, von Sperling (1999) identified values of K_b for facultative and maturation ponds varying from 0.2 to 43.6 d^{-1} (20 °C). This is an extremely wide range, which gives little reliability for design purposes. The highest values were due to the fact that, in case the complete-mix regime had been assumed for a pond that did not behave in practice as an ideal complete mix, there was a tendency of obtaining overestimated values of K_b (see the discussion in Section 8.4.11).

Von Sperling (1999) investigated data from **33** *facultative* and *maturation* ponds in Brazil. The ponds analysed were distributed from the Northeast (latitude 7° S) to the South (latitude 23.5° S) of the country, covering a tropical to subtropical range of climates. The ponds had different volumes and physical configurations, with 13 being pilot units and the other 20 in full scale. The ponds represented a wide spectrum of operational conditions, with the length / breadth ratio (L/B) varying from 1 to 142 and the detention times from 0.5 to 114 days. In most cases, the coliform removal efficiency was based on average or long-term geometric means. The total number of data used was 66.

Complete-mix and dispersed-flow regimes were analysed in the work. It was observed that the values of the coefficient K_b for dispersed flow were related to the depth of the pond and to the hydraulic detention time. The lower the depth and the detention time, the larger the value of the coefficient K_b. As mentioned, the influence of the smaller depths is a result of the larger penetration of sunlight in the whole water mass (larger photosynthesis, larger dissolved oxygen, and larger pH values), besides the greater penetration of the ultraviolet radiation, which is bactericide. No significant relationship was observed between K_b and the depth or detention time for the complete-mix model.

An equation correlating K_b (dispersed flow) with the depth and the hydraulic detention time was determined through non-linear regression analysis with the available data (von Sperling, 1999):

$$\boxed{K_b\ (\text{dispersed}) = 0.917.H^{-0.877}.t^{-0.329} \qquad \textit{(33 ponds in Brazil)}} \qquad (17.4)$$

The Coefficient of Determination was very high ($R^2 = 0.847$), indicating a good fitting of the proposed model to the experimental data. Even though it was known, *a priori*, that a model with such a simple structure would have difficulty in reproducing the wide diversity of situations that occur in practice, there was the advantage of depending only on variables which, in a design application, are known beforehand (H and t). Some of the models available in the literature are

Table 17.3. Values of K_b (dispersed flow), obtained from Equation 17.5 ($K_b = 0.542.H^{-1.259}$), for facultative and maturation ponds

H (m)	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0	2.2	2.4
K_b (d^{-1})	1.03	0.72	0.54	0.43	0.35	0.30	0.26	0.23	0.20	0.18

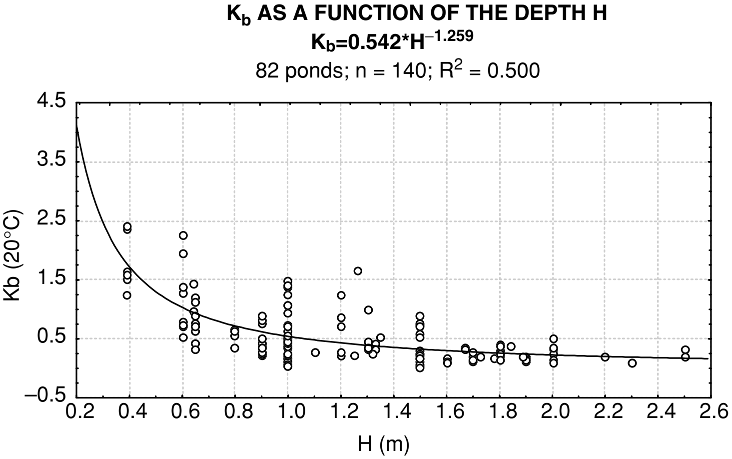


Figure 17.5. Regression analysis between K_b (20 °C, dispersed flow) and the depth H of the ponds. Dispersion number adopted as $d = 1/(L/B)$. 140 results from 82 facultative and maturation ponds in the world.

less practical, because they depend on variables that are not known at the design stage. In spite of the limitations, the model lead to a very good prediction of the logarithm of the effluent coliform concentrations from the 33 ponds ($R^2 = 0.959$).

Subsequently, the author enlarged the database to 82 ponds (140 mean data) in Brazil and in other countries (Argentina, Colombia, Chile, Venezuela, Mexico, Spain, Belgium, Morocco and Palestine). Equation 17.4 was still shown to be valid, although the Coefficient of Determination was reduced to $R^2 = 0.505$. In this enlarged data set, it was observed that the hydraulic detention time exerted a smaller influence and that it could be removed from the equation, without significantly affecting the performance of the model. The new equation obtained is presented below (see also Figure 17.5 and Table 17.3, showing the values of K_b and the best-fit curve). The prediction of the log of the effluent coliform concentration was still entirely satisfactory.

$$\boxed{K_b\ (\text{dispersed}) = 0.542.H^{-1.259} \qquad \textit{(82 ponds in the world)}} \qquad (17.5)$$

To allow a better visualisation of the results from both equations (Equations 17.4 and 17.5), Figure 17.6 presents the resulting curves for detention times varying

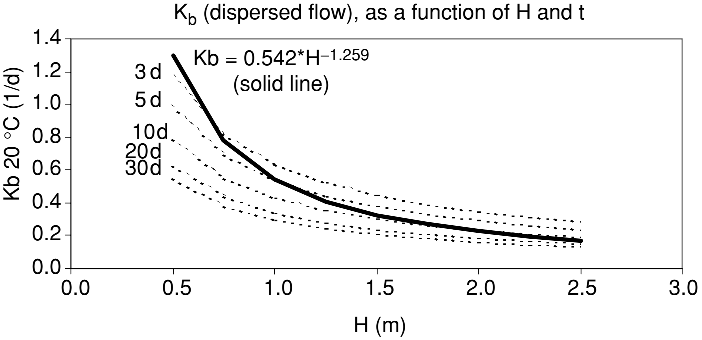


Figure 17.6. Relation between K_b, H, and t, according to the models proposed for K_b (20 °C, dispersed flow), for facultative and maturation ponds. Dashed curves: Equation 17.4 (33 ponds in Brazil); solid curve: Equation 17.5 (82 ponds in the world).

from 3 to 30 days, and depths varying from 0.5 to 2.5 m. It can be observed that the simpler model (Equation 17.5), based only on the depth H of the pond, is situated in an intermediate range between the curves of the model based on H and t (Equation 17.4), especially for depths greater than 1.0 m. For depths lower than 1.0 m, Equation 17.5 approaches Equation 17.4 only for low values of the hydraulic detention time. Low values of H and t occur simultaneously in maturation ponds in series, which also justifies that the simpler model keeps its practical applicability also for this range of values of H and t.

With the 140 data from the 82 facultative and maturation ponds in the world, it was tested whether the position of the pond in the series would have any influence on the coefficient K_b. The reason is due to the fact that primary and possibly secondary ponds tend to receive a higher BOD surface loading rate, not being, therefore, optimised for the production of high DO and pH values, as in tertiary and subsequent ponds. Even though an statistically significant difference has not been detected, if a refinement in the calculation is desired, the data suggest the following corrections in the values obtained from Equation 17.5 ($K_b = 0.542.H^{-1.259}$):

- *Primary and secondary ponds* –K_b: 5 to 15% lower than the value from the general equation
- *Tertiary and subsequent ponds* –K_b: 5 to 15% higher than the value from the general equation

Although Equation 17.5 has been derived from a large number of ponds distributed in several places of the world, specific local conditions can always prevail and lead to different values of K_b. For instance, places with very high solar radiation are prone to having high K_b values (higher UV radiation, higher photosynthesis, higher DO and higher pH). As mentioned, to incorporate this and other factors in the equation would lead to a very sophisticated model structure, requiring input data difficult to obtain in practice.

17.3.5 The coliform die-off coefficient K_b according to the complete-mix regime

In spite of the great advantages widely recognised for the dispersed-flow model, it is accepted that the idealised complete-mix model has been more utilised by designers. Von Sperling (2002) analysed the theoretical relationship between the coefficients, according to the hydraulic regimes of complete mix and dispersed flow, and proposed equations, based on regression analysis, which lead to an easy conversion between them. The equations allow the estimation of K_b for the complete-mix regime, based on the coefficient K_b for dispersed flow, on the detention time t (product $K_{b\,disp}.t$) and the dispersion number d. Two equations have been proposed, with different applicability ranges: one for a narrower range (more accurate in this narrow range) and another for a wider range of $K_b.t$ and d, covering most of the ponds found in practice (see also Section 8.4.11):

Wider range (d varying from 0.1 to 4.0; $K_{disp}.t$ varying from 0 to 10):

$$\frac{K_{mix}}{K_{disp}} = 1.0 + \left[0.0020 \times (K_{disp}.t)^{3.0137} \times d^{-1.4145}\right] \qquad (17.6)$$

Narrower range (d varying from 0.1 to 1.0; $K_{disp}.t$ varying from 0 to 5):

$$\frac{K_{mix}}{K_{disp}} = 1.0 + \left[0.0540 \times (K_{disp}.t)^{1.8166} \times d^{-0.8426}\right] \qquad (17.7)$$

where:

K_{disp} = bacterial die-off coefficient according to the dispersed flow regime (d^{-1})
K_{mix} = bacterial die-off coefficient according to the complete-mix regime (d^{-1})

These equations are valid, not only for coliforms, but also for other constituents that follow first-order kinetics, such as BOD.

The coefficient K_b for complete mix can be obtained from Equations 17.6 or 17.7, within the applicability range of each equation. It may be observed in both equations that, due to the factor of 1.0 on the right-hand side, *the coefficient for complete mix will always be greater than that for dispersed flow*.

The coefficient K_b for dispersed flow can be obtained from Equations 17.4 or 17.5. The dispersion number can be obtained from the formulas presented in Chapter 13 (Polprasert & Batharai, 1983; Agunwamba et al, 1992; Yanez, 1993; von Sperling, 1999). However, it is believed that the formula $d = 1/(L/B)$ (von Sperling, 1999) (Equation 13.14) can be adopted, given its simplicity and similarity of results with the other formulas.

It should be highlighted that, in principle, the die-off coefficient should not vary with the hydraulic model, but only represent the coliform decay according to its kinetics (as determined in a batch test). However, the inadequacy of the idealised hydraulic regimes in representing in a perfect way the hydrodynamic conditions of the pond leads to the deviations that occur in practice. In this sense,

Table 17.4. Values of K_b for complete mixing, at the temperature of 20 °C, for different values of the depth H, the L/B ratio, and the detention time t, for facultative and maturation ponds

		K_b complete mix (d^{-1})						K_b complete mix (d^{-1})			
		L/B ratio						L/B ratio			
t (d)	H (m)	1	2	3	4	t (d)	H (m)	1	2	3	4
3	*1.0*	0.61	0.67	0.72	0.77	20	*1.0*	1.97	4.34	7.29	10.68
	1.5	0.34	0.36	0.37	0.38		*1.5*	0.51	0.82	1.19	1.63
	2.0	0.23	0.24	0.24	0.25		*2.0*	0.42	0.57	0.71	0.84
	2.5	0.17	0.18	0.18	0.18		*2.5*	0.26	0.33	0.39	0.45
5	*1.0*	0.72	0.86	0.99	1.12	25	*1.0*	3.34	7.99	13.76	20.40
	1.5	0.37	0.40	0.43	0.46		*1.5*	0.69	1.29	2.03	2.88
	2.0	0.24	0.25	0.27	0.28		*2.0*	0.31	0.45	0.62	0.82
	2.5	0.18	0.18	0.19	0.19		*2.5*	0.20	0.24	0.30	0.36
10	*1.0*	1.17	1.67	2.13	2.57	30	*1.0*	*	*	*	*
	1.5	0.48	0.59	0.70	0.81		*1.5*	0.95	1.99	3.28	4.76
	2.0	0.28	0.32	0.36	0.40		*2.0*	0.37	0.62	0.92	1.26
	2.5	0.20	0.21	0.23	0.25		*2.5*	0.22	0.30	0.39	0.51
15	*1.0*	1.86	2.90	3.87	4.78	40	*1.0*	*	*	*	*
	1.5	0.64	0.89	1.11	1.33		*1.5*	*	*	*	*
	2.0	0.34	0.43	0.51	0.59		*2.0*	0.57	1.15	1.87	2.69
	2.5	0.22	0.26	0.30	0.34		*2.5*	0.28	0.47	0.70	0.97

(*) Considerable departure from the validity range of equations 17.6 and 17.7
Shaded cells: more usual values in facultative and maturation ponds
K_b for complete mix: Equations 17.6 and 17.7
K_b for dispersed flow: Equation 17.5
Dispersion number: $d = 1/(L/B)$

there are the following situations:

- in the *complete-mix* regime, the coefficients obtained experimentally are *larger* than those determined purely according to the kinetics, owing to the fact that the complete-mix reactors are less efficient
- in the *plug-flow* regime, the coefficients obtained experimentally are *smaller* than those obtained purely according to the kinetics, because the plug-flow reactors are more efficient
- in the *dispersed-flow* regime, the coefficients should be *close* to the values according to the kinetics, provided the dispersion number adopted for the pond is correct.

Table 17.4 presents the values of K_b for the complete-mixing hydraulic regime, obtained according to the methodology described above ($K_{b\ disp}$ estimated from Equation 17.5 and $K_{b\ mix}$ estimated from Equations 17.6 or 17.7, according to its applicability range). The values of the dispersion number d were converted to L/B values using Equation 13.14 [$d = 1/(L/B)$], to make the table more practical. The table presents only L/B ratios up to 4. Higher values could be calculated using equations 17.6 or 17.7 but, for a conceptual point of view, the ideal would be to use the dispersed-flow model, since, in practice, it is known that elongated ponds do not behave as complete-mix reactors.

Table 17.5. Summary of the ranges of typical values of K_b (20 °C) for facultative and maturation ponds, according to the dispersed-flow and complete-mix models

Pond type	Detention time t (d)	Depth H (m)	L/B ratio	K_b dispersed flow (d^{-1})	K_b complete mix (d^{-1})
Facultative	10 to 20	1.5 to 2.0	2 to 4	0.2 to 0.3	0.4 to 1.6
	20 to 40				1.6 to 5.0
Maturation (unbaffled, in series)	3 to 5 (in each pond)	0.8 to 1.0	1 to 3	0.4 to 0.7	0.6 to 1.2
Maturation (baffled, single pond)	10 to 20	0.8 to 1.0	6 to 12	0.4 to 0.7	(*)
Maturation (baffled, in series)	3 to 5 (in each pond)	0.8 to 1.0	6 to 12	0.4 to 0.7	(*)

Larger values of K_b: associated to smaller values of t, smaller values of H and larger values of L/B
For values outside the typical ranges: use methodology described in Sections 17.3.4 and 17.3.5
(*) Baffled maturation ponds: adoption of the dispersed-flow model is recommended

17.3.6 Summary of the coliform die-off coefficients

As a summary of all these considerations, Table 17.5 presents the typical range of resultant values of the coefficient K_b, for facultative and maturation ponds, according to the dispersed-flow and complete-mix hydraulic regimes. Values outside the typical ranges may be calculated using the methodologies in Sections 17.3.4 and 17.3.5. It can be observed that the ranges of K_b for dispersed flow are much narrower than those for complete mix, indicating a greater reliability in their estimation.

For other temperatures, K_b can be corrected by the formula:

$$\boxed{K_{bT} = K_{b20}.\theta^{(T-20)}} \qquad (17.8)$$

where:
θ = temperature coefficient

The values of θ also vary, according to the literature. Very high values ($\theta = 1.19$) were reported by Marais (1974). However, according to Yanez (1993) these values are overestimated, and the values of θ to be adopted should be in the range of **1.07** (7% increase in K_b for an increase of 1 °C in the temperature).

17.4 QUALITY REQUIREMENTS FOR THE EFFLUENT

Normally there are no discharge standards for coliforms. The water quality standards are usually with respect to the receiving body, as a function of its intended uses. See Section 3.5 for a discussion on quality standards.

If the effluent is to be used for unrestricted irrigation (for cultures that can present contamination risks), the recommended values according to the World Health Organisation (WHO, 1989) are (see Section 3.5.3.3):

- *faecal coliforms*: $\leq$1,000 faecal coliforms/100 mL (*geometric mean*)
- *helminth eggs*: $\leq$1 egg/L (*arithmetic average*)

For restricted irrigation, there is a limit for only helminth eggs (≤ 1 egg/L), and no limits for coliforms.

In any case, in terms of the receiving body or for agricultural reuse, the coliform counts in the effluent should be very low. Considering that the faecal (thermotolerant) coliform concentrations are in the order from 10^6 to 10^9 org/100mL in the raw sewage, the removal efficiencies in the treatment should be extremely high. To comply with the above criteria, coliform removal efficiencies of the order of 3 to 6 log units (99.9 to 99.9999%) are necessary in the wastewater treatment plant.

It should be noted that the mean referred above for the coliform concentration is expressed in terms of the **geometric mean**. Therefore, it is worthwhile to analyse this statistical parameter. For variables whose *values vary within several orders of magnitude*, it is more convenient to utilise the geometric mean, instead of the arithmetic mean. This is the case in the monitoring of coliforms, which vary within a very wide range, for instance, from 10^6 to 10^9 FC/100mL in raw wastewater. The higher values have a great weight on the arithmetic mean, distorting the concept of the mean as a measure of central tendency. In the range cited, the higher value is 1000 (10^3) greater than the lower value. The calculation of the geometric mean is presented below and illustrated in Example 17.1.

The geometric mean is given by the *n* root of the product of the *n* terms:

$$\boxed{\text{Geometric mean} = (x_1.x_2 \ldots x_n)^{1/n}} \qquad (17.9)$$

The geometric mean can be also calculated by:

$$\boxed{\text{Geometric mean} = 10^{\text{(arithmetic mean of the logarithms)}}} \qquad (17.10)$$

The following statement is also important, and easily obtainable from the considerations above:

$$\boxed{\text{Log}_{10} \text{ of the geometric mean} = \text{arithmetic mean of the log}_{10}} \qquad (17.11)$$

Example 17.1

In a monitoring programme, the following values of faecal (thermotolerant) coliforms have been obtained in four samples: 50, 400, 3000 and 20000 FC/100mL. These data, together with the base-10 logarithms ($\log_{10}$) are presented in the table below.

Coliform data (original data and log transformation)

Data	FC (FC/100 mL)	Log_{10}(FC)
1	5.00E + 01	1.699
2	4.00E + 02	2.602
3	3.00E + 03	3.477
4	2.00E + 04	4.301

***Example 17.1* (*Continued*)**

Calculate the geometric and the arithmetic means of the coliform concentrations.

Solution:

Applying Equation 17.9:

$$\text{Geometric mean} = (x_1.x_2.x_3.x_4)^{1/4} = (50 \times 400 \times 3000 \times 20000)^{1/4}$$
$$= 1047 = 1.047 \times 10^3 \text{FC}/100 \text{ mL}$$

The geometric mean can be also calculated through Equation 17.10. In the example, the arithmetic mean of the $\log_{10}$ of the FC values presented in the table is:

$$\text{Arithmetic mean of the logarithms} = (1.699 + 2.602 + 3.477 + 4.301)/4$$
$$= 3.020$$

Hence:

$$\text{Geometric mean} = 10^{(3.020)} = 1047 = 1.047 \times 10^3 \text{FC}/100\text{mL}$$

The value found is, of course, equal to the one obtained from Equation 17.9. The calculation using Equation 17.11 leads to:

$$\text{Log}_{10}(1.047) = 3.020$$

If the *arithmetic mean* of the original FC data had been calculated, the following value would have been obtained: 5863 FC/100mL = 5.863×10^3 CF/100mL. This value is much higher than that found through the geometric mean, being greater than 3 from the 4 data available, and not giving, therefore, a good indication of the central tendency of the data.

17.5 DESIGN CRITERIA FOR COLIFORM REMOVAL

The requirement of high efficiencies brings about the need to select a hydraulic regime that allows such high efficiencies. Hence, the maturation ponds should be designed according to one of the following two configurations:

- *baffled pond(s)* (aiming at approaching plug-flow conditions)
- *ponds in series* (preferably three or more)

The main design parameters are: hydraulic detention time (t), pond depth (H), number of ponds (n) and the length/breadth ratio (L/B).

In order to allow a preliminary analysis from the designer with respect to these parameters, Tables 17.6 (temperature of 20 °C) and 17.7 (temperature of 25 °C) present the coliform removal efficiencies that can be obtained in a single pond, for different values of t, H and L/B. The removal efficiencies are reported in terms of logarithmic units removed. The tables were composed according to the methodology proposed for dispersed flow – Equation 17.5 for K_b, Equation 13.14 for d and the formulas in Table 17.1. Table 17.7 was constructed correcting the coefficient K_b for $T = 25$ °C using the temperature coefficient $\theta = 1.07$. In order to broaden the application of the tables, they include typical depths and detention times, not just for maturation ponds, but also for facultative ponds.

The **overall removal efficiency** in a system comprised by a series of ponds with *different dimensions and characteristics* is given by:

$$\boxed{E = 1 - [(1 - E_1) \times (1 - E_2) \times \ldots \times (1 - E_n)]} \qquad (17.12)$$

where:

E = overall removal efficiency
E_1 = removal efficiency in pond 1
E_2 = removal efficiency in pond 2
E_n = removal efficiency in pond n

In this equation, all removal efficiencies should be expressed as a fraction, and not as percentage (e.g. 0.9, and not 90%).

In case the ponds have the *same dimensions and characteristics*, the formula is simplified to:

$$\boxed{E = 1 - (1 - E_n)^n} \qquad (17.13)$$

where:

E = overall removal efficiency
E_n = removal efficiency in any pond of the series
n = number of ponds in the series

In this equation, all removal efficiencies should be expressed as a fraction, and not as percentage (e.g. 0.9, and not 90%).

If the removal efficiencies are expressed in terms of *log units removed*, the overall removal is given by the sum of the individual efficiencies in each pond, irrespective of the dimensions and characteristics being the same or not:

$$\boxed{\text{log units} = (\text{log units pond 1}) + (\text{log units pond 2}) + \ldots + (\text{log units pond n})} \qquad (17.14)$$

where:

log units = log units removed in the overall system
log units pond 1 = log units removed in pond 1
log units pond 2 = log units removed in pond 2
log units pond n = log units removed in pond n

Table 17.6. Coliform removal efficiencies, expressed in terms of logarithmic units removed, for different values of the hydraulic detention time t, depth H and L/B ratio (dispersed flow). Temperature = **20 °C**

		Log units removed									
		L/B ratio									
t (d)	H (m)	1	2	3	4	6	8	10	12	16	32
3	1.0	0.48	0.51	0.54	0.56	0.59	0.61	0.62	0.63	0.65	0.67
	1.5	0.32	0.34	0.35	0.36	0.38	0.38	0.39	0.39	0.40	0.41
	2.0	0.24	0.25	0.26	0.26	0.27	0.28	0.28	0.28	0.28	0.29
	2.5	0.19	0.20	0.20	0.20	0.21	0.21	0.21	0.21	0.22	0.22
5	1.0	0.68	0.75	0.81	0.85	0.91	0.95	0.97	1.00	1.03	1.09
	1.5	0.48	0.51	0.54	0.56	0.59	0.61	0.62	0.63	0.65	0.67
	2.0	0.36	0.39	0.40	0.41	0.43	0.44	0.45	0.45	0.46	0.47
	2.5	0.29	0.31	0.32	0.32	0.33	0.34	0.35	0.35	0.35	0.36
10	1.0	1.05	1.21	1.33	1.42	1.55	1.65	1.72	1.78	1.87	2.05
	1.5	0.77	0.86	0.92	0.98	1.05	1.10	1.14	1.17	1.21	1.29
	2.0	0.60	0.66	0.70	0.74	0.78	0.81	0.84	0.85	0.88	0.92
	2.5	0.49	0.54	0.56	0.59	0.62	0.64	0.65	0.66	0.68	0.71
15	1.0	1.34	1.57	1.74	1.88	2.08	2.24	2.35	2.45	2.60	2.92
	1.5	0.99	1.13	1.24	1.32	1.44	1.52	1.59	1.64	1.71	1.87
	2.0	0.79	0.89	0.95	1.01	1.09	1.14	1.18	1.21	1.26	1.34
	2.5	0.66	0.72	0.77	0.81	0.87	0.90	0.93	0.95	0.98	1.04
20	1.0	1.57	1.87	2.09	2.27	2.54	2.75	2.91	3.04	3.25	3.72
	1.5	1.17	1.36	1.50	1.61	1.78	1.90	1.99	2.06	2.17	2.41
	2.0	0.95	1.08	1.17	1.25	1.36	1.43	1.49	1.54	1.61	1.75
	2.5	0.79	0.89	0.96	1.01	1.09	1.15	1.19	1.22	1.26	1.35
25	1.0	1.77	2.13	2.40	2.62	2.95	3.21	3.41	3.58	3.85	4.47
	1.5	1.34	1.57	1.74	1.88	2.08	2.24	2.36	2.45	2.60	2.92
	2.0	1.08	1.25	1.37	1.46	1.60	1.71	1.78	1.85	1.94	2.13
	2.5	0.91	1.04	1.13	1.20	1.30	1.37	1.43	1.47	1.53	1.66
30	1.0	1.95	2.37	2.68	2.94	3.33	3.63	3.87	4.08	4.40	5.17
	1.5	1.48	1.76	1.96	2.12	2.37	2.55	2.70	2.82	3.00	3.41
	2.0	1.20	1.40	1.55	1.66	1.83	1.96	2.06	2.13	2.25	2.50
	2.5	1.02	1.17	1.28	1.36	1.49	1.58	1.65	1.71	1.79	1.95
40	1.0	2.27	2.79	3.18	3.50	4.00	4.38	4.70	4.97	5.40	6.46
	1.5	1.73	2.08	2.34	2.55	2.87	3.12	3.32	3.48	3.74	4.32
	2.0	1.42	1.68	1.87	2.02	2.25	2.42	2.55	2.66	2.83	3.20
	2.5	1.21	1.41	1.55	1.67	1.84	1.97	2.07	2.14	2.26	2.52

K_b (dispersed flow) $= 0.542.H^{-1,259}$ $d = 1/(L/B)$

Log units removed. $= -\log_{10}(1 - \text{Efficiency}/100)$

Efficiency (%) $= 100.\,(N_o - N)/N_o = 100.(1 - 10^{-\text{log units removed}})$

Log units removed in a system with ponds in series = sum of the log units removed in each individual pond in the series

Table 17.7. Coliform removal efficiencies, expressed in terms of logarithmic units removed, for different values of the hydraulic detention time t, depth H and L/B ratio (dispersed flow). Temperature = **25 °C**

		Log units removed									
		L/B ratio									
t (d)	H (m)	1	2	3	4	6	8	10	12	16	32
3	1.0	0,61	0,66	0,71	0,74	0,79	0,82	0,84	0,86	0,88	0,93
	1.5	0,42	0,45	0,47	0,49	0,51	0,52	0,53	0,54	0,55	0,57
	2.0	0,32	0,33	0,35	0,36	0,37	0,38	0,38	0,39	0,39	0,40
	2.5	0,25	0,26	0,27	0,28	0,29	0,29	0,29	0,30	0,30	0,31
5	1.0	0,85	0,96	1,04	1,10	1,19	1,25	1,29	1,33	1,39	1,49
	1.5	0,61	0,67	0,71	0,74	0,79	0,82	0,84	0,86	0,88	0,93
	2.0	0,47	0,51	0,53	0,55	0,58	0,60	0,61	0,62	0,63	0,66
	2.5	0,38	0,40	0,42	0,43	0,45	0,46	0,47	0,48	0,49	0,50
10	1.0	1,29	1,51	1,67	1,79	1,99	2,13	2,24	2,33	2,47	2,76
	1.5	0,95	1,08	1,18	1,25	1,36	1,44	1,50	1,55	1,62	1,76
	2.0	0,76	0,84	0,91	0,96	1,03	1,08	1,12	1,14	1,18	1,26
	2.5	0,63	0,69	0,74	0,77	0,82	0,85	0,88	0,90	0,92	0,97
15	1.0	1,61	1,93	2,16	2,35	2,63	2,85	3,02	3,16	3,38	3,88
	1.5	1,21	1,41	1,56	1,67	1,84	1,97	2,07	2,15	2,27	2,52
	2.0	0,98	1,11	1,22	1,29	1,41	1,49	1,56	1,61	1,68	1,83
	2.5	0,82	0,92	1,00	1,05	1,14	1,19	1,24	1,27	1,32	1,42
20	1.0	1,88	2,28	2,58	2,82	3,18	3,47	3,70	3,89	4,19	4,90
	1.5	1,43	1,69	1,88	2,03	2,26	2,43	2,57	2,68	2,85	3,22
	2.0	1,16	1,34	1,48	1,59	1,75	1,86	1,95	2,02	2,13	2,36
	2.5	0,98	1,12	1,22	1,30	1,42	1,50	1,56	1,61	1,69	1,84
25	1.0	2,12	2,59	2,95	3,23	3,68	4,02	4,30	4,54	4,92	5,84
	1.5	1,61	1,93	2,16	2,35	2,63	2,85	3,02	3,16	3,38	3,88
	2.0	1,32	1,55	1,71	1,85	2,05	2,20	2,31	2,41	2,55	2,86
	2.5	1,12	1,29	1,42	1,52	1,67	1,78	1,87	1,93	2,03	2,24
30	1.0	2,33	2,87	3,28	3,61	4,13	4,53	4,86	5,14	5,60	6,71
	1.5	1,78	2,15	2,42	2,64	2,97	3,23	3,44	3,61	3,88	4,51
	2.0	1,46	1,73	1,93	2,09	2,33	2,51	2,65	2,77	2,95	3,34
	2.5	1,25	1,45	1,61	1,73	1,91	2,04	2,15	2,23	2,36	2,63
40	1.0	2,70	3,37	3,87	4,28	4,92	5,44	5,86	6,22	6,82	8,32
	1.5	2,07	2,53	2,88	3,15	3,58	3,92	4,19	4,42	4,78	5,66
	2.0	1,71	2,06	2,31	2,51	2,83	3,07	3,26	3,42	3,67	4,24
	2.5	1,47	1,74	1,94	2,10	2,34	2,52	2,66	2,78	2,96	3,36

K_b (dispersed flow) = $0.542.H^{-1,259}$ d = 1/ (L/B)
Log units removed. = $-\log_{10}(1 - \text{Efficiency}/100)$
Efficiency (%) = 100. $(N_o - N)/N_o = 100.(1 - 10^{-\text{log units removed}})$
Log units removed in a system with ponds in series = sum of the log units removed in each individual pond in the series

Regarding the **depth**, *maturation ponds* are usually designed with shallow depths, in order to maximise photosynthesis and the bactericidal effect of the UV radiation. Commonly adopted values are:

$$\boxed{\text{Depth} : \text{H} = 0.8 \text{ to } 1.0 \text{ m}}$$

Some researches (von Sperling *et al.*, 2003) have demonstrated the great advantages in terms of efficiency when using ponds with depths lower than 0.8 m. However, the possibility of the growth of rooted plants and the faster filling with sludge are aspects that need to be further investigated.

The introduction of baffles is facilitated due to the low depth of the maturation ponds. The baffles can be built with embankments, wood, pre-cast concrete walls, tarpaulin or plastic membranes supported on structures like internal fences.

When designing the maturation ponds, the previous coliform removal in the upstream units (e.g. anaerobic ponds, anaerobic reactors, facultative ponds) should be taken into consideration. Coliform removal in the facultative ponds can be estimated following the methodology presented in this chapter. For design purposes, the coliform removal in anaerobic ponds or UASB reactors can be adopted as 90% (1 logarithmic unit removed).

Mara (1996) also proposes the observation of the following criterion:

Minimum detention time in each pond, in order to avoid short circuits and the washing-out of the algae: 3 days

Example 17.2

Design a maturation pond system to treat the effluent from a facultative pond (Example 13.3), given the following characteristics:

- Population = 20,000 inhab
- Influent flow = 3,000 m^3/d
- Temperature: T = 23 °C (liquid)
- Faecal (thermotolerant) coliform concentration in the raw wastewater: $N_0 = 5 \times 10^7$ FC/100mL

Data from the facultative ponds (Example 13.3):

- Number of ponds in parallel: 2
- Length of each pond: L = 245 m
- Breadth of each pond: B = 98 m
- Depth: H = 1.8 m
- Hydraulic detention time: t = 28.8 d

***Example 17.2* (*Continued*)**

Solution:

1. Coliform removal in the facultative ponds

a) Hydraulic regime to be adopted in the calculations

Adopt the dispersed flow regime.

b) Dispersion number d

Adopting Equation 13.14, and knowing that the L/B ratio in each facultative pond is 2.5 (245 m/98 m = 2.5):

$$d = 1/(L/B) = 1/2.5 = 0.40$$

If the formula of Agunwamba (1992) and Yanez (1993) had been used, the values of d = 0.42 and d = 0.37, respectively, would have been obtained, which are very close to the values obtained above.

c) Coliform removal coefficient

Using Equation 17.5 for dispersed flow, the value of the bacterial decay coefficient is obtained:

$$K_b \text{ (dispersed flow)} = 0.542.H^{-1.259} = 0.542 \times 1.80^{-1.259} = 0.26\ d^{-1}(20\,^\circ C)$$

If Equation 17.4 (based on H and t) had been used, $K_b = 0.18\ d^{-1}$ would have been obtained.
Correcting K_b for 23 °C:

$$K_{bT} = K_{b20}.\theta^{(T-20)} = 0.26 \times 1.07^{(23-20)} = 0.32 d^{-1}$$

d) Effluent coliform concentration

Adopting the equation for dispersed flow (Table 17.1), and knowing that the detention time in the facultative ponds is 28.8 days:

$$a = \sqrt{1 + 4K.t.d} = \sqrt{1 + 4 \times 0.32 \times 28.8 \times 0.40} = 3.95$$

$$N = N_0.\frac{4ae^{1/2d}}{(1+a)^2e^{a/2d} - (1-a)^2e^{-a/2d}}$$

$$= 5.0 \times 10^7.\frac{4 \times 3.95.e^{1/(2\times 0.40)}}{(1+3.95)^2.e^{3.95/(2\times 0.40)} - (1-3.95)^2.e^{-3.95/(2\times 0.40)}}$$

$$= 8.2 \times 10^5 FC/100mL$$

This effluent concentration from the facultative pond is the influent concentration to the maturation ponds.

Example 17.2 (*Continued*)

The coliform removal efficiency in the facultative pond is:

$$E = \frac{N_o - N}{N_o} \times 100 = \frac{5.0 \times 10^7 - 8.2 \times 10^5}{5.0 \times 10^7} \times 100 = 98.4\%$$

2. Alternative: three maturation ponds in series

e) Volume of the ponds

Adopt a total detention time equal to 12 days (4 days in each pond).
Volume of each pond:

$$V = t.Q = 4\,d \times 3{,}000\,m^3/d = 12{,}000\,m^3$$

f) Dimension of the ponds

Depth: H = 1.0 m (adopted)

Surface area of each pond: $A = V/H = 12{,}000\ m^3/1.0\ m = 12{,}000\ m^2$
Total surface area: $12{,}000 m^2 \times 3 = 36{,}000\ m^2$

Dimensions: adopt square ponds (L/B ratio = 1.0) in this example

Number of ponds: 3
Length = 110 m
Breadth = 110 m
Depth = 1.0 m

Rectangular ponds could have been also adopted, in order to improve the hydraulic characteristics and minimise the dispersion number.

The total area required by the maturation ponds (including banks, roads etc) is around 25% greater than the net area determined. Therefore, the total area required is estimated as $1.25 \times 36{,}000\,m^2 = 45{,}000\,m^2 = 4.5$ ha ($2.25\ m^2$/inhab.).

g) Coliform concentration in the final effluent

Calculation according to the dispersed flow model:

Dispersion number according to Equation 13.14, for L/B = 1:

$$d = 1/(L/B) = 1/1.0 = 1.0$$

If the formula of Yanez (1993), Equation 13.13, had been applied, a value of d = 0.99 would have been obtained (very close to the value obtained above).

The value of the coliform die-off coefficient is given by (Equation 17.5):

$$K_b\ (\text{dispersed flow}) = 0.542.H^{-1.259} = 0.542 \times 1.0^{-1.259} = 0.54\,d^{-1}(20\,^\circ C)$$

If Equation 17.4 (based on H and t) had been used, a value of $K_b = 0.58\ d^{-1}$ would have been obtained.

Example 17.2 (Continued)

For T = 23 °C, the value of K_b is:

$$K_{bT} = K_{b20}.\theta^{\,(T-20)} = 0.54 \times 1.07^{(23-20)} = 0.66 d^{-1}$$

The effluent coliform concentration from the 1st pond in the series is:

$$a = \sqrt{1 + 4K.t.d} = \sqrt{1 + 4 \times 0.66 \times 4.0 \times 1.0} = 3.42$$

$$N = N_o.\frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}}$$

$$= 8.2 \times 10^5.\frac{4 \times 3.42.e^{1/(2\times 1.0)}}{(1+3.42)^2.e^{3.42/(2\times 1.0)} - (1-3.42)^2.e^{-3.42/(2\times 1.0)}}$$

$$= 1.7 \times 10^5 FC/100mL$$

The removal efficiency in the 1st pond of the series is:

$$E = \frac{N_o - N}{N_o} \times 100 = \frac{8.2 \times 10^5 - 1.7 \times 10^5}{8.2 \times 10^5} = 0.789 = 79\%$$

Considering that the three ponds have the same dimensions, the efficiency of the series of n = 3 ponds can be calculated:

$$E_n = 1 - (1 - E_1)^n = 1 - (1 - 0.789)^3 = 0.991 = 99.1\%$$

The coliform concentration in the final effluent is:

$$N = N_0.(1 - E) = 8.2 \times 10^5.(1 - 0.991) = 7.7 \times 10^3 \text{ FC}/100mL$$

Calculation according to the complete-mix model:

For illustration and comparison, the calculation for the complete-mix hydraulic regime is presented.

Coefficient K_b (20 °C) for complete mix, based on the coefficient K_b for dispersed flow ($K_b = 0.54\ d^{-1}$, for T = 20 °C), t = 4.0 d and d = 1.0 – according to Equation 17.7:

$$\frac{K_{b\,mix}}{K_{b\,disp}} = 1.0 + \left[0.0540 \times (K_{b\,disp}.t)^{1.8166} \times d^{-0.8426}\right]$$

$$= 1.0 + \left[0.0540 \times (0.54 \times 4.0)^{1.8166} \times 1.0^{-1.4145}\right] = 1.22$$

$$K_{mix} = 1.22 \times K_{disp} = 1.22 \times 0.54 = 0.66\,d^{-1}(20^\circ C)$$

For T = 23 °C, K_b is corrected to $K_b = 0.81 d^{-1}$.

Example 17.2 (*Continued*)

The coliform concentration in the final effluent is given directly by the following equation, considering the total detention time of 12 d in all the ponds and the number of ponds n = 3 (see Table 17.1):

$$N = \frac{N_o}{\left(1 + K_b.\frac{t}{n}\right)^n} = \frac{8.2 \times 10^5}{\left(1 + 0.81.\dfrac{12}{3}\right)^3} = 1.0 \times 10^4 \text{FC/100mL}$$

The efficiency of the maturation ponds is:

$$E = \frac{N_o - N}{N_o} \times 100 = \frac{8.2 \times 10^5 - 1.0 \times 10^4}{8.2 \times 10^5} = 0.987 = 98.7\%$$

h) Overall removal efficiency

The *overall* efficiency of the facultative ponds – maturation ponds system in the removal of coliforms is:

- Dispersed-flow model for the maturation ponds:

$$E = \frac{N_o - N}{N_o} \times 100 = \frac{5.0 \times 10^7 - 7.7 \times 10^3}{5.0 \times 10^7} \times 100 = 99.984\%$$

- Complete-mix model for the maturation ponds:

$$E = \frac{N_o - N}{N_o} \times 100 = \frac{5.0 \times 10^7 - 1.0 \times 10^4}{5.0 \times 10^7} \times 100 = 99.980\%$$

$$\begin{aligned}\text{Log units removed} &= -\log(1 - E/100) = -\log(1 - 99.984/100)\\ &= 3.80 \text{ log units removed}\end{aligned}$$

Notes: the dispersed-flow and complete-mix models lead to a global removal efficiency of 99.98% (facultative pond - maturation ponds). The effluent coliform estimations led to: dispersed-flow model: 7.7×10^3 FC/100mL; complete-mix model: 1.0×10^4 CF/100 mL. These deviations are small and should be interpreted taking into account the whole uncertainty in the computations involving coliforms and the rounding-ups made in the calculations.

The proposed system of ponds does not comply with the WHO guidelines for unrestricted irrigation (1×10^3 FC/100 mL), but it can comply with some water body standards, depending on the dilution ratio of the receiving watercourse. In any case, the high contribution given by the maturation ponds in the removal of faecal coliforms can be clearly seen.

If higher removal efficiencies are desired, the total detention time and/or number of ponds can be increased, until the desired effluent quality is reached. In addition, each pond may be more elongated, instead of being square.

However, *the increase in the detention time in each pond must be achieved through the increase in the surface area, and not in the depth*. If the depth is

Example 17.2 (_Continued_)

increased, the value of K_b will be reduced, and the efficiency will not rise as desired.

If a higher number of ponds in series is adopted, the detention time in each individual pond must be verified to see whether it is greater than or equal to 3 d. For instance, 4 ponds in series, with a total detention time of 12 days will lead to t = 3 days in each pond, which should be the minimum acceptable value, according to Mara (1996).

3. Alternative: Single pond with baffles

j) Volume of the pond

Adopt a detention time equal to 12 days.

Volume of the maturation pond:

$$V = t.Q = 12\,d \times 3{,}000\ m^3/d = 36{,}000\ m^3$$

k) Dimensions of the pond

Depth: H = 1.0 m (adopted)

Surface area: $A = V/H = 36{,}000\ m^3/1.0\ m = 36{,}000\ m^2$

Adopt square external dimensions, but internal dimensions divided with 3 baffles. The baffles can be of tarpaulin, wood, earth banks, or other appropriate material.

External dimensions:

Length: L = 190 m
Breadth: B = 190 m

The internal L/B ratio of the pond will be (Equation 17.3):

$$L/B = \frac{L}{B}(n+1)^2 = \frac{190}{190}.(3+1)^2 = 16$$

Due to the division of the internal area with 3 baffles, the pond will have 4 compartments, each one with a length of 190 m and a width of 190/4 = 47.5 m. The pond can be considered as behaving as a rectangular pond, with a L/B ratio = 16, total length L = 190 × 4 = 760 m and width 47.5 m.

The total area required for the maturation pond (including banks, roads, etc.) is around 25% greater than the calculated net area. Therefore, the total area required is estimated as $1.25 \times 36{,}000\,m^2 = 45{,}000\,m^2 = 4.5\,ha$ ($2.25\ m^2$/inhab.).

l) Hydraulic regime to be adopted in the calculations

Adopt the dispersed-flow regime.

m) Dispersion number

Adopting Equation 13.14, with L/B = 16:

d = 1/ (L/B) = 1/16 = 0.06

Example 17.2 (*Continued*)

If the formula of Agunwamba (1992) had been used, the value $d = 0.11$ would have been obtained, along with $d = 0.06$ for the formula of Yanez (1993).

n) Coliform die-off coefficient

The value of the bacterial die-off coefficient can be given by (Equation 17.5):

K_b (dispersed flow) $= 0.542.H^{-1.259} = 0.542 \times 1.0^{-1.259} = 0.54\ d^{-1}$ (20 °C)

If Equation 17.4 (based on H and t) had been used, a value of $K_b = 0.40\ d^{-1}$ would have been obtained.

For T = 23 °C, the value of K_b is:

$$K_{bT} = K_{b20}.\ \theta^{(T-20)} = 0.54 \times 1.07^{(23-20)} = 0.66\ d^{-1}$$

o) Effluent coliform concentration

Adopting the equation for dispersed flow (Table 17.1):

$$a = \sqrt{1 + 4K.t.d} = \sqrt{1 + 4 \times 0.66 \times 12 \times 0.06} = 1.73$$

$$N = N_o.\frac{4ae^{1/2d}}{(1+a)^2e^{a/2d} - (1-a)^2e^{-a/2d}}$$

$$= 8.2 \times 10^5.\frac{4 \times 1.73.e^{1/(2\times0.06)}}{(1+1.73)^2.e^{1.73/(2\times0.06)} - (1-1.73)^2.e^{-1.73/(2\times0.06)}}$$

$$= 2.2 \times 10^3\ FC/100\,mL$$

This system also does not comply (although it comes close) with the WHO guidelines for unrestricted irrigation (1×10^3 FC/100 mL), but it can comply with some water body standards, depending on the dilution ratio of the receiving watercourse. In this specific example, the results are slightly better than in the case of the three maturation ponds in series. In any case, the high contribution given by the maturation ponds in the removal of faecal coliforms can be clearly seen.

See comments in item h regarding the improvement in the effluent quality.

p) Removal efficiencies

The efficiency of the maturation pond is:

$$E = \frac{N_o - N}{N_o} \times 100 = \frac{8.2 \times 10^5 - 2.2 \times 10^3}{8.2 \times 10^5} \times 100 = 99.7\%$$

The *overall* efficiency of the facultative ponds – maturation pond systems in the removal of coliforms is:

$$E = \frac{N_o - N}{N_o} \times 100 = \frac{5.0 \times 10^7 - 2.2 \times 10^3}{5.0 \times 10^7} \times 100 = 99.996\%$$

Example 17.2 (Continued)

$$\text{Log units removed} = -\log(1 - E/100) = -\log(1 - 99.996/100)$$
$$= 4.35 \text{ log units removed}$$

Note: If the complete-mix model had been adopted (although it is not indicated for ponds with high L/B ratios), the following results would have been obtained, using the methodology exemplified in item i: $K_{mix}/K_{disp} = 17.84$ (Equation 17.5, with K_b dispersed $= 0.54\ d^{-1}$ for $T = 20\,°C$); K_b complete-mix $= 0.54 \times 17.84 = 9.67\ d^{-1}$ ($20\,°C$) and K_b complete-mix $= 11.85\ d^{-1}$ ($23\,°C$); effluent FC $= 5.7 \times 10^3$ FC/100 mL. This value of effluent faecal coliforms is close to the value estimated according to the dispersed flow model (2.2×10^3 FC/100mL), indicating the suitability of the proposed approach for the estimation of the effluent coliforms of the ponds. Naturally, priority should be given to the utilisation of the dispersed flow model, due to it being conceptually more adequate.

4. Comparison between the two alternatives

Item	Alternative: 3 maturation ponds in series	Alternative: 1 maturation pond with 3 baffles (4 compartments)
Number of ponds	3 in series	1
Number of baffles	–	3
Total detention time (d)	12	12
Detention time in each pond (d)	4	12
Net area required (ha)	3.6	3.6
Gross area required (ha)	4.5	4.5
Length of each pond (m)	110	190
Width of each pond (m)	110	190
Depth (m)	1.0	1.0
FC in the influent to the facultative pond (FC/100 mL)	5.0×10^7	5.0×10^7
FC in the influent to the maturation pond (FC/100 mL)	8.2×10^5	8.2×10^5
FC in the final effluent (FC/100 mL)	7.7×10^3	2.2×10^3
Efficiency of the maturation ponds (%)	99.1	99.7
Global efficiency (facultative + maturation) (%)	99.984	99.996
Log units removed (global)	3.80	4.35

It can be observed that both alternatives are equivalent from the point of view of land requirements and not so different in terms of the quality of the final effluent. In each alternative, it is still possible to have an optimisation in the design, leading to improvements in the effluent quality. In the selection of the alternative, other items should be investigated, related to costs, topography, soil and other local factors.

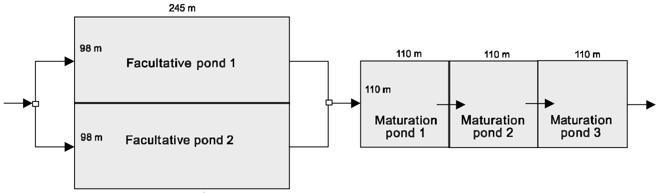
Example 17.2 (Continued)

Note: in the calculations, small differences may occur due to rounding errors (the calculations have been done using a spreadsheet, which does not round numeric values).

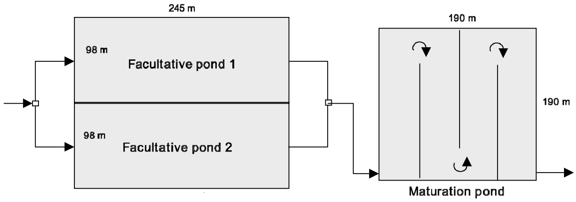
5. Arrangement of the ponds (including the facultative ponds)

LAYOUT OF THE FACULTATIVE PONDS - MATURATION PONDS SYSTEM

MATURATION PONDS IN SERIES



BAFFLED MATURATION PONDS



17.6 REMOVAL OF HELMINTH EGGS

17.6.1 Removal of helminth eggs from the wastewater

Helminth eggs are removed by sedimentation, which largely occurs in the anaerobic and facultative ponds. If there are eventually still eggs remaining in the effluent from those ponds, there will be further sedimentation in the maturation ponds. If the WHO guidelines for restricted and unrestricted irrigation ($\leq$1 egg/litre) must be satisfied, it can be considered that a system of ponds is likely to produce an effluent that contains frequently zero eggs per litre.

Ayres et al (1992), analysing data of helminth eggs removal in ponds in Brazil, Kenya, and India, developed equations 17.15 and 17.16, valid for anaerobic, facultative and maturation ponds. The equations should be applied sequentially in each pond of the series, so that the number of eggs in the final effluent can be determined (Mara et al, 1992). The model of Ayres et al (1992), applied to a baffled pilot pond in Southeast Brazil, showed good results (von Sperling et al, 2001, 2002).

- Average removal efficiency (to be used to represent average operation conditions):

$$\boxed{E = 100 \times \left[1 - 0.14.e^{(-0.38.t)}\right]} \tag{17.15}$$

Table 17.8. Removal efficiency of helminth eggs, according to the model of Ayres et al (1992)

Hydraulic detention time (d)	Removal efficiency (%)		Logarithmic units removed	
	Average values	95% confidence	Average values	95% confidence
2	93.45	84.08	1.18	0.80
4	96.94	93.38	1.51	1.18
6	98.57	97.06	1.84	1.53
8	99.33	98.60	2.17	1.85
10	99.69	99.29	2.50	2.15
12	99.85	99.61	2.83	2.41
14	99.93	99.77	3.16	2.64
16	99.97	99.86	3.49	2.85
18	99.985	99.90	3.82	3.02
20	99.993	99.93	4.15	3.17
22	99.997	99.95	4.48	3.28
24	99.998	99.957	4.81	3.37
26	99.999	99.962	5.14	3.42
28	99.9997	99.965	5.47	3.45
30	99.9998	99.964	5.80	3.45

Log units removed $= -\log(1 - E/100)$
Efficiency (%): $E = 100.(1 - 10^{-\text{log units removed}})$

- Removal efficiency according to the lower confidence limit of 95% (to be used for design, as a safety measure):

$$\boxed{E = 100 \times \left[1 - 0.41.e^{(-0.49.t+0.0085.t^2)}\right]} \qquad (17.16)$$

where:

E = removal efficiency of helminth eggs (%)
t = hydraulic detention time in each pond of the series (d)

Table 17.8 and Figure 17.7 present the values of the removal efficiency resulting from the application of Equations 17.15 and 17.16.

The concentration to be reached in the effluent also depends largely on the influent concentration. The concentration of eggs in the raw sewage is a function of the sanitary conditions of the population. Typical values are situated in the wide range of **10^1 to 10^3 eggs/L**, with the range between 10^2 and 10^3 eggs/L associated to populations with very unfavourable sanitary conditions. Hence, to reach a final effluent with less than 1 egg/L, for restricted and unrestricted irrigation, the removal efficiencies should be between 90 and 99.9% (1 to 3 log units).

The WHO guidelines specify arithmetic mean values for the helminth eggs. It should be noted, however, that the arithmetic mean is not always the best measure of central tendency, especially in this case, where most of the effluent data have a value of zero, and only a few data have values greater than zero.

Cavalcanti et al (2001) and von Sperling et al (2001, 2002) comment that the removal of helminth eggs is assumed as being a process of discrete settling,

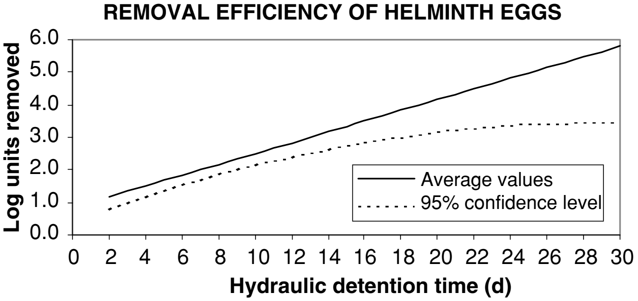


Figure 17.7. Removal efficiency of helminth eggs, expressed as logarithmic units removed, according to the model of Ayres et al (1992)

which, in theory, is associated with the hydraulic surface loading rate ($m^3/m^2.d$) and is independent of the depth. Total elimination of helminth eggs was obtained in pilot ponds in Brazil operating with surface loading rates between **0.12** and **0.20 m^3/m^2.d**. The more conservative loading rate of 0.12 m^3/m^2.d with a depth of 1.0 m corresponds to a hydraulic detention time of (1.0 m) / (0.12 m^3/m^2.d) = 8 d.

The WHO (1989) suggests that a series of ponds with total hydraulic detention times of **8 to 10 days** can produce on average effluents with less than 1 egg/litre.

According to Ayres equation (Equation 17.15, for average values), for 8 and 10 days of detention time, the removal efficiency is 2.17 and 2.50 log units (99.3% and 99.7%, respectively). In this case, mean effluent concentrations lower than 1 egg/L will be obtained if the influent has less than 150 to 300 eggs/L.

Figure 17.8 presents the distribution of helminth eggs in the raw wastewater, effluent from a UASB reactor and effluent from the first pond, obtained from five pond systems in Brazil (von Sperling et al, 2003). Some systems had only one pond, while others had ponds in series. It is seen that, already in the effluent from the first pond (or in some cases, the only pond), the eggs concentrations are mostly equal to zero or lower than 1 egg/L. It is worth commenting again that, given the high variability of the data, the *arithmetic mean* is not a good representation of the central tendency, because a few high values tend to increase substantially the average. After the first pond, the *median* values are systematically equal to zero. *Geometric means* may not be calculated, because the occurrence of a single zero value in the whole series leads to a geometric mean of zero, regardless of the other values.

17.6.2 Helminth eggs in the sludge

Research conducted in a baffled pilot pond in Brazil (von Sperling et al, 2002) presented various data of interest with relation to the eggs in the sludge. The settled eggs are incorporated in the bottom sludge, and tend to remain viable for a long period (Figure 17.9). Figure 17.10 presents the longitudinal profile of egg accumulation in the bottom sludge, showing the decreasing tendency along the various compartments of the baffled pond. Also presented are the values of

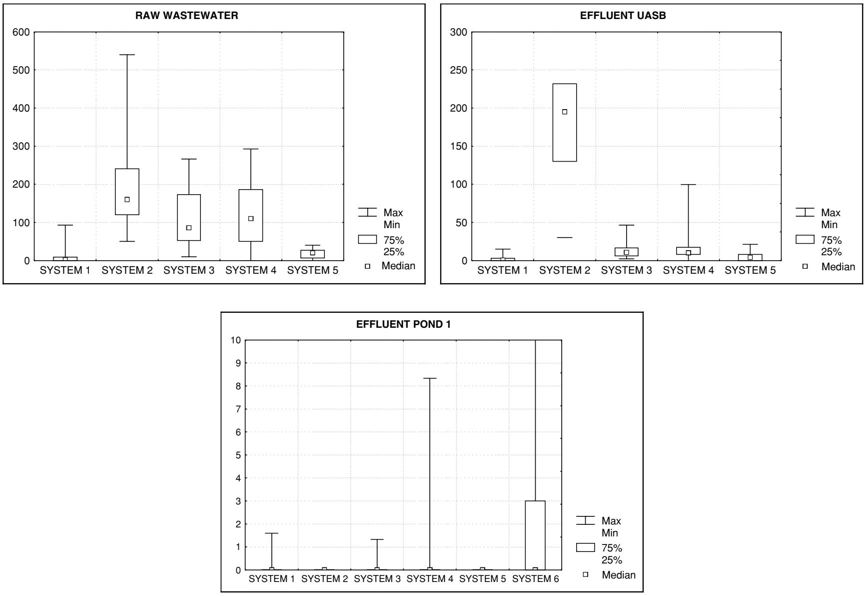


Figure 17.8. Box-and-whisker plot of the helminth eggs concentrations (eggs/L) in five systems in Brazil, monitored in the following points: influent, effluent from UASB reactors and effluent from first pond

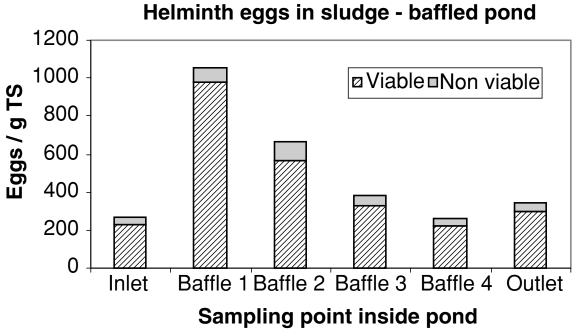


Figure 17.9. Distribution of helminth eggs in the sludge along a baffled pilot pond in Brazil, after one year of operation, with the indication of the viability and non-viability of the eggs.

the egg counts per gram of total solids, which is a unit usually used for sludge characterisation. Figure 17.11 shows the distribution of the species of helminth eggs in the sludge. It can be observed that the relative distribution is not substantially different along the length of the pond. In terms of the global values in the sludge, the

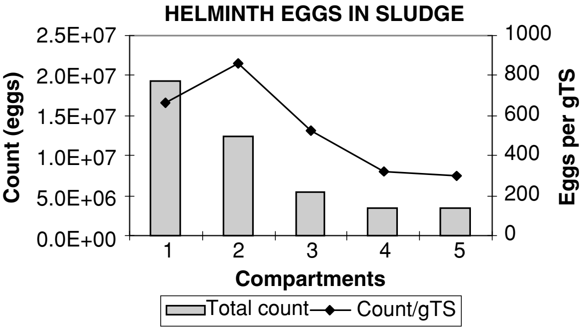


Figure 17.10. Longitudinal profile of the accumulation of helminth eggs in the sludge in a baffled pilot pond in Brazil, after one year of operation

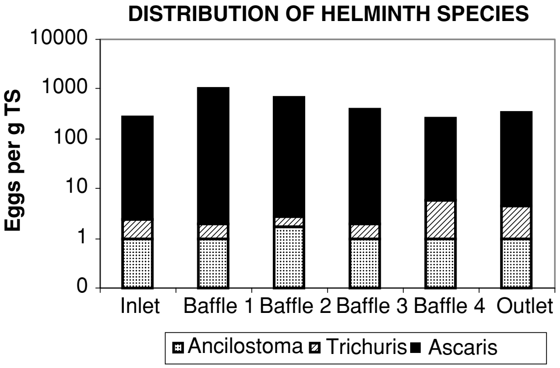


Figure 17.11. Distribution of the helminth species in the sludge along a baffled pilot pond in Brazil, after one year of operation

following relation was found: *Ascaris lumbricoides*: 99.1%, *Trichuris trichiura*: 0.8%, *Ancilostoma* sp.: 0.1%.

Example 17.3

Estimate the concentration of helminth eggs in the effluent from a system composed of facultative pond – baffled maturation pond (Examples 13.3 and 17.2), with the following characteristics:

Population = 20,000 inhab
Influent flow = 3,000 m^3/d
Concentration of helminth eggs in the raw sewage: 200 eggs/L (assumed)

***Example 17.3* (*Continued*)**

Hydraulic detention time in the facultative pond: t = 28.8 d
Hydraulic detention time in the baffled maturation pond: t = 12.0 d

Solution:

a) Removal of helminth eggs in the facultative pond

For design purposes, the removal efficiency of helminth eggs in the facultative pond is given by Equation 17.16:

$$\begin{aligned} E &= 100 \times \left[1 - 0.41.e^{(-0.49.t+0.0085.t^2)}\right] \\ &= 100 \times \left[1 - 0.41.e^{(-0.49\times 28.8+0.0085\times 28.8^2)}\right] = 99.965\% \end{aligned}$$

This value is naturally in agreement with the value presented in Table 17.8.

The concentration of eggs in the effluent from the facultative pond is:

$$C_e = C_o \times (1 - E/100) = 200 \times (1 - 99.965/100) = 0.07\text{eggs/L}$$

The effluent from the facultative pond already complies with the guidelines of the WHO for restricted and unrestricted irrigation (1 egg/L).

b) Removal of helminth eggs in the maturation pond

Again, for design purposes, the removal efficiency of helminth eggs in the maturation pond is given by Equation 17.16:

$$\begin{aligned} E &= 100 \times \left[1 - 0.41.e^{(-0.49.t+0.0085.t^2)}\right] \\ &= 100 \times \left[1 - 0.41.e^{(-0.49\times 12.0+0.0085\times 12.0^2)}\right] = 99.61\% \end{aligned}$$

This value is of course the same as that from Table 17.8.

The concentration of eggs in the effluent from the maturation pond (final effluent of the system) is:

$$C_e = C_o \times (1 - E/100) = 0.07 \times (1 - 99.61/100) = 2.7 \times 10^{-3}\text{eggs/L}$$

This value corresponds, in practical terms, to a concentration of zero in the effluent.

18

Nutrient removal in ponds

18.1 NITROGEN REMOVAL

The main mechanisms of *nitrogen* removal in stabilisation ponds are (Arceivala, 1981; EPA, 1983; Soares et al, 1995):

- ammonia stripping
- ammonia assimilation by algae
- nitrate assimilation by algae
- nitrification–denitrification
- sedimentation of the particulate organic nitrogen

Of these mechanisms, the most important is **ammonia stripping**, that is, its release to the atmosphere. In the liquid medium, the ammonia presents itself according to the following equilibrium reaction:

$$\boxed{NH_3 + H^+ \leftrightarrow NH_4{}^+} \qquad (18.1)$$

The free ammonia (NH_3) is susceptible to stripping, while the ionised ammonia cannot be removed by stripping. With the rise of the pH, the equilibrium of the reaction is shifted to the left, favouring the larger presence of NH_3. For 20 °C, in a pH around neutrality, practically all the ammonia is in the form of $NH_4{}^+$. In a pH close to 9.5, approximately 50% of the ammonia are in the form of NH_3 and 50% in the form of $NH_4{}^+$. In a pH greater than 11, practically all the ammonia is in the form of NH_3 (see Section 2.2.3.4).

Biological Wastewater Treatment in Warm Climate Regions by Marcos von Sperling and Carlos Augusto de Lemos Chernicharo. ISBN: 1 84339 002 7. Published by IWA Publishing, London, UK.

The photosynthesis that takes place in the facultative and maturation ponds contributes to the increase of the pH, through the removal from the liquid of CO_2, that is, carbonic acidity. In conditions of high photosynthetic activity, the pH can rise to values higher than 9.0, providing conditions for the stripping of the NH_3. In addition, under high photosynthetic activity, the high algal production contributes to the direct consumption of NH_3 by the algae (Arceivala, 1981).

The stripping mechanism tends to be more important in *maturation ponds*, which, as a result of their low depths and consequent photosynthetic activity along the whole water column, usually have very high pH values. Additionally, in maturation ponds, the release of oxygen bubbles in the supersaturated liquid phase can accelerate the release of NH_3 (van Haandel and Lettinga, 1994).

In maturation ponds in series, the ammonia removal efficiency can be between 70 and 80%, and in especially shallow maturation ponds it can be greater than 90%, eventually leading to effluent values lower than 5 mg/L of ammonia (van Haandel and Lettinga, 1994; Soares et al, 1995). In facultative and aerated ponds, nitrogen removal efficiency is between 30 and 50%.

The loss of nitrogen through its *assimilation by the algae*, and consequent exit with the effluent is of a smaller importance, in case high removal efficiencies are desired. The nitrogen constitutes around 6 to 12%, in dry weight, of the cellular material of the algae (Arceivala, 1981). Assuming a concentration of 80 mg/L of algae in the effluent, the nitrogen loss will be $0.06 \times 80 \approx 5$ mgN/L to $0.12 \times 80 \approx 10$ mgN/L. Assuming a TKN (ammonia + organic nitrogen) level in the influent in the order of 50 mgN/L, the percentage removal through loss with the final effluent is between 10 and 20%.

The other nitrogen removal mechanisms act simultaneously, but they are considered of less importance. Nitrification is not very representative in facultative and aerated ponds. There is naturally no ammonia oxidation reaction in anaerobic ponds, due to the absence of oxygen.

The literature presents some equations developed in North America for the estimation of the effluent *ammonia* (Equations 18.2 and 18.3) and *nitrogen* (Equations 18.4 and 18.5) concentrations.

Ammonia removal (Pano and Middlebrooks, 1982):

$T < 20\,^\circ C$:

$$\boxed{C_e = \frac{C_o}{1 + [(A/Q).(0.0038 + 0.000134.T).e^{(1.041+0.044.T).(pH-6.6))}]}} \qquad (18.2)$$

$T \geq 20\,^\circ C$:

$$\boxed{C_e = \frac{C_o}{1 + [5.035 \times 10^{-3}.(A/Q).e^{(1.540\times(pH-6.6))}]}} \qquad (18.3)$$

Nitrogen removal (WPCF, 1990; Crites and Tchobanoglous, 2000):

Facultative ponds with a hydraulic regime closer to plug flow:

$$\boxed{C_e = C_o.e^{\{-K.[t+60.6\times(pH-6.6]\}}} \tag{18.4}$$

$K = 0.0064 \times 1.039^{(T-20)}$

Facultative ponds with a hydraulic regime closer to complete mix:

$$\boxed{C_e = \frac{C_o}{1+[t.(0.000576T-0.00028).e^{(1.08-0.042\times T).(pH-6.6))}]}} \tag{18.5}$$

where:

C_o = influent concentration (mg/L)
C_e = effluent concentration (mg/L)
Q = influent flow (m^3/d)
A = surface area of the pond (m^2)
T = temperature of the liquid (°C)
pH = pH in the pond
t = hydraulic detention time in the pond (d)
K = removal coefficient (d^{-1})

The appropriate equation should be applied sequentially in each pond of the series, in order to lead to the value of the concentration in the final effluent.

Equations 18.2 and 18.3 do not lead to a continuous solution for temperatures lower and greater than 20 °C. The use of Equation 18.2 for values of T close to 20 °C leads to effluent concentration values lower than those from Equation 18.3. Regarding Equation 18.5, it can be observed that it is not very sensitive to variations in the values of pH and T.

The use of the above equations assumes the knowledge of the pH value, a variable that is not known in the design phase. The references above also present the following equation that can be used for the estimation of the pH in the pond, as a function of the alkalinity of the influent sewage:

$$pH = 7.3\,e^{(0.0005.alk)} \tag{18.6}$$

where:

alk = alkalinity in the influent sewage ($mgCaCO_3/L$)

However, Equation 18.6 does not take into consideration the depth of the pond. It is known (Cavalcanti et al, 2001) that the lower the pond depth, the larger the penetration of the light energy along the water column, photosynthetic activity, consumption of carbonic acidity and rise in the pH. In maturation ponds, pH values higher than those predicted by Equation 18.6 can be reached.

Tables 18.1 and 18.2 and Figures 18.1 and 18.2 respectively present the ammonia and nitrogen removal efficiencies, based on the use of Equations 18.3 and 18.4, for a temperature of 20 °C. For a temperature of 25 °C, Equation 18.3 leads to the same ammonia removal efficiencies, while Equation 18.4 increases the nitrogen

Table 18.1. Ammonia removal efficiency as a function of the Hydraulic Loading Rate (Q/A) and the pH ($T \geq 20°C$)

Q/A	Ammonia removal efficiency (%)				
($m^3/m^2.d$)	pH = 7.0	pH = 7.5	pH = 8.0	pH = 8.5	pH = 9.0
0.025	27	45	63	79	89
0.050	16	29	47	65	80
0.075	11	21	37	56	73
0.100	9	17	30	48	67
0.125	7	14	26	43	62
0.150	6	12	22	39	57

Removal efficiency calculated according to Equation 18.3

Table 18.2. Nitrogen removal efficiency as a function of the hydraulic detention time (t) and the pH ($T = 20$ °C)

	Nitrogen removal efficiency (%)				
t(d)	pH = 7.0	pH = 7.5	pH = 8.0	pH = 8.5	pH = 9.0
3	16	31	43	53	61
5	17	32	44	54	62
10	20	34	46	55	63
15	22	36	47	57	64
20	25	38	49	58	65
30	29	42	52	61	67
40	34	45	55	63	69

Removal efficiency calculated according to Equation 18.4

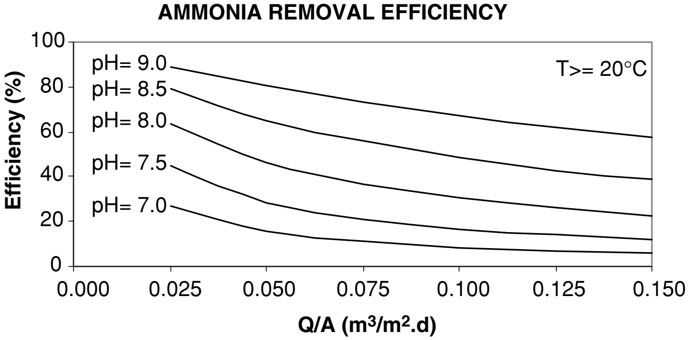


Figure 18.1. Ammonia removal efficiency as a function of the Hydraulic Loading Rate (Q/A) and the pH ($T \geq 20$ °C) (values from Table 18.1)

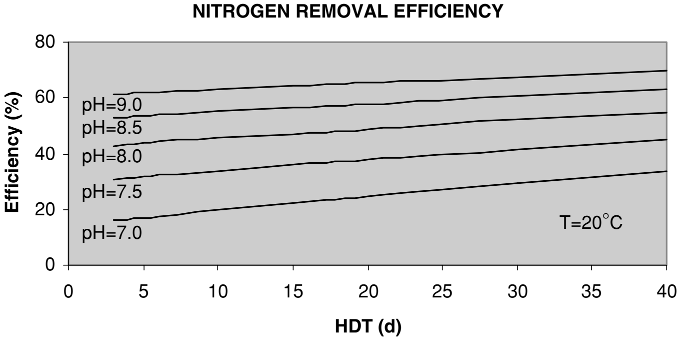


Figure 18.2. Nitrogen removal efficiency as a function of the hydraulic detention time (t) and the pH (T = 20 °C) (values from Table 18.2)

removal efficiency between 3 and 7%, when compared with the temperature of 20 °C.

Example 18.1

Estimate the ammonia and nitrogen removal in the facultative pond of Example 13.3, whose data are:

- Influent flow: Q = 3,000 m^3/d
- Surface area: A = 48,000 m^2
- Hydraulic detention time: t = 28.8
- Temperature: T = 23 °C (liquid in the coldest month)

The data assumed for the influent are:

- Ammonia = 30 mg/L
- Total nitrogen = 45 mg/L
- Alkalinity: 150 mg/L

Solution:

a) Ammonia removal

A/Q ratio (reciprocal of the hydraulic loading rate): A/Q = (48,000 m^2) / (3,000 m^3/d) = 16 d/m

(hydraulic loading rate Q/A = 1/16 = 0.0625 $m^3/m^2.d$)

pH in the pond (Equation 18.6):

$$pH = 7.3\ e^{(0.0005.alk)} = 7.3.\ e^{(0.0005\times 150)} = 7.87$$

***Example 18.1* (*Continued*)**

Effluent ammonia concentration (Equation 18.3):

$$\begin{aligned} C_e &= \frac{C_o}{1 + [5.035 \times 10^{-3}.(A/Q).e^{(1.540\times(pH-6.6))}]} \\ &= \frac{30}{1 + [5.035 \times 10^{-3} \times 16 \times e^{(1.540\times(7.87-6.6))}]} \\ &= 19.1\,\text{mg/L} \end{aligned}$$

Ammonia removal efficiency:

$$E = 100 \times (C_o - C_e)/C_o = 100 \times (30 - 19.1)/30 = 36\%$$

This efficiency is in agreement with Table 18.1 and Figure 18.1. The calculations above took into account only the ammonia present in the raw sewage, without considering the fact that a large fraction of the organic nitrogen will be converted into ammonia in the pond itself.

b) Nitrogen removal

Coefficient K:

$$K = 0.0064 \times 1.039^{(T-20)} = 0.0064 \times 1.039^{(23-20)} = 0.0072\,d^{-1}$$

Effluent nitrogen concentration (Equation 18.4):

$$C_e = C_o.e^{(-K.[t+60.6\times(pH-6.6)]\}} = 45 \times e^{\{-0.0072.[28.8+60.6\times(7.87-6.6)]\}} = 21.0\,\text{mg/L}$$

Nitrogen removal efficiency:

$$E = 100 \times (C_o - C_e)/C_o = 100 \times (45 - 21.0)/45 = 53\%$$

This efficiency is in agreement with Table 18.2 and Figure 18.2.

18.2 PHOSPHORUS REMOVAL

The phosphorus present in the sewage is composed of organic phosphorus and phosphates, with the latter representing the greatest fraction. The main mechanisms of phosphorus removal in stabilisation ponds are (Arceivala, 1981; van Haandel and Lettinga, 1994):

- removal of the *organic phosphorus* contained in the algae and bacteria through its exit with the final effluent
- precipitation of *phosphate* under high pH conditions

The organic phosphorus composes part of the cellular material of the algae. In dry weight, the phosphorus corresponds to values around 1.0% of the algae mass (Arceivala, 1981). Therefore, assuming a concentration of 80 mg/L of algae in the effluent, the phosphorus loss will be around $0.01 \times 80 \approx 0.8$ mgP/L. Admitting a phosphorus concentration in the influent around 8 mgP/L, the percentage removal through loss with the final effluent is only about 10%.

More substantial phosphorus removal can occur through the precipitation of the phosphates under high pH conditions. The phosphates can precipitate in the form of hydroxyapatite or struvite. The same considerations made in Section 18.1 are valid here, emphasising the relation between shallow ponds and high pH values. In the case of phosphorus removal, the dependence of high pH values is larger than with nitrogen: the pH should be at least 9 so that there is a significant phosphorus precipitation. In especially shallow ponds with low hydraulic loading rates, the phosphorus removal is between 60 and 80% (Cavalcanti et al, 2001), while in facultative and aerated ponds, the removal efficiency is usually lower than 35%.

19

Ponds for the post-treatment of the effluent from anaerobic reactors

Anaerobic sewage treatment, and especially the anaerobic sludge blanket reactor (UASB reactor), has grown in popularity and accessibility in many warm-climate countries. Anaerobic reactors reach a good level of efficiency in the removal of BOD (around 60 to 80%), considering the low detention times, the simplicity of the process and the non-existence of equipment, such as aerators. However, this efficiency is most of the time insufficient, bringing about the need for a post-treatment of the anaerobic effluent (see Chapter 29). The post-treatment can aim at one or some of the following items:

- additional BOD removal
- nutrient removal
- pathogenic organism removal

A very attractive post-treatment alternative is represented by stabilisation ponds, because they maintain in the system the conceptual simplicity already assumed for the anaerobic reactors. This approach of combining anaerobic sludge blanket reactors with stabilisation ponds is believed to have an extremely wide application for developing and warm-climate countries.

The non-mechanised ponds that receive the anaerobic reactor effluent have been designated as **polishing ponds**, to differentiate between the classic concepts of facultative and maturation ponds.

Catunda et al (1994) and Cavalcanti et al (2001) argue that, owing to the BOD removal that takes place in the anaerobic reactors, *the anaerobic effluent can be*

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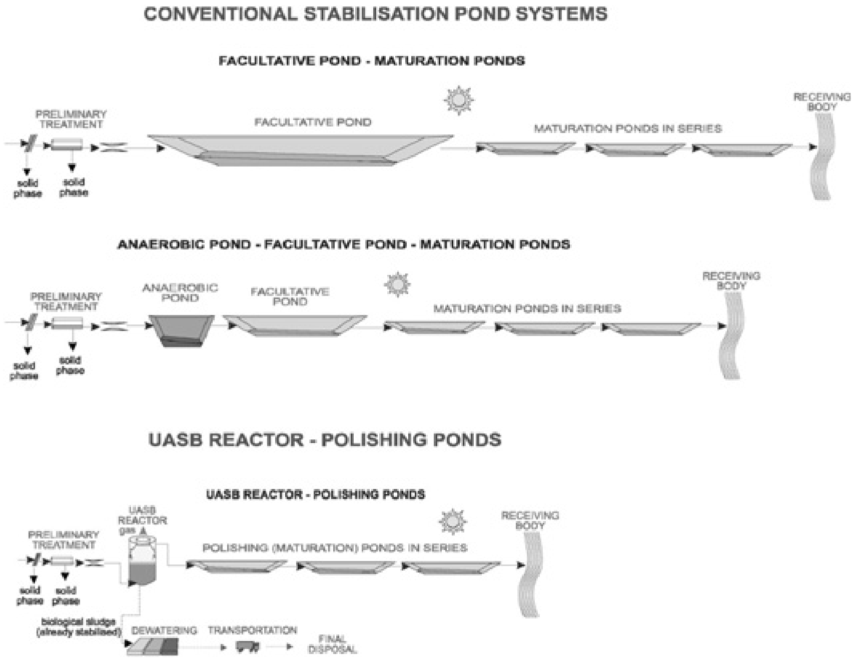

Figure 19.1. Comparison between classical stabilisation pond configurations and the more recent approach of UASB reactors followed by polishing ponds

discharged directly into baffled or in-series polishing ponds, without problems of organic overloading in the first pond of the series or in the initial compartment of the baffled pond. This statement is endorsed by experience acquired in the operation of several polishing ponds in Brazil, as part of PROSAB (Brazilian Research Programme on Basic Sanitation). These pond configurations optimise coliform removal, as commented in Chapter 17. Therefore, the evidence currently available suggests that polishing ponds do not need to be designed as classic facultative ponds, but as maturation ponds (using the design approaches of maturation ponds, regarding the geometric configuration, detention time and depth – see Chapter 17). However, unlike maturation ponds, they not only provide an excellent pathogen removal, but also contribute in a further removal of BOD (hence the name "polishing").

Figure 19.1 presents a comparison of the classical pond configurations with the recent approach of UASB reactors followed by polishing ponds. A significant advantage of this system is the saving in land requirements. It should be understood that the UASB reactor is not simply replacing the anaerobic lagoon, but also the facultative pond.

a) Additional BOD removal

In relation to an additional BOD removal from the anaerobic effluent, this objective can be well accomplished by unaerated ponds or aerated ponds. The first alternative is the most attractive, since it allows a system without mechanisation and with a very low amount of sludge to be treated. The design of the ponds is now for a load of around 20% to 40% of the load of the raw sewage. The land savings are substantial and can make the implementation of ponds possible in locations where mechanised systems would have been previously the only choice. Also in the cases in which the earth movement associated with the construction of a conventional ponds system is excessive, the inclusion of a compact unit such as the anaerobic reactor can contribute to a substantial reduction in the construction costs.

Systems working with this configuration have shown the following characteristics (Cavalcanti et al, 2001):

- absence of mal-odour problems in the ponds (even under high organic load conditions)
- low sludge accumulation in the ponds
- possibility of the use of ponds in series or baffled (without problems of organic overloading in the first pond of the series or in the first compartment of the baffled pond)

The BOD removal coefficients (K) are slightly lower than those of primary facultative ponds, because the stabilisation ponds are already receiving a partially treated influent, in which the easily degradable organic matter has been already removed. However, the coefficients are similar to those used for secondary facultative ponds, following anaerobic ponds.

b) Nutrient removal

Anaerobic treatment systems practically do not remove nutrients. If a high nutrient removal efficiency is required, it should be kept in mind that stabilisation ponds (facultative and aerated) are also not particularly efficient in the removal of N and P. However, polishing (and maturation) ponds can play relatively well this additional role, mainly through the volatilisation of ammonia and the precipitation of phosphates (see Chapter 18).

Ammonia and phosphate removal is greater in polishing ponds with lower depths (less than 1.0 m). In these ponds, the liquid tends to have high pH values, due to the intense photosynthesis that takes place in all the pond volume. The high pH values allow the volatilisation of the free ammonia and the precipitation of the phosphates.

c) Removal of pathogenic organisms

Owing to the low detention times in the anaerobic reactors (in the order of hours), the removal of pathogenic organisms is low in these units (around 1 log unit of coliforms). In this sense, stabilisation ponds, and mainly maturation ponds, can substantially contribute to a high removal efficiency. In the context of post-treatment of anaerobic effluents, the polishing ponds play this role very well, and this is one of their main purposes.

The coliform die-off coefficients are of the same order of magnitude (or maybe slightly higher, owing to receiving a more clarified influent) from those obtained

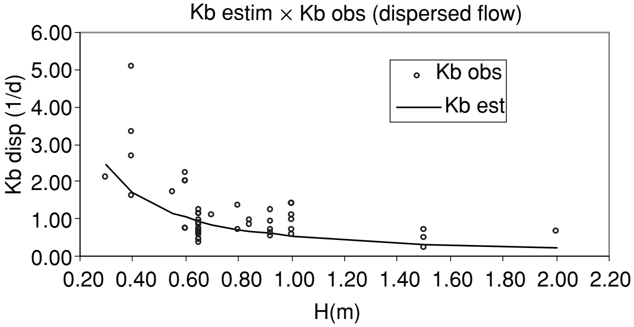

Figure 19.2. Values of the K_b coefficient (dispersed flow) obtained in 19 polishing ponds in Brazil ($n = 45$), together with the plot of equation $K_b = 0.542.H^{-1,259}$, based on facultative and maturation ponds in the world (dispersion number $d = 1/(L/B)$)

in facultative and maturation ponds. Figure 19.2 shows K_b values (dispersed flow) obtained in 19 polishing ponds (45 data) in 7 different UASB-polishing pond systems in Brazil (von Sperling et al, 2003b). Equation 17.15 ($K_b = 0.542.H^{-1.259}$), based on 82 facultative and maturation ponds in the world, is also plotted, showing a reasonable fitting to the data. Visually, it is seen that most of the ponds have K_b values slightly higher than those predicted by the overall equation. It can also be seen that shallow ponds ($H < 1.0$ m) have very high K_b values.

With respect to the removal of helminth eggs, polishing ponds have also been shown to be efficient, similarly to maturation ponds. Effluents with arithmetic means lower than 1 egg per litre are easily achievable and, in most cases, zero counts in the effluent are obtained.

20

Construction of stabilisation ponds

20.1 INTRODUCTION

The operational success of stabilisation ponds depends not only on the process aspects discussed in the previous chapters, but also on the design detailing and on the construction aspects. In general terms, the aspects associated with earthmoving are of fundamental importance, and are likely to have a decisive influence on the economy of the plant. In a more specific aspect, the several details regarding inlet, outlet and interconnection between units are also very important as they have a direct impact on the hydraulic behaviour of the ponds. The detailing aspects should also be considered from the point of view of the operator's needs, in order to make the operational routine of the plant as simple and easy as possible.

This text does not have the objective of furthering the detailing aspects, and other textbooks should be consulted for this purpose. The following topics emphasise just the most important aspects, to which designers and those in charge of the project should give special attention.

20.2 LOCATION OF THE PONDS

The main aspects that should be analysed in selecting the area for the future pond are presented in Table 20.1 (Arceivala, 1981; Silva, 1994).

The simultaneous compliance with the various criteria is usually very difficult, and priority should be given, in each case, to factors of larger importance, which shall be observed according to the local reality.

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Table 20.1. Aspects related to the location of the ponds

Aspect	Comment
Area availability	The availability can lead to the selection of the type of pond to be adopted
Location of the area in relation to the wastewater generation location	The closer the pond, the lower the wastewater transportation costs
Location of the area in relation to the receiving body	The closer the pond, the lower the transportation costs of the treated wastewater to the location of its final disposal
Location of the area related to the nearest residences	Anaerobic ponds require a minimum distance of approximately 500 m from the nearest residences, in view of possible bad odours; the other ponds can be located at a shorter distance from the residences
Flood levels	It should be verified whether the land is floodable and the maximum flood levels, for definition of the height of the embankments
Level of the groundwater	The level of the groundwater can determine the settlement level of the ponds and the need to waterproof their bottom
Topography of the area	The topography of the area has a large influence on the earthmoving and, consequently, on the cost of the plant; little sloped areas are preferable
Shape of the area	The shape of the area influences the arrangement of the various units in the floor. In order to save in earthmoving, the shape of the contours can be utilised (provided they are smooth, thus avoiding the creation of dead zones)
Characteristics of the soil	The type of soil has a large influence on the planning of the compensation between cut and fill, on the need of borrow material, on the inclination of the slopes, on the costs of the works (e.g. stones), and on the need of an impermeable bottom
Winds	The location of the pond should permit free wind access, which is important to guarantee a smooth mixing in the pond
Access conditions	Access of the construction teams and of the future operation and maintenance teams should not be difficult
Facility to purchase land	Expropriation difficulties can be an element that affects the feasibility of the pond location in the desired area
Cost of the land	In urban areas or in areas near towns or some important element, the cost of land can be very high, which may lead to the need to adopt more compact treatment processes

With relation to the positioning of the pond in the land, a great effort should be made in the design stage to minimise earthmoving, based on the local topography and geology. Another influencing factor in the location is the direction of the predominant winds. In order to allow a smooth mixing by the wind, *the longest dimension of the pond should be towards the predominant winds*. Should the direction of the winds change seasonally, priority should be given to the direction

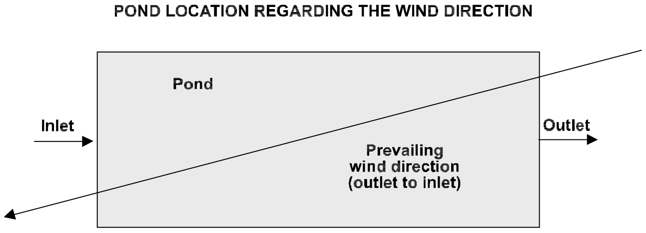


Figure 20.1. Location of the pond in relation to the direction of the predominant winds

of the wind in the warm period, when thermal stratification is larger. To reduce hydraulic short circuits, *the direction of the wind should be from the outlet to the inlet of the pond* (Mara et al, 1992) (Fig. 20.1).

20.3 DEFORESTATION, CLEANING AND EXCAVATION OF THE SOIL

Deforestation comprises cutting and removal of the trees existing in the area to be occupied by the pond and access roads. The removed material should be moved away from the work site. The cutting of trees shall be approved by the environmental agency.

After the cutting of the trees, the small-sized vegetation is stumped and removed, being, most of the times, burnt in the location itself. The surface of the soil is then raked by motor graders, until the area becomes pure soil (Silva, 1993).

Two different situations may take place in the excavation of the pond (Silva, 1993):

- *Usable excavated material*. This is the desirable situation, in which the cut volumes and the embankment volumes (dikes) are balanced, in order to minimise earthmoving. A large part of the economic feasibility of the construction of a stabilisation pond is associated with the possibilities of earthmoving minimisation. The bottom and the dikes should be compacted in successive earth layers, with humidity and compaction control.
- *Useless excavated material*. This is the case of very sandy soils or those with large quantities of organic matter (peat). In these conditions, the material removed by excavation cannot be used in the construction of the dikes and should be moved away from the location. Borrow material from good quality soil existing near the location should be used for the embankments. After finishing the excavation, the bottom of the pond and the slopes need to be scarified, so that the soil is closely linked to the material used.

20.4 SLOPES

The dikes of the pond are formed by the internal slopes (in contact with the liquid in the pond) and by the external slopes. The aspects listed in Table 20.2 are

Table 20.2. Construction aspects of the pond dikes

Item	Comment
Internal slope	• Usual slope: 1:2 to 1:3 (vert/horiz) • Minimum slope: 1:6 (avoid shallow areas, which allow the growth of vegetation) • Maximum slope: 1:2 (due to land stability) • Clayey soils: slope greater than 1:2 • Sandy soils: slope between 1:3 and 1:6
External slope	• Usual slope: 1:1.5 to 1:2 • Clayey soils: slope greater than 1:2.5 • Sandy soils: slope between 1:5 and 1:8
Slope crest (lane at the crest of the slope)	• Wider than 1.5 m; usually between 2.0 and 4.0 m, in order to allow traffic of the machines during the construction, movement of the maintenance and operation teams, and a possible increase in the height of the dike, if necessary
Freeboard	• Small ponds (<1 ha of area): adopt 0.5 m • Ponds between 1 ha and 3 ha: 0.5 to 1.0 m • Larger ponds: $\textit{free board} = [\log\ (\textit{pond area})]^{0.5} - 1$ (area in m^2) • Purposes: safety in case of increased water level exceeding the design conditions (outlet obstruction, effect of strong winds, new design conception) and safety in case of land settlement due to a contingent lowering of the dike
Waterproofing	• Should the dike material be extremely permeable, it may be necessary to waterproof the dike embankment with clay, geomembranes, sheet piles or concrete slabs • After compaction, the coefficient of permeability should be $<10^{-7}$ m/s
Protection of the internal slopes	• The internal slopes in contact with the water level should be protected against waves, erosion and vegetation growth • The growth of vegetation enables the development of mosquitoes in the ponds (eggs laid in the water and in the shadow of the vegetation) • The types of protection more commonly employed are: large stones (15 to 20 cm), slightly-reinforced concrete slabs (thickness between 7 and 13 cm), concrete plates, reinforced mortar, asphalt pavement, soil cement or plastic membrane • Discontinuous protection (such as stones) enables the growth of vegetation • The protection should extend for at least 0.4 m above and 0.4 m below the water level • Grass or crushed stone should be placed on the rest of the slope over the protection
External slope	• The external slope should be grassed to provide protection against erosion
Corners of the slopes	• The corners of the ponds should be slightly rounded to facilitate the construction and maintenance, and avoid small dead zones
Material of the slopes	• The dikes should be constructed with soil, preferably from the occupied land itself. • The material should be dense, fine, cohesive, and well granulated. • It shall consist of (a) clean soil, without stones and organic matter, and (b) of clay with a little sand
Stormwater drainage	• In ponds that have a side made up of a natural slope (e.g. hill), the stormwater should be collected by ditches parallel to this side, thus preventing the stormwater from passing over the slope

Source: Arceivala (1981), Mara et al (1992), Silva (1993), Jordão and Pessôa (1995)

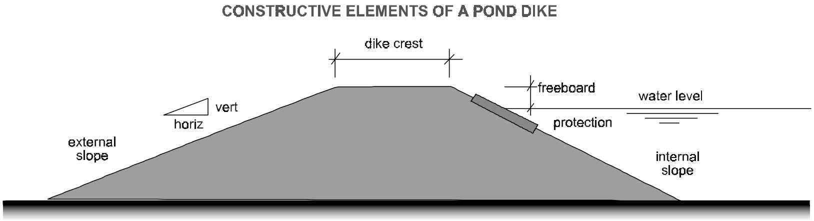


Figure 20.2. Main elements of a pond dike

important in the construction of the slopes (Silva, 1993; Jordão and Pessôa, 1995). Figure 20.2 lists the main elements of a pond dike.

The length and width determined in the pre-dimensioning are at *mid depth*. The dimensions of the ponds at the bottom, at the water level (WL) and at the crest of the slope depend on the inclination of the internal slope. Assuming that the internal embankment has a slope of 1:d (vertical/horizontal), the referred to dimensions will be:

Length:

- length at the bottom = length at mid depth − 2d.(H/2)
- length at the water level = length at mid depth + 2d.(H/2)
- length at the crest of the slope = length at water level + 2d.(free board)

Width:

- width at the bottom = width at mid depth − 2d.(H/2)
- width at the water level = width at mid depth + 2d.(H/2)
- width at the crest of the slope = width at water level + 2d.(free board)

Example 20.1

Calculate the total dimensions of a pond that has the following dimensions determined in the preliminary design:

- length (at mid depth) = 100.00 m
- width (at mid depth) = 30.00 m
- depth = 2.20 m
- freeboard = 0.60 m
- internal slope = 1:2.5

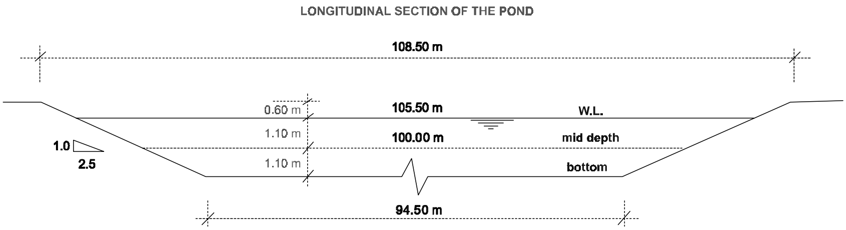
Solution:

According to the concepts and formulas above:
Internal slope = 1:2.5 → d = 2.5

- length at the bottom = length at mid depth – d.H = 100.00 – 2.5 × 2.20 = 94.50 m
- length at WL = length at mid depth + d.H = 100.0 + 2.5 × 2.20 = 105.50 m

***Example 20.2* (*Continued*)**

- length at the crest of the slope = length at the WL + 2d.(freeboard) = $105.50 + 2 \times 2.5 \times 0.60 = 108.50$ m
- width at the bottom = width at mid depth – d.H = $30,00 - 2.5 \times 2.20 = 24.50$ m
- width at WL = width at mid depth + d.H = $30.0 + 2.5 \times 2.20 = 35.50$ m
- width at the crest of the slope = width at WL + 2d.(freeboard) = $35.50 + 2 \times 2.5 \times 0.60 = 38.50$ m



20.5 BOTTOM OF THE PONDS

The bottom of the stabilisation ponds should not lead to excessive seepage, which could cause one of the following problems:

- contamination of the groundwater
- difficulty in maintaining the liquid level in the ponds

The permeability of the soil and the possible interference with the groundwater should be investigated by means of boreholes. It is worth to mention that the sites usually available for possible construction of wastewater treatment plants are frequently located in swamps, marshy areas or with a high groundwater level. The permeability of the bottom tends to decrease as time goes by, as a result of the clogging caused by solids from the sewage and by the biomass. According to Arceivala (1981), under favourable conditions, the losses by infiltration amount to less than 10% of the flow from the pond, being frequently lower than 1%.

Mara et al (1992) propose the following interpretations of the coefficient of permeability k:

- $k > 10^{-6}$ m/s: the soil is very permeable and the bottom should be protected
- $k > 10^{-7}$ m/s: some infiltration may occur, but not enough to prevent the pond from being filled
- $k < 10^{-8}$ m/s: the bottom of the pond will be naturally sealed
- $k < 10^{-9}$ m/s: there is no risk of contamination
- $k > 10^{-9}$ m/s: if the groundwater is used for domestic supply, hydrogeological studies should be performed

A *reduced percolation rate* can be achieved by means of a well-compacted 5–10 cm thick homogeneous clay layer. The *waterproofing* of the bottom can be accomplished through (Jordão and Pessôa, 1995):

- clay layer, with a minimum thickness of 40 cm
- asphalt coating
- plastic geomembranes

The solution to be adopted will naturally have a great impact on the total cost of the pond and on its own economic feasibility.

Should clay be adopted, after preparing the bottom, one should not wait too long for the filling of the pond with liquid (although partial, 1/3 of the height), in order to prevent drying and cracking of the bottom layer.

When estimating the required clay volume, it should be remembered that the clay, after compaction, has its volume reduced. Thus, the clay volume to be acquired should be greater than the volume of the bottom layer.

In aerated ponds, the placement of a concrete plate underneath each aerator is necessary in order to avoid erosion problems caused by the turbulence generated by the aerator.

The pond bottom should be made as level as possible, unless there is an specifically designed hopper near the inlet to retain settleable solids. The finished elevation should not vary more than 15 cm from the average elevation of the bottom (Metcalf & Eddy, 1991).

20.6 INLET DEVICES

The influent wastewater should undergo a preliminary treatment consisting of:

- *Screen.* The screens are usually manually cleaned in most of the ponds. The adoption of mechanised screens is justifiable in ponds of large dimensions or in special situations.
- *Grit chamber*. Although the amount of sand is relatively small compared with all the sludge volume accumulated at the bottom of the pond, the sand tends to settle close to the inlet, which may cause localised problems. As the grit chamber is a small unit of easy maintenance, its inclusion is recommended in all pond systems.
- *Flume or weir for flow measurement.* The flow measurement is essential for the operational control of the pond. The flume also carries out the function of regulating the velocity in rectangular grit chambers. A convenient location for the collection of samples of the influent to the pond is close to the flow measurement unit.

The inlet to the pond should meet the following conditions:

- guarantee a broad homogenisation of the liquid, avoiding the occurrence of hydraulic short-circuits and dead zones
- be submerged, in order to avoid the release of malodorous gases
- avoid erosion of the slopes and the pond bottom (for this purpose, a concrete plate is placed on the bottom, underneath the pipe discharge)

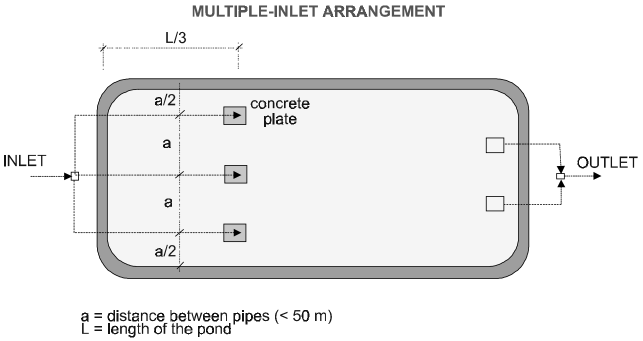


Figure 20.3. Distribution of the inlet pipes in a pond. Alternative of multiple inlets and outlets in wide ponds.

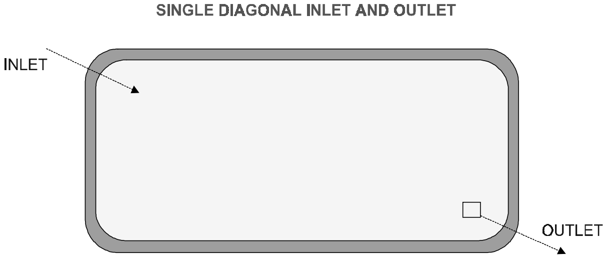


Figure 20.4. Distribution of the inlet and outlet pipes in a pond. Alternative of single inlets and outlets, located in diagonally opposite ends.

With relation to the number of inlet pipes, there are two approaches in the literature. One states that the homogeneous distribution of wastewater along the width of the pond should be guaranteed through the placement of a sufficient number of inlet pipes (**multiple inlets**). Only small ponds should have a single inlet. Larger ponds should have two, three, or more inlets, distant 50 m at the most one from the other (see Figure 20.3). The inlet should not be located in front of the outlet of the pond, even if at long distances, as this facilitates the occurrence of hydraulic short circuits.

Another approach (Mara et al, 1992) suggests, for simplicity, **single inlets and outlets** in each pond, located diagonally in opposite ends. The argument is that in the case of multiple inlet and outlet structures there may be differential settlements in the structures, altering the relative distribution of the flows (Fig. 20.4).

Should a hydraulic regime approaching that of *complete mix* be desired in the pond, the inlet pipes should extend to *1/4 to 1/3 of the length* of the pond (Figure 20.3). Should an approximation to a *plug-flow* reactor be desired, the inlet pipes should discharge *closer to the inlet side*. It is worth reminding that, for primary ponds, plug-flow conditions should be avoided as they can cause organic overload close to the inlet end. However, for single-cell maturation ponds, the plug-flow system is thoroughly more advantageous, and no overloading problems at the inlet end are expected.

The inlet tubing should be designed for an average flow velocity equal to or higher than 0.5 m/s (Silva, 1993).

In deeper ponds, dead zones are more likely to occur. In these ponds, the design of the inlets and outlets should be made very carefully. In anaerobic ponds, there are indications that a homogeneous distribution at the bottom by means of perforated laterals can contribute to a larger contact between the wastewater and the biomass, thus increasing the efficiency of the pond.

Figure 20.5 presents some types of inlet devices commonly used by designers (Jordão and Pessôa, 1995). Inlets right at the bottom may suffer blocking problems due to localised silting, in case grit removal is inefficient or the wastewater collection system receives large portions of stormwater, which may include substantial loads of sand.

By-pass tubing should be included, allowing the start-up of the facultative pond prior to the anaerobic pond, the interruption in the feeding to a certain pond during sludge removal or for any other operational or maintenance reason.

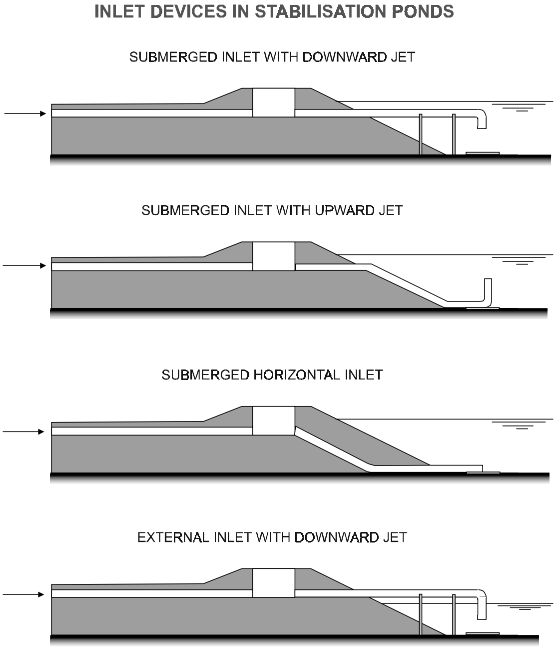


Figure 20.5. Different inlet schemes in stabilisation ponds (adapted from Jordão and Pessôa, 1995)

20.7 OUTLET DEVICES

The design of the effluent outlet of stabilisation ponds should take into consideration the following aspects (Mara et al, 1992; Silva, 1993; Jordão and Pessôa, 1995):

- the outlet should be located at the opposite end to the inlet, to avoid short circuits
- the outlet should not be aligned with the inlet, in order to minimise short circuits
- the outlet devices can be of either fixed or variable level (the latter is preferable as it allows more flexibility)
- the outlet should have baffles reaching below the water level, to prevent the exit of floating material, such as algae, in the facultative ponds, or scum, in the anaerobic ponds

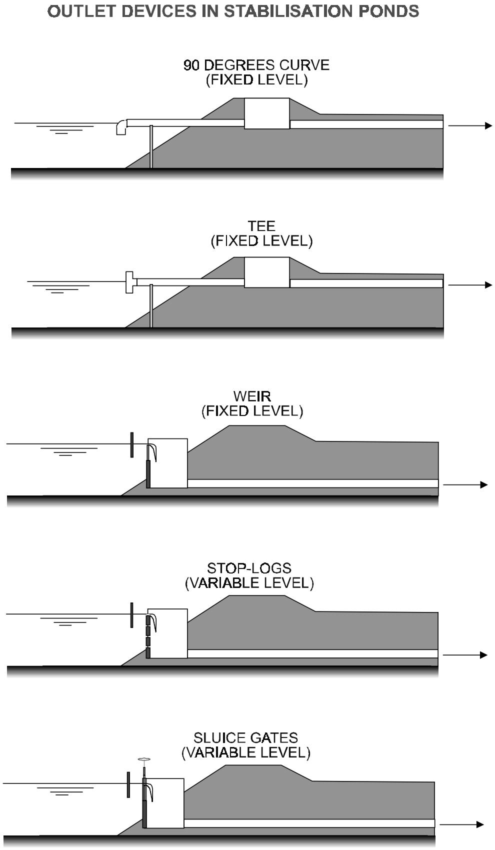


Figure 20.6. Some outlet devices for stabilisation ponds

- the effluent removal level, dictated by the baffles, has the following conflicting results: shallower removal – higher DO and SS contents; deeper removal – lower DO and SS contents
- approximate values for the effluent removal level shall have the following depths below the water level: anaerobic ponds: 0.30 m; facultative ponds: 0.60 m; maturation ponds: 0.05 m
- the design should allow operational flexibility to adjust the removal level below the water level, in order to achieve the desired point regarding the conflicting objectives between the DO and SS concentrations
- access to the outlet device should be easy, in order to allow flow measurements, collection of samples and changes in the pond water level
- a bottom discharge system can be adopted in the outlet structure itself (although its infrequent or unlikely use may make its utilisation difficult, after a long time of closure)

There are several types of outlet devices. Figure 20.6 illustrates some of these types, such as: (a) fixed water level: *90° curve*, cast iron *tee*, *weir* and (b) variable water level: wooden *stop-logs* (the placement or removal of the wooden boards allows variation in the level of the pond) and *sluice gates*. Stop-logs do not provide a good watertightness, and liquid may pass between the boards, although, in principle, this does not represent a great problem. Screw-driven sluice gates can be adopted, which allow variation of the water level, with a greater watertightness (although not complete).

When the effluent from a pond goes to another pond downstream, the interconnection between the ponds should include an open visiting *box*, in order to enable the collection of samples and the unblocking of pipes.

21

Maintenance and operation of stabilisation ponds

21.1 INTRODUCTION

The conceptual simplicity of stabilisation ponds brings as a consequence the simplicity of the operation and maintenance procedures. The ponds are inherently simple, and they should be designed to be so in their operational routine. In this simplicity lies the great sustainability of wastewater treatment by stabilisation ponds, mainly in developing countries. However, the operational simplicity should not be an excuse for a lack of care with the plant and the process. There are several operation and maintenance procedures that should be carried out following a certain routine, without which environmental problems and a reduced treatment efficiency treatment will take place.

The present chapter deals with the following aspects related to the operation and maintenance of the ponds:

- dimensioning of the operational staff
- inspection and monitoring
- start-up of operation
- operational problems

The coverage of these items in the book is very simple. The references WEF (1990), Mara et al (1992), Yanez (1993) or Jordão and Pessôa (1995) should be consulted for further details with relation to these topics.

It is essential that the design of the stabilisation pond includes an Operation Manual, providing the main guidelines for the suitable operation of the designed system. During the plant operation, the operator can seek the optimisation of the process, based on his accumulated experience in the pond at issue.

21.2 OPERATIONAL STAFF

In a stabilisation pond, most of the personnel are associated with simple maintenance activities, such as grass cutting, cleaning and others. The need for qualified technical personnel is low, compared with most of the other treatment processes. Table 21.1 presents some suggestions for structuring the operational staff with different pond sizes.

21.3 INSPECTION, SAMPLING AND MEASUREMENTS

The operator should carry out a daily inspection throughout the pond and its complementary units. Table 21.2 shows an example of an inspection checklist.

The sampling and measurement scheduling can follow the model presented in Table 21.3. Certainly, depending on the size and importance of the pond, the number of parameters to be included, as well as the frequency of their determination, can be altered and adapted to local needs. Small-sized ponds in remote and lower-income locations can naturally have a more simplified sampling scheduling, concentrated on the determination of the flow and parameters set forth by the environmental legislation. Should effluents be reused in agriculture, agronomic (electric conductivity, Ca, Mg, Na, boron and others) and sanitary (helminth eggs) parameters of interest should be investigated.

Owing to the daily variation of several constituents in stabilisation ponds, composite sampling is preferable. The portions that constitute the composite sample are collected either automatically (automatic samplers) or manually, at 1–3-hour intervals. Should there be any difficulty to collect the composite samples, the collection of a single sample from the water column in the pond leads to results comparable with those of the composite sample. Mara et al (1992) present details of the column sampler.

An aspect of fundamental importance in a monitoring programme relates to the real use of the data surveyed. There is no sense in obtaining data if they are not checked and interpreted. Pond performance monitoring graphs should be produced, with participation of the operator. Data input in computer spreadsheets in the head office, including loading rates, efficiency parameters and associated graphs, is the best form to use these data.

21.4 OPERATION START-UP

21.4.1 Loading of the ponds

The initial loading of the ponds can be done by means of one of the two procedures described below (CETESB, 1989). The loading should be performed preferably in summer, when temperatures are higher.

Table 21.1. Dimensioning of the staff for administration, operation, and maintenance of stabilisation ponds, as a function of the population served

	Pop $\leq$ 10,000 inhab.		Pop = 20,000 to 50,000 inhab.		Pop > 50,000 inhab.	
Personnel	Facultative pond	Aerated pond	Facultative pond	Aerated pond	Facultative pond	Aerated pond
Administration						
Superintendent engineer	–	–	1/2	1/2	1	1
Secretary	–	–	1/2	1/2	1	1
Assistant	–	–	1	1	1	1
Driver	–	–	1	1	1	1
Operation/maintenance						
Head engineer	1/4	1/4	1/2	1/2	1	1
Chemist	–	–	1/4	1/4	1/2	1/2
Laboratory technician	–	–	1/2	1/2	1	1
Mechanical–electrician	–	–	–	1/2	–	1
Operator- 08:00–16:00 h	1	1	1	1	1	1
Operator 16:00–24:00 h	–	–	–	1	1	1
Operator 24:00–08:00 h	–	–	–	1	–	1
Hand labourers	2	2	2–5	2–7	6–10	7–12

Source: adapted from Yanez (1993)

Table 21.2. Aspects to be included in a daily inspection and occurrence record

Day			
Weather conditions			
weather (sunny, cloudy, rainy) *wind* (none, weak, strong)			
Item	Yes	No	Comment / location / quantity / measures
Observations in the pond			
Is there sludge rising in the pond?			
Are there green patches on the surface?			
Are there black patches on the surface?			
Are there oil stains on the surface?			
Is there vegetation in contact with the water?			
Is there erosion on the slopes?			
Is there visible seepage?			
Are birds present?			
Are insects present?			
Other aspects			
Are the fences in good condition?			
Are the stormwater ditches clean?			
Is the flow meter working?			
Have weeds been removed?			
Has scum been removed?			
Have solids been removed from the screen?			
Has grit been removed from grit chamber?			
Has there been any power failure?			
Has the by-pass to the receiving body been used?			

Source: adapted from Jordão and Pessôa (1995) and Soares (1995)

a) Filling of the pond with water pumped from a neighbouring stream or from a public supply system

- Fill the pond with a minimum water depth, preferably reaching 1 m.
- Close the outlet devices.
- Begin the introduction of sewage until reaching the water depth adopted in the design.

The adoption of this procedure:

- prevents the uncontrolled growth of vegetation, which occurs in conditions of low water depth;
- allows testing of the watertightness of the system;
- enables the correction of occasional problems resulting from a deficient compaction (before the introduction of sewage).

b) Filling of the pond with a mixture of water pumped from the stream and wastewater to be treated

- Mix the wastewater and the water (dilution at a ratio equal to or greater than 1/5)

Table 21.3. Measurement and sampling programme in stabilisation ponds

Frequency	Parameter	Measurem. / analysis	Influent	Facultative pond	Aerated lagoon	Effluent
Daily	Flow (m^3/d)	*In situ*	x			x
	Air temperature (°C)	*In situ*				
	Liquid temperature (°C)	*In situ*	x	x	x	x
	pH	*In situ*	x	x	x	x
	Settleable solids (mL/L)	*In situ*	x			x
	Dissolved oxygen (mg/L)	*In situ*		x	x	
Weekly	Total BOD (mg/l)	Central lab.	x			x
	Total COD(mg/l)	Central lab.	x			x
	Filtered BOD or COD (mg/L)	Central lab.				x
	Faecal coliforms (or *E. coli*) (org/100 mL)	Central lab.	x			x
	Total suspended solids (mg/L)	Central lab.	x			x
	Volatile suspended solids (mg/L)	Central lab.	x			x
Monthly	Organic nitrogen (mg/L)	Central lab.	x			x
	Ammonia (mg/L)	Central lab.	x			x
	Nitrate (mg/L)	Central lab.				x
	Phosphorus (mg/L)	Central lab.	x			x
	Sulphate (mg/L)	Central lab.	x			x
	Sulphide (mg/L)	Central lab.	x			x
	Alkalinity (mg/L)	Central lab.	x			
	Oils and greases (mg/L)	Central lab.	x			x
Occasional	Counting of zooplankton	Central lab.		x		
	Counting of phytoplankton	Central lab.		x		
	Main genera of algae	Central lab.		x		
	DO produced by photosynthesis (mg/L.h)	*In situ*		x		
	DO consumed by respiration (mg/L.h)	*In situ*		x		
	Hourly flow (m^3/h) (24h,every h)	*In situ*	x			
	Hourly DO (mg/L) (24h,every h)	*In situ*		x	x	

Notes: Anaerobic ponds – the programme can be similar to that of facultative ponds (including raw sewage), excluding the determination of algae genera, DO production studies, and light penetration; *Maturation ponds* – the programme can be similar to that of facultative ponds; *Use of effluent in agriculture* – include agronomic (electric conductivity, Ca, Mg, Na, B and others) and sanitary (helminth eggs) parameters
Source: adapted from CETESB (1989), WEF (1990), Yanez (1993), Silva (1993), Jordão and Pessôa (1995)

- Fill up the pond to a depth of approximately 0.40 m
- Await some days, until the appearance of algae is visible
- In the subsequent days, add more wastewater, or wastewater/water mixture, until algal blooming occurs
- Interrupt feeding for a period of 7 to 14 days
- Fill up the pond with wastewater until the operation level
- Interrupt the feeding
- Await the establishment of a population of algae (around 7 to 14 days)
- Feed the pond normally with the wastewater

Should no water be available, the ponds can be filled up with raw sewage and left for about 3 to 4 weeks, in order to allow the development of the microbial population. Some odour release will be unavoidable in this period (Mara et al, 1992).

The whole loading period should be monitored by operators with experience in the process. The total loading period can last 60 days, until a balanced biological community is established in the medium.

The following two procedures should be avoided:

- *Feed with the wastewater load adopted in the design, but without a balanced biological community established in the pond.* If this happens, the pond will suffer from anaerobiosis, with release of bad odours. The reversal of the anaerobiosis process can take two months.
- *Feed the ponds with small, continued loads*, which frequently occur when there are few housing connections. In this case, as the soil is not clogged yet, the liquid could percolate through the slopes, accumulating decomposable solids and releasing bad odours.

21.4.2 Beginning of operation of anaerobic ponds

The beginning of the operation of anaerobic ponds requires the following procedures (CETESB, 1989):

- Begin the introduction of sewage according to the recommendations in Section 21.4.1.
- Maintain the pH of the medium slightly alkaline (7.2 to 7.5). To facilitate the occurrence of these conditions, digested sludge from sewage treatment plants or from Imhoff tanks, or limestone, vegetable ash or sodium bicarbonate can be added after 30 days of operation.

Anaerobic ponds should be started-up after the facultative ponds. This avoids the release of odours from the discharge of anaerobic effluents into an empty facultative pond. Should the concentration of raw sewage be very low, or its flow be small in the beginning of the operation of the system, it would be better to divert the raw sewage to the facultative pond, until a volumetric organic load of at least 0.1 kgBOD/m^3.d is reached in the anaerobic pond (Mara et al, 1992). If there is more than one anaerobic pond in parallel, only one pond could be loaded, so that the load applied to this pond is the same as or higher than the minimum value of 0.1 kgBOD/m^3.d.

21.4.3 Beginning of operation of facultative ponds

The following procedures are recommended (CETESB, 1989):

- Begin the introduction of sewage according to the recommendations of Section 21.4.1.
- The maintenance of a slightly alkaline pH should happen naturally, in case the recommendations of Section 21.4.1 are followed.
- Measure the dissolved oxygen daily.

21.4.4 Beginning of operation of ponds-in-series systems

The ponds located downstream of the primary pond can be started-up according to the following recommendations (CETESB, 1989):

- Begin the filling of the ponds when the water depth in the primary pond reaches a minimum value of 1.0 m.
- Close the outlet devices of the ponds.
- Water should be added to the ponds until a depth of 1.0 m is reached.
- When the primary pond reaches the operational level, its effluent can be directed to the subsequent cell, taking the following precautions:
    - Remove the stop-logs slowly, preventing the water depth of the previous unit from dropping below 1.0 m
    - Do not perform bottom discharge operations from the primary cell
    - Equalise the water depth in all ponds slowly
    - Avoid the situation in which a pond is totally full, while the subsequent unit is empty

21.5 OPERATIONAL PROBLEMS

The main operational problems of anaerobic, facultative and aerated ponds are listed in Tables 21.4, 21.5 and 21.6, together with the main measures to be taken for their possible solution (source: CETESB, 1989; WEF, 1990; Silva, 1995; Jordão e Pessôa, 1995).

Table 21.4. Main operational problems of **anaerobic ponds** and their possible solutions

Problem: bad odours

Causes

- Sewage overload and small detention time
- Very low load and an excessively high detention time (the pond behaves as a facultative one, with the presence of DO in the liquid)
- Presence of toxic substances
- Abrupt fall of the wastewater temperature

Prevention and control measures

- Recirculate the effluent from the facultative or maturation pond to the inlet of the anaerobic pond (recirculation ratio of approximately 1/6)
- Improve the distribution of the influent to the pond (distribution by perforated tubes on the bottom of the pond)
- In case of overload apply occasional partial by-pass to the facultative pond (if it supports the increased load)
- In the case of long detention times, operate with only one anaerobic pond (if there are two or more ponds in parallel)
- Add sodium nitrate to several points of the pond
- Add lime (~12 g/m^3 of the pond) to raise the pH, reducing the acid conditions responsible for the inhibition of methanogenic organisms and for the larger presence of sulphide in the free, toxic form
- Add products that remove sulphides
- Avoid the addition of chlorine, because it will cause subsequent problems to the restart of the biological activities

Problem: proliferation of insects

Causes

- Screened material or sand removed not conveniently disposed of
- Growth of vegetation where the water level is in contact with the internal slope
- Oil and scum layer always present
- Poor maintenance

Prevention and control measures

- Bury the material removed from the screens and grit chambers
- Cut the grown vegetation
- Revolve, with a rake or water jet, the layer of floating material that covers the ponds
- Apply carefully insecticides or larvicides to the scum layer

Problem: growth of vegetation

Causes

- Inadequate maintenance

Prevention and control measures

- Aquatic vegetation (that grows on internal slopes): total removal, preventing it from falling in the pond
- Terrestrial vegetation (that grows on external slopes): remove weeds from the soil; add chemical products for control of weeds

Problem: green patches where the water level is in contact with the slope

Causes

- Proliferation of algae, in view of the small depth in the water level-slope section

Prevention and control measures

- Remove the algae colonies

Problem: blocking of the inlet pipes

Causes

- Inlet pipes obstructed

Prevention and control measures

- Clean the pipes with a stick or steel wire

Problem: surface of the pond covered with a scum layer

Causes

- Scum, oils and plastics

Prevention and control measures

- No measure needs to be taken: the scum layer is totally normal in anaerobic ponds, helping to maintain the absence of oxygen, to control the temperature and to hinder the release of bad odours

Table 21.5. Main operational problems of **facultative ponds** and their possible solutions

Problem: scum and floating material (preventing the passage of light energy)

Causes

- Excessive blooming of algae (forming a greenish surface)
- Discharge of unwanted material (e.g.: rubbish)
- Sludge lumps released from the bottom
- Little circulation and wind influence

Prevention and control measures

- Break the scum with water jets or with a rake (broken scum usually sinks)
- Remove the scum with cloth sieves, burying it later
- Break or remove the sludge lumps
- Remove physical obstacles to penetration of the wind (if possible)

Problem: bad odours caused by overload

Causes

- Overload of sewage, causing lowering of the pH, reduced DO concentration, change in the effluent colour from green to yellowish green (predominance of rotifers and crustaceans, which eat algae), appearance of grey zones close to the influent, and bad odours

Prevention and control measures

- Change the operation of the ponds from serial to parallel
- Remove temporarily the problematic pond from operation (provided there are at least two ponds in parallel)
- Recirculate the effluent at a ratio of 1/6
- Consider the adoption of multiple inlets, to avoid preferential paths
- In case of consistent overloads, consider the inclusion of aerators in the pond
- Add occasionally sodium nitrate, as a supplementary source of combined oxygen

Problem: bad odours caused by poor atmospheric conditions

Causes

- Long periods with cloudy weather and low temperature

Prevention and control measures

- Reduce the water depth
- Put a pond in parallel in operation
- Install surface aerators close to the influent inlet

Problem: bad odours caused by toxic substances

Causes

- Toxic substances from industrial discharges, generating sudden anaerobic conditions in the pond

Prevention and control measures

- Perform a complete physical–chemical analysis of the influent, in order to identify the possible toxic compound
- Identify, in the catchment area, the industry causing the discharge, taking the measures provided for by the legislation
- Isolate the affected pond
- Place a second unit in parallel in operation, provided with aeration, if possible

Table 21.5 (*Continued*)

Problem: bad odours caused by hydraulic short circuits

Causes

- Poor distribution of the influent
- Dead zones resulting from the excessive utilisation of the contours when shaping the pond
- Presence of aquatic vegetation in the pond

Prevention and control measures

- Collect samples at several points in the pond (e.g.: DO) to verify whether there are significant differences from one point to another
- In case of multiple inlets, provide a uniform distribution of influent flow in all inlets
- In case of a simple inlet, build new inlets
- Cut and remove aquatic vegetation
- In case of dead zones, introduce aeration to cause small mixing

Problem: bad odours caused by masses of floating algae

Causes

- Excessive blooming of algae, preventing the penetration of light energy, and causing problems associated with the mortality of the excessive population

Prevention and control measures

- Water jet with hose
- Destruction by rake
- Removal by sieves

Problem: high concentrations of algae (SS) in the effluent

Causes

- Environmental conditions that favour the growth of certain algae populations

Prevention and control measures

- Remove the effluent submerged through baffles, which retain the algae
- Use multiple cells in series, with a small detention time in each cell
- Undertake the post-treatment of the effluent from the pond, to remove excessive SS

Problem: presence of cyanobacteria

Causes

- Incomplete treatment
- Overload
- Unbalanced nutrients

Prevention and control measures

- Break the blooming of algae (cyanobacteria)
- Add judiciously copper sulphate

Problem: presence of filamentous algae and moss, which limit the penetration of light energy

Causes

- Overdesigned ponds
- Influent load seasonally reduced

Prevention and control measures

- Increase the unit load, through the reduction of the number of ponds in operation
- Use operation in series

(Continued)

Table 21.5 (*Continued*)

Problem: progressive tendency to reduce the DO (DO below 3 mg/L in the warm months)
Causes • Low penetration of sun light • Low detention time • High BOD load • Toxic industrial wastewater
Prevention and control measures • Remove floating vegetation • Reduce load in the primary pond through operation in parallel • Introduce complementary aeration • Recirculate the final effluent
Problem: progressive tendency to reduce the pH (ideal pH above 8), with mortality of the green algae
Causes • Overload • Long periods with adverse atmospheric conditions • Organisms eating algae
Prevention and control measures • See measures related to low DO or bad odours due to overload
Problem: proliferation of insects
Causes • Presence of vegetation on the internal slopes of the ponds in contact with the water level
Prevention and control measures • Reduce the water level, causing the larvae trapped in the vegetation of the slopes to disappear when the area dries • Operate the pond with variation in the water level • Protect the internal slope with concrete plates, reinforced mortar, geomembrane, etc • Place fish in the pond, such as carps • Destroy the scum • Apply chemical products judiciously
Problem: vegetation inside the pond
Causes • Low operational level of the pond (below 60 cm) • Excessive seepage • Low wastewater flow
Prevention and control measures • Operate the ponds with a level higher than 90 cm • Cut the vegetation on the internal borders, preventing it from falling in the ponds • Protect the slope internally with concrete plates, reinforced mortar, rip-rap, geomembranes, etc • Remove the vegetation inside the pond with canoes or dredges (lower the water level to facilitate the operation) • Reduce the permeability of the pond with a layer of clay (if possible) • Apply herbicides judiciously

Table 21.6. Main operational problems of **aerated ponds** and their possible solutions

Problem: DO absent in some points
Causes • Poor positioning of the aerators • Overload in the initial sections
Prevention and control measures • Change the position of the aerators • Place more aerators close to the inlet end • Analyse overloading conditions (see corresponding items in Table 21.5)
Problem: occurrence of bad odours and flies
Causes • Accumulated scum in the corners and in the internal slopes
Prevention and control measures • Remove the floating material
Problem: variable DO, dispersed floc and foam
Causes • Shock loads • Over aeration • Industrial wastewater
Prevention and control measures • Control the operation of the aerators by switching on-off • Monitor DO to establish the ideal form of operation of the aerators • Maintain DO around 1 mg/L or more • Identify the industrial wastewater causing the foams and require its pre-treatment

22

Management of the sludge from stabilisation ponds

22.1 PRELIMINARIES

As in all biological wastewater treatment processes, there is also production of sludge in stabilisation ponds. This sludge is associated with the solids present in the raw sewage and, mainly, with the biomass developed in the biological treatment itself. The various chapters that cover stabilisation pond variants in this book present values for the estimated volumetric sludge production (expressed in m^3/inhab.year or in cm/year). This chapter, based on Gonçalves (1999), presents additional details about the characteristics of the sludge and, mainly, about the management of the sludge from stabilisation ponds. However, the reference Gonçalves (1999) should be consulted for further details on the theme.

One of the main advantages of the *facultative* ponds is the possibility to accumulate sludge on the bottom of the pond, during the whole operational period, with no need for its removal. However, in the most *compact* ponds (anaerobic ponds, facultative aerated lagoons and sedimentation ponds), the occupation of the useful volume of the pond with the accumulated sludge is more significant, requiring an appropriate management, including removal, occasional processing and final disposal.

A further description about the treatment and final disposal of the sludge is not intended here as these items are dealt with in the quoted reference and in Part 7 of this book.

Biological Wastewater Treatment in Warm Climate Regions by Marcos von Sperling and Carlos Augusto de Lemos Chernicharo. ISBN: 1 84339 002 7. Published by IWA Publishing, London, UK.

22.2 CHARACTERISTICS AND DISTRIBUTION OF THE SLUDGE IN STABILISATION PONDS

The characteristics of the sludge accumulated in the stabilisation ponds vary according to its retention time in the pond, which usually amounts from some to many years. In this period, the sludge undergoes *thickening* and *anaerobic digestion*, which are reflected on the high contents of total solids (TS) and on the low volatile solids / total solids ratio (VS/TS).

The sludge removed from primary ponds usually presents high contents of total solids, frequently higher than 15%. Because of thickening, the solids concentration varies along the sludge layer, with higher values in the lower parts. Sludges from shallow polishing ponds accumulated over short time periods (one year or less) have average solids concentrations of approximately 4 to 6% (Brito et al, 1999; von Sperling et al, 2002b).

The sludge from ponds operating for several years is usually well digested, with VS/TS ratios lower than 50%.

In terms of nutrients (nitrogen, phosphorus and potassium), the data obtained from an anaerobic pond and from a primary facultative pond (Gonçalves, 1999) suggest nutrient contents lower than those obtained from other wastewater treatment processes. The average values obtained were: TKN: 2.0% of the TS; P: 0.2% of the TS; K: 0.04% of the TS.

With relation to heavy metals, the considerations made in Chapter 2 concerning the dependence between these characteristics and the presence and type of industrial wastes are also valid here.

Regarding coliforms, the contents in the sludge range between 10^2 and 10^4 FC/gTS, and their decay takes place during the accumulation period in the pond.

Helminth eggs are found in large quantities in pond sludge, since the main egg removal mechanism from the liquid phase is sedimentation. The figures vary substantially from one wastewater treatment plant to another, in view of the variable counting in the raw sewage in each location. Values obtained from the sludge of an anaerobic pond (Gonçalves, 1999) and two polishing ponds (von Sperling et al, 2002), both in Brazil, ranged largely from 30 to 800 eggs/gTS. A long sludge digestion period in the pond seems to contribute to a reduced viability of the eggs. However, it is important to highlight that the sludge from ponds, even after several years, still contains viable eggs, what must be taken into account in their management. Data on the sludge from the anaerobic pond mentioned above, operating for several years, are associated with a percentage of viability between 1 and 10%, while the sludge from the polishing ponds, after operation periods of only six months and one year, presented much higher percentages of viability, between 60 and 90%. The helminth species prevailing in the referred to ponds was *Ascaris lumbricoides*, ranging from 50 to 99% of the total counting of eggs found. On worldwide terms, the most prevailing helminth species is *Ascaris lumbricoides*, but of course the countings and the percentage distribution will vary from place to place.

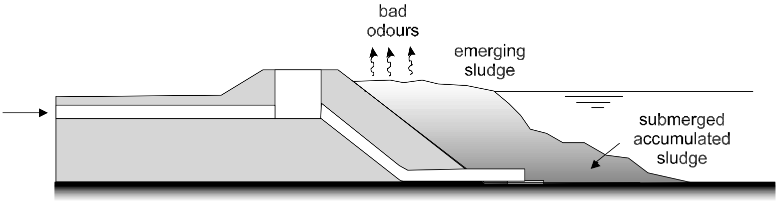


Figure 22.1. Non-homogeneous spatial distribution of the sludge, with sludge rising to the surface and possible release of malodorous compounds (adapted from Gonçalves, 1999)

The thickness and the characteristics of the sludge layer vary inside the ponds, depending on their geometry and on the positioning of the inlet and outlet structures. Different profile patterns were observed by Gonçalves (1999), but the most frequent one, mainly in primary ponds and in elongated (baffled) ponds, is that of a higher sludge layer close to the inlet. The greatest concern occurs when the sludge layer rises up to and over the water surface, allowing the release of malodorous compounds (Figure 22.1). This situation happens more frequently in ponds without previous grit removal and in anaerobic ponds. In case the sludge is not removed, at least the inclusion of a grit chamber and the redistribution of the emerging sludge layer and of the pond inlets should be performed.

22.3 REMOVAL OF SLUDGE FROM STABILISATION PONDS

22.3.1 Introduction

The removal of sludge is likely to be a compulsory task of a significant scale in the operation of many ponds. However, there is still no widely accepted engineering solution for that. The removal needs to be well planned, since the technique used can change the characteristics of the sludge (increase the water content), and hinder its final disposal.

Gonçalves et al (1999) present in detail the planning and the techniques employed to remove sludge from the ponds. The present item is integrally based on this reference.

22.3.2 Information on the sludge volume to be removed

The planning of the sludge removal from a pond has the purpose of minimising costs, anticipating solutions to occasional problems, and reducing impacts related to the sludge removal and disposal. The following stages are essential in the cleaning operation:

1. Determination of the pond geometry based on the design or on a topographic survey.

2. Accomplishment of the pond bathymetry, defining bathymetric sections, liquid height of the pond, and depth of the sludge layer.
3. Physical–chemical and microbiological characterisation of the sludge.
4. Definition of the technique to be adopted in the removal of the sludge and, if necessary, in the sludge dewatering and transportation.
5. Definition of the adequate final destination of the sludge, considering the lowest possible environmental impacts.

Certainly, stages 1, 2, and 3 are pre-requisites for the implementation of stage 4, which defines the technique to remove the sludge from the pond. Although there is no consensus on the technique, its selection has a direct impact on the water content of the sludge and, therefore, on the sludge volume to be disposed of later on.

The subsequent items describe the sludge removal stage. The possible sludge processing (dewatering, disinfection) and its disposal are dealt with in Part 7 of this book.

22.3.3 Techniques for sludge removal from ponds

22.3.3.1 Main techniques for sludge removal

The main pond sludge removal techniques can be classified as follows:

- mechanised or non mechanised
- with interruption or no interruption of the pond operation

This second classification was adopted in the following description, due to the importance of the decision of whether to maintain the pond in operation or not.

For the cases in which the sludge should be submitted to dewatering after removal, the following alternatives can be considered: natural drying in the pond itself, use of drying beds, sludge lagoons, or even the use of mechanical equipment.

In locations with a large number of ponds in the surroundings, the use of a mobile dewatering unit (e.g. with centrifuges) could be taken into consideration.

22.3.3.2 Sludge removal with temporary interruption of the pond operation

The temporary deactivation of a pond can be a simple operational measure, if the primary pond stage has been designed in modules, and if there is an idle treatment capacity. However, if this stage consists of a single pond, or if the nominal design load has been already reached, the temporary deactivation may put in risk the stability of the subsequent treatment stage.

Another important aspect is related to emptying the pond. This operation, necessary for drying the sludge in the pond itself, requires previous planning and consent from the environmental agency. In case of very fast emptying, mainly in anaerobic ponds, the impact of the anaerobic effluent on the receiving body can exceed its self-purification capacity. Fish death, unpleasant odours and protests by the population may arise as a consequence.

a) Manual removal

In this case, the sludge is submitted to drying inside the pond itself, until it is consistent enough to be removed by spades and wheelbarrows (TS>30%).

The disadvantage of this technique is that it requires a long drying period. Considering the period of time necessary to empty the pond, the drying period, and the period for the manual removal of the sludge, the pond will certainly remain deactivated for more than 3 months.

However, the sludge volume to be removed under these conditions is much lower than the volume existent prior to the drying. Another positive aspect is the possible complementary disinfection of the sludge by sunlight-induced pasteurisation. This can be a feasible solution for small sewage treatment plants (<5000 inhabitants).

b) Mechanical removal (by tractors)

As in the previous technique, the sludge is submitted to drying in the pond and removed soon after. In view of the higher yield of the machines in the sludge removal, the pond can start to work again more quickly than in case of manual removal. However, for tractors or shovels to gain access to the bottom of the pond, the soil support capacity should be previously verified, so that neither the pond bottom sealing nor the stability of the slopes are affected.

The ease of access of the machines into the pond should be evaluated, considering the option of partial rupture of the slopes for further reconstruction. There have been cases of tractors stuck in the sludge in ponds, for which reason it is recommended that the bottom of the pond should not be accessed while the sludge presents a pasty consistency (20% < TS < 30%).

c) Mechanised scraping and pumping of the sludge

When the pond cannot be deactivated for a very long period of time, the sludge is partially dried in the air, mechanically scraped, and then pumped. This technique requires the aid of a tractor or another device to convey the sludge still in the liquid state to a lower point from where it will be pumped.

The use of positive displacement pumps (piston, diaphragm, rotating lobes, high-pressure piston, etc.) is recommended due to their capacity to move the sludge mass. Torque pumps (centrifuges) can be used, although they require dilution of the highly concentrated sludge, which results in an increased volume of sludge removed.

22.3.3.3 Sludge removal with the pond in operation

a) Removal by hydraulic sludge discharge pipe

The hydraulic sludge discharge pipe (bottom drain) is the device more frequently included in the design of anaerobic or aerated stabilisation ponds. Nevertheless, it is a solution highly criticised by operators.

There are several reports on clogging and loss of function of this device during the operation of the pond. The problem occurs in view of the evolution of solids contents in the sludge over the years, making its consistency change from liquid to pasty. Should the sludge be discarded with a higher frequency (<5 years), which would prevent its thickening at levels higher than 7% on the bottom of the pond, this device could be useful in small sewage treatment plants. For Victoretti (1975), sludge discharge devices are unnecessary, because the ponds operate for long periods with no need of sludge removal. According to the author, the units should be designed to be deactivated for drainage and removal of the sludge.

In case this technique of pond sludge removal is adopted, pipe diameters equal to or larger than 200 mm are recommended (Metcalf and Eddy, 1991).

b) Removal by septic tank cleaning truck

Septic tank cleaning or similar trucks are provided with a vacuum suction system with a flexible pipe that removes the sludge and conveys it to the sludge storage compartment in the trucks themselves.

The disadvantage of this solution is that it removes the sludge with a high water level, once pumping requires the dilution of the sludge layers already in an advanced thickening stage. The result can be the need of many trips to transport the sludge from the sewage treatment plant to the disposal site. However, its great advantage is that it removes and transports the sludge in the same operation. The equipment can also be easily found and rented in medium- and large-sized cities.

c) Dredging

The use of dredges allows the removal of sludge with TS contents higher than 15%, if the sludge is scraped by mechanical means. For sludge with higher solids levels, this type of removal process is affected due to the consistency of the material.

The dredges can also be provided with a sludge-layer-breaking device, so that the removal is accomplished by pumping. In this case, the sludge is removed with water contents higher than those in case of mechanical scraping. Remote control equipment is available.

The dredging may suspend solids at the pond outlet, following revolvement of the bottom sludge layer. This fact can cause a significant load of solids to the secondary pond, if existent. Another important aspect refers to the stability of the waterproofing seal on the bottom of the pond, which may be damaged by the dredging.

Table 22.1. Advantages and disadvantages of the sludge removal techniques from stabilisation ponds

Technology	Advantages	Disadvantages
Sludge removal techniques used with deactivation of the pond		
Manual removal	• Sludge humidity is removed in the pond itself • Cleaning of the pond is done in a controlled way • Sludge with high TS contents reduces transport costs • Almost complete removal of the sludge	• The pond is deactivated for a long period of time • Employees have direct contact with the sludge
Mechanical removal (by tractors)	• Sludge humidity is removed in place • Cleaning of the pond is done in a controlled way • Sludge with high TS contents reduces transport costs • Higher yield than that of manual sludge removal • Almost complete removal of the sludge	• The pond is deactivated for a long period of time • Possible demolition of part of the slope for machine access • The bottom of the pond may be damaged, requiring repairs • Tractor may get stuck in the sludge
Mechanised scraping and pumping	• Shorter sludge drying time in the pond • Almost complete removal of the sludge	• Removal of sludge with a high water content • Requires tractor access in the pond
Sludge removal techniques with the pond in operation		
Technology	Advantages	Disadvantages
Vacuum system from a septic tank cleaning truck	• Operational simplicity • The equipment is easily available • The sludge is removed and transported in the same operation	• Sludge removal with higher frequency – requires low TS contents • Removal of sludge with a high water content due to the mixing with the liquid during the operation • Requires natural or mechanical dewatering of the sludge removed
Hydraulic discharge pipe	• Operational simplicity • Low cost	• Discharge device gets blocked • Sludge discharge with higher frequency – requires low TS contents • Requires natural or mechanical dewatering of the sludge removed • Difficult control of the discharge operation

(*Continued*)

Table 22.1 (*Continued*)

Sludge removal techniques with the pond in operation		
Technology	Advantages	Disadvantages
Dredging	• Removes the sludge almost completely • Sludge removed with high concentration of solids • Cleaning can be done at a lower frequency	• Need of natural or mechanical dewatering of the sludge removed • Difficult control of the sludge removal operation • Cost of the equipment
Pumping from raft	• Operational simplicity • The equipment is easily available	• Sludge removal with higher frequency – requires low TS contents • Requires natural or mechanical dewatering of the sludge removed • Difficult control of the sludge removal operation
Robotic system	• Removes the sludge almost completely • Sludge with high TS contents reduces transport costs • Allows pond cleaning at lower frequency	• Cost of the equipment • Little availability of the equipment in developing countries

d) Pumping from a raft

The sludge can be pumped from the bottom of the pond by a motor pump installed on a raft. The use of positive displacement pumps (piston, diaphragm, rotating lobes, high-pressure piston, etc.) is also recommended. The motor pump can be propelled by either electricity or fuel. Remote control equipment is available.

The use of centrifugal pumps is only feasible in cases in which the sludge still has a liquid consistency (TS contents <6%), or in cases in which the motor pump is provided with a device for scarifying the sludge on the bottom. The sludge removed by pumping is conveyed outside the pond, where it can be either transported or dewatered in place.

e) Robotic system

This alternative is not largely used in developing countries yet. It can be considered a promising technology in sludge extraction, and consists of a small remote-controlled robotic tractor that moves on a crawler. In the front part of the tractor, the sludge layer is broken and aspired, being then removed from the pond by pumping. The process seems to be capable of removing sludge with high concentrations of solids (TS>20%), allowing the pond to be cleaned at longer intervals. Its main disadvantages are the absence of experience with the equipment in developing countries and the fact that it is imported.

Table 22.2. Comparison of the main factors for selection of the sludge removal technique in stabilisation ponds

Technique	Performance	Ease of operation	Execution time	Sludge volume	Cost
Manual removal	* * * *	* *	* * * *	*	* *
Mechanical removal (by tractors)	* * * *	* * *	* * *	*	* *
Mechanised scraping and pumping	* * *	* *	* * *	* *	* *
Vacuum system from a septic tank cleaning truck	*	* * *	* *	* * * *	* * *
Hydraulic discharge pipe	*	* * * *	* *	* * *	*
Dredging	* * *	* *	* *	* * *	* * *
Pumping from raft	* *	* *	* *	* * *	* *
Robotic system	* * * *	*	* *	* *	* * * *

Scale: * * * * Larger →→→ * Smaller

22.3.4 Advantages and disadvantages of the sludge removal techniques

The main advantages and disadvantages of the different sludge removal techniques mentioned previously are summarised in Table 22.1. A comparison among the different techniques considered, involving factors such as process performance, operational ease, flexibility with relation to the final disposal of the sludge, amount of sludge removed, and operational cost, is presented in Table 22.2. The comparison is just for an initial analysis, since the specific conditions of each stabilisation pond can change completely the applicability of the techniques at issue.

Part Three

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PART FOUR

Anaerobic reactors

23

Introduction to anaerobic treatment

23.1 APPLICABILITY OF ANAEROBIC SYSTEMS

As a result of expanded knowledge, anaerobic sewage treatment systems, especially upflow anaerobic sludge blanket (UASB) reactors, have grown in maturity, occupying an outstanding position in several tropical countries in view of their favourable temperature conditions. Their acceptance changed from a phase of disbelief, which lasted until the early 1980s, to the current phase of widespread acceptance.

However, this great acceptance has frequently led to the development of projects and the implementation of treatment plants with serious conceptual problems. In this sense, the following chapters aim at providing information related to the principles, design and operation of anaerobic sewage treatment systems, with emphasis on upflow anaerobic sludge blanket reactors and anaerobic filters.

In principle, all organic compounds can be degraded by an anaerobic process, which is more efficient and economic when the waste is easily biodegradable.

Anaerobic digesters have been largely used in the treatment of solid wastes, including agricultural wastes, animal excrements, sludge from sewage treatment plants and urban wastes, and it is estimated that millions of anaerobic digesters have been built all over the world with this purpose. Anaerobic digestion has also been largely used in the treatment of effluents from agricultural, food and beverage industries, both in developed and developing countries, as shown in Table 23.1.

Also concerning the treatment of domestic sewage in warm-climate regions, a substantial increment has been verified in the use of anaerobic technology, notably by means of the UASB-type reactors. Naturally, in this case, the application of

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Table 23.1. Main types of industries whose effluents can be treated by anaerobic process

Slaughterhouses and cold storage facilities	Alcohol production	Potato processing
Breweries	Starch production	Coffee processing
Leather factories	Yeast production	Fruit processing
Dairies	Soft drink production	Fish processing
Sugar refineries	Wine production	Vegetable processing

Source: GTZ/TBW (1997)

anaerobic technology depends much more on the temperature of the sewage, due to the low activity of anaerobic microorganisms at temperatures below 20 °C, and to the unfeasibility of heating the reactors. This is because domestic sewage is more diluted than industrial effluents, resulting in low volumetric production rates of methane gas, which makes its use as a source of heat energy uneconomical. Therefore, anaerobic treatment of domestic sewage becomes much more attractive for tropical- and subtropical-climate countries, which are mainly developing countries.

23.2 POSITIVE ASPECTS

Several favourable characteristics of anaerobic systems, likely to be operated under high solids retention times and very low hydraulic detention times, provide them with great potential for application to the treatment of low-concentration wastewaters. They are also simple, low-cost technologies, with some advantages regarding operation and maintenance, as illustrated in Table 23.2.

Table 23.2. Advantages and disadvantages of the anaerobic processes

Advantages	Disadvantages
• Low production of solids, about 3 to 5 times lower than that in aerobic processes • Low energy consumption, usually associated with an influent pumping station, leading to very low operational costs • Low land requirements • Low construction costs • Production of methane, a highly calorific fuel gas • Possibility of preservation of the biomass, with no reactor feeding, for several months • Tolerance to high organic loads • Application in small and large scale • Low nutrient consumption	• Anaerobic microorganisms are susceptible to inhibition by a large number of compounds • Process start-up can be slow in the absence of adapted seed sludge • Some form of post-treatment is usually necessary • The biochemistry and microbiology of anaerobic digestion are complex, and still require further studies • Possible generation of bad odours, although they are controllable • Possible generation of effluents with unpleasant aspect • Unsatisfactory removal of nitrogen, phosphorus and pathogens

Source: Adapted from Chernicharo and Campos (1995); von Sperling (1995); Lettinga *et al.* (1996)

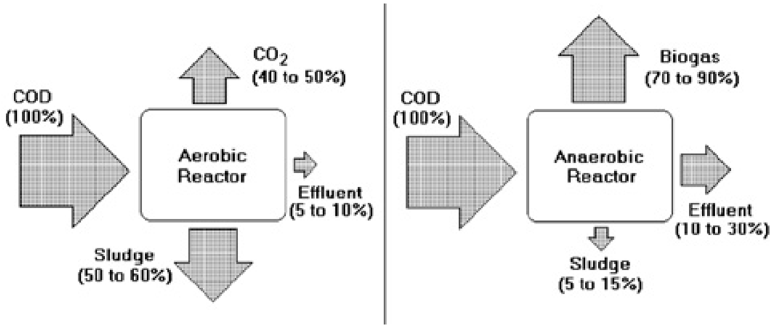

Figure 23.1. Biological conversion in aerobic and anaerobic systems

Figure 23.1 enables a clearer visualisation of some of the advantages of anaerobic digestion in relation to aerobic treatment, notably regarding the production of methane gas and the very low production of solids.

In *aerobic* systems, only about 40 to 50% of biological stabilisation occurs, with its consequent conversion into CO_2. A very large incorporation of organic matter as microbial biomass (about 50 to 60%) is verified, constituting the excess

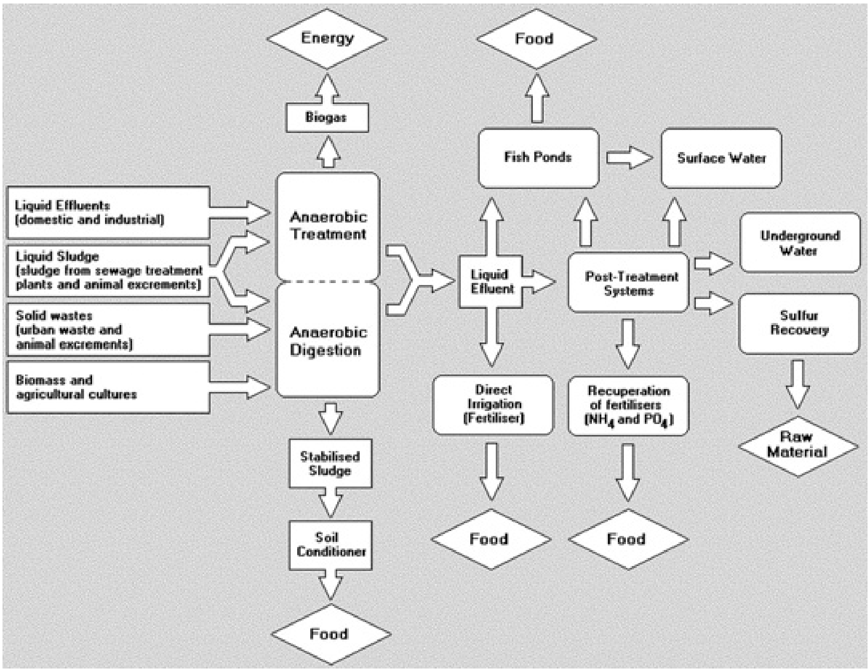

Figure 23.2. Anaerobic digestion as integrated technology for sewage treatment and by-product recovery (adapted from Lettinga, 1995)

sludge of the system. The organic material, not converted into carbon dioxide or into biomass, leaves the reactor as non-degraded material (5 to 10%).

In *anaerobic* systems, most of the biodegradable organic matter present in the waste is converted into biogas (about 70 to 90%), which is removed from the liquid phase and leaves the reactor in a gaseous form. Only a small portion of the organic material is converted into microbial biomass (about 5 to 15%), which then constitutes the excess sludge of the system. Besides the small amount produced, the excess sludge is usually more concentrated, with better dewatering characteristics. The material not converted into biogas or into biomass leaves the reactor as non-degraded material (10 to 30%).

Another interesting approach is made by Lettinga (1995), who emphasises the need for the implementation of integrated environmental protection systems that conciliate sewage treatment and the recovery and reuse of its by-products. The approach has a special appeal to developing countries, which present serious environmental problems, lack of resources and power and, frequently, insufficient food production. In this sense, anaerobic digestion becomes an excellent integrated alternative for sewage treatment and recovery of by-products, as illustrated in Figure 23.2.

24

Principles of anaerobic digestion

24.1 INTRODUCTION

Inorganic electron acceptors, such as SO_4^{2-} or CO_2, are used in the oxidation process of organic matter under anaerobic conditions. Methane formation does not occur in mediums where oxygen, nitrate or sulfate is readily available as electron acceptors. Methane production occurs in different natural environments, such as swamps, soil, river sediments, lakes and seas, as well as in the digestive organs of ruminant animals, where the redox potential is around −300 mV. It is estimated that anaerobic digestion with methane formation is responsible for the complete mineralisation of 5 to 10% of all the organic matter available on the Earth.

Anaerobic digestion represents an accurately balanced ecological system, where different populations of microorganisms present specialised functions, and the breakdown of organic compounds is usually considered a two-stage process. In the first stage, a group of facultative and anaerobic bacteria converts (by hydrolysis and fermentation) the complex organic compounds (carbohydrates, proteins and lipids) into simpler organic materials, mainly volatile fatty acids (VFA), as well as carbon dioxide and hydrogen gases.

In the second stage, the organic acids and hydrogen are converted into methane and carbon dioxide. This conversion is performed by a special group of microorganisms, named methanogens, which are strictly anaerobic prokaryotes. The methanogenic archaea depend on the substrate provided by the acid-forming microorganisms, consisting, therefore, in a syntrophic interaction.

The methanogens carry out two primordial functions in the anaerobic ecosystems: they produce an insoluble gas (methane) which enables the removal of organic

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carbon from the environment, and they also keep the H_2 partial pressure low enough to allow conditions in the medium for fermenting and acid-producing bacteria to produce more oxidised soluble products, such as acetic acid. Once the methanogens occupy the terminal position in the anaerobic environment during organic compound degradation, their inherent low growth rates usually represent a limiting factor in the digestion process as a whole.

Some of the concepts presented here are also discussed in Chapters 7, 8 and 9.

24.2 MICROBIOLOGY OF ANAEROBIC DIGESTION

Anaerobic digestion can be considered an ecosystem where several groups of microorganisms work interactively in the conversion of complex organic matter into final products, such as methane, carbon dioxide, hydrogen sulfide, water and ammonia, besides new bacterial cells.

Although anaerobic digestion is generally considered a two-phase process, it can be subdivided into various metabolic pathways, with the participation of several microbial groups, each with a different physiological behaviour, as illustrated in Figure 24.1 and described in the following items.

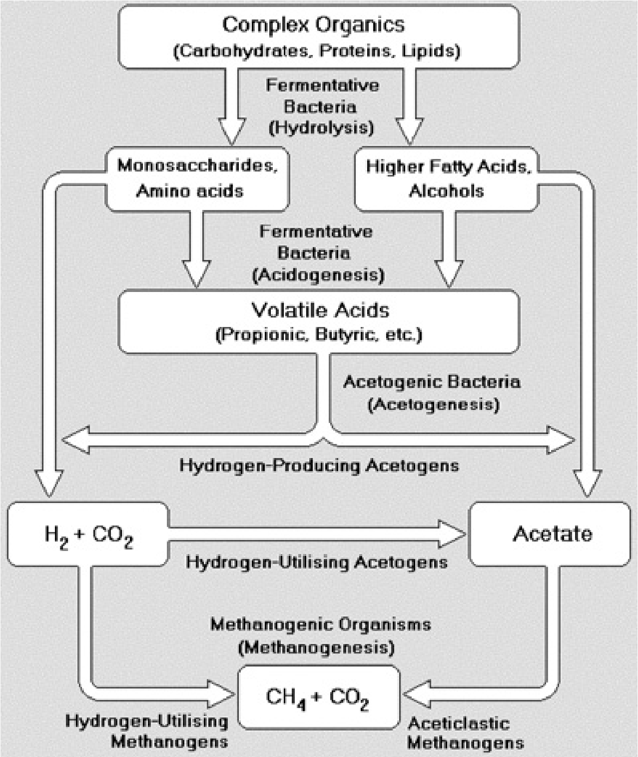


Figure 24.1. Metabolic pathways and microbial groups involved in anaerobic digestion
Adapted from: Lettinga *et al.* (1996)

(a) Hydrolysis and acidogenesis

Since the microorganisms are not capable of assimilating particulate organic matter, the first phase in the anaerobic digestion process consists in the hydrolysis of complex particulate material (polymers) into simpler dissolved materials (smaller molecules), which can penetrate through the cell membranes of the fermentative bacteria. Particulate materials are converted into dissolved materials by the action of exoenzymes excreted by the hydrolytic fermentative bacteria. The hydrolysis of polymers usually occurs slowly in anaerobic conditions, and several factors may affect the degree and rate at which the substrate is hydrolysed (Lettinga *et al.*, 1996):

- operational temperature of the reactor
- residence time of the substrate in the reactor
- substrate composition (e.g. lignin, carbohydrate, protein and fat contents)
- size of particles
- pH of the medium
- concentration of $NH_4{}^+{-}N$
- concentration of products from hydrolysis (e.g. volatile fatty acids)

The soluble products from the hydrolysis phase are metabolised inside the cells of the fermentative bacteria and are converted into several simpler compounds, which are then excreted by the cells. The compounds produced include volatile fatty acids, alcohols, lactic acid, carbon dioxide, hydrogen, ammonia and hydrogen sulfide, besides new bacterial cells.

Acidogenesis is carried out by a large and diverse group of fermentative bacteria. Usual species belong to the clostridia group, which comprises anaerobic species that form spores, able to survive in very adverse environments, and the family *Bacteroidaceaea*, organisms commonly found in digestive tracts, participating in the degradation of sugars and amino acids.

(b) Acetogenesis

Acetogenic bacteria are responsible for the oxidation of the products generated in the acidogenic phase into a substrate appropriate for the methanogenic microorganisms. In this way, acetogenic bacteria are part of an intermediate metabolic group that produces substrate for methanogenic microorganisms. The products generated by acetogenic bacteria are acetic acid, hydrogen and carbon dioxide.

During the formation of acetic and propionic acids, a large amount of hydrogen is formed, causing the pH in the aqueous medium to decrease. However, there are two ways by which hydrogen is consumed in the medium: (i) through the methanogenic microorganisms, that use hydrogen and carbon dioxide to produce methane; and (ii) through the formation of organic acids, such as propionic and butyric acids, which are formed through the reaction among hydrogen, carbon dioxide and acetic acid.

Among all the products metabolised by the acidogenic bacteria, only hydrogen and acetate can be directly used by the methanogenic microorganisms. However, at least 50% of the biodegradable COD are converted into propionic and butyric acids, which are later decomposed into acetic acid and hydrogen by the action of the acetogenic bacteria.

(c) Methanogenesis

The final phase in the overall anaerobic degradation process of organic compounds into methane and carbon dioxide is performed by the methanogenic archaea. They use only a limited number of substrates, comprising acetic acid, hydrogen/carbon dioxide, formic acid, methanol, methylamines and carbon monoxide. In view of their affinity for substrate and extent of methane production, methanogenic microorganisms are divided into two main groups, one that forms methane from acetic acid or methanol, and the other that produces methane from hydrogen and carbon dioxide, as follows:

- acetate-using microorganisms (aceticlastic methanogens)
- hydrogen-using microorganisms (hydrogenotrophic methanogens)

Aceticlastic methanogens. Although only a few of the methanogenic species are capable of forming methane from acetate, these are usually the microorganisms prevailing in anaerobic digestion. They are responsible for about 60 to 70% of all the methane production, starting from the methyl group of the acetic acid. Two genera utilise acetate to produce methane: *Methanosarcina* prevails above 10^{-3} M acetate, while *Methanosaeta* prevails below this acetate level (Zinder, 1993). *Methanosaeta* may have lower yields and be more pH-sensitive, as compared to *Methanosarcina* (Schimidt and Ahring, 1996). *Methanosarcina* has a greater growth rate, while *Methanosaeta* needs a longer solids retention time, but can operate at lower acetate concentrations. The *Methanosaeta* genus is characterised by exclusive use of acetate, and having a higher affinity with it than the methanosarcinas. They are developed in the form of filaments, being largely important in the formation of the bacterial texture present in the granules. The organisms belonging to the *Methanosarcina* genus are developed in the form of coccus, which group together forming “packages”. They are considered the most versatile among the methanogenic microorganisms, since they own species capable of using also hydrogen and methylamines (Soubes, 1994).

Hydrogenotrophic methanogens. Unlike the aceticlastic organisms, practically all the well-known methanogenic species are capable of producing methane from hydrogen and carbon dioxide. The genera more frequently isolated in anaerobic reactors are *Methanobacterium, Methanospirillum* and *Methanobrevibacter*. Both the aceticlastic and the hydrogenotrophic methanogenic microorganisms are very important in the maintenance of the course of anaerobic digestion, since they are responsible for the essential function of consuming the hydrogen produced in the previous phases. Consequently, the partial pressure of hydrogen in the medium is lowered, thus enabling the production reactions of the acidogenic and acetogenic bacteria (see Section 24.3.3).

(d) Sulfate reduction

In reactors treating wastewater containing sulfate or sulfite, these compounds can be used by sulfate-reducing bacteria (SRB) as acceptors of electrons released during the oxidation of organic materials (Lettinga *et al.*, 1996).

The metabolism of SRB is important in the anaerobic process, mostly because of their end product, hydrogen sulfide. SRB group species have in common the dissimilatory sulfate metabolism under strict anaerobiosis, and are considered a very versatile group of microorganisms, capable of using a wide range of substrate, including the whole chain of volatile fatty acids, several aromatic acids, hydrogen, methanol, ethanol, glycerol, sugars, amino acids and several phenol compounds.

Two major metabolic groups of SRB can be distinguished: (i) a group of species that is able to oxidise incompletely its substrates to acetate, like the genera *Desulfobulbus* sp. and *Desulfomonas* sp., and most of the species of the genera *Desulfotomaculum* and *Desulfovibrio* belong to this group; and (ii) a group which is able to oxidise its organic substrates, including acetate, to carbon dioxide. The genera *Desulfobacter*, *Desulfococcus*, *Desulfosarcina*, *Desulfobacterium* and *Desulfonema* belong to this group.

In the absence of sulfate, the anaerobic digestion process occurs according to the metabolic sequences presented in Figure 24.1. With the presence of sulfate in the wastewater, many of the intermediate compounds formed by means of the metabolic routes identified in Figure 24.1 start to be used by the SRB, causing a change in the metabolic routes in the anaerobic digester (see Figure 24.2). Hence, the SRB start to compete with the fermentative, acetogenic and methanogenic microorganisms for the substrate available, resulting in a decrease in the production of methane from a given amount or organic material present in the influent. The importance of this bacterial competition is greater when the relative concentration of SO_4^{2-} is increased in relation to the COD concentration (see Section 24.3.7).

24.3 BIOCHEMISTRY OF ANAEROBIC DIGESTION

24.3.1 Preliminaries

Anaerobic digestion of organic compounds comprises several types of methanogenic and acidogenic microorganisms, and the establishment of an ecological balance among the types and species of anaerobic microorganisms is of fundamental importance to the efficiency of the treatment system. The VFA parameter is frequently used for the evaluation of this ecological balance.

The volatile fatty acids are formed, as intermediate products, during the degradation of carbohydrates, proteins and lipids. The most important components resulting from the biochemical decomposition of the organic matter are the short-chain volatile acids, such as formic, acetic, propionic, butyric and, in smaller amounts, valeric and isovaleric acids. These low-molecular-weight fatty acids are named volatile acids because they can be distilled at atmospheric pressure. The volatile acids represent intermediate compounds, from which most of the methane is produced, through conversion by the methanogenic microorganisms.

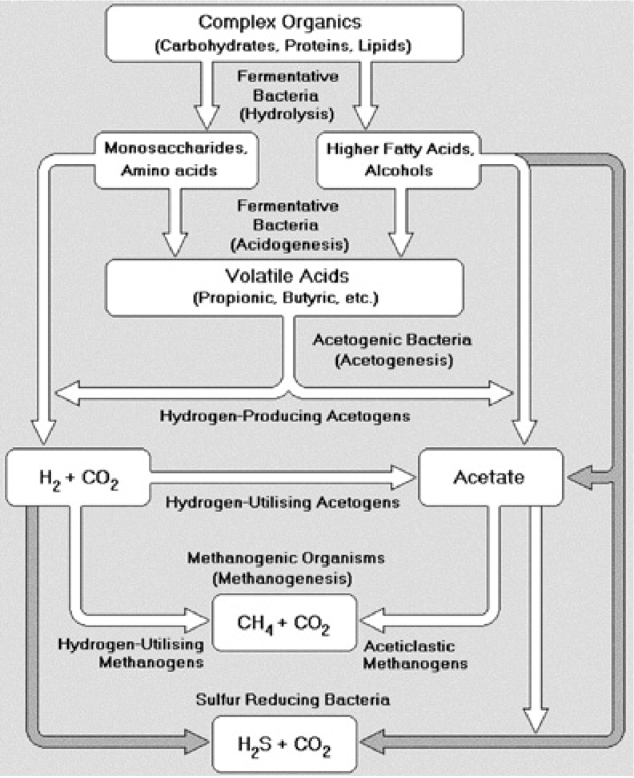


Figure 24.2. Metabolic pathways and microbial groups involved in anaerobic digestion (with sulfate reduction). Source: Adapted from Lettinga *et al.* (1996)

When a population of methanogenic microorganisms is present in a sufficient amount, and the environmental conditions inside the treatment system are favourable, they use the intermediate acids as quickly as they are formed. Consequently, the acids do not accumulate beyond the neutralising capacity of the alkalinity naturally present in the medium, the pH remains in a range favourable for the methanogenic organisms and the anaerobic system is balanced. However, if the methanogenic organisms are not present in sufficient amount, or if they are exposed to unfavourable environmental conditions, they will not be capable of using the volatile acids at the same rate at which they are produced by the acidogenic bacteria, resulting in an accumulation of acids in the system. In these conditions, the alkalinity is quickly consumed, and the non-neutralised free acids cause the pH to drop. When that occurs the reactor is referred to by operators as ‘sour’ (because of its odour).

An identification of the individual acids present in a reactor with unbalanced bacterial populations can indicate which types of methanogenic microorganisms are not fulfilling their role in the treatment.

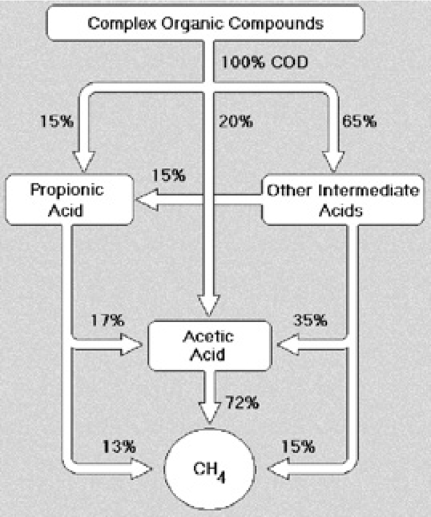


Figure 24.3. Methane formation routes from the fermentation of complex substrates (adapted from McCarty, 1964)

24.3.2 Intermediate volatile acids

The most important intermediate volatile acids, precursors of methane formation, are the acetic and propionic acids. Some of the various metabolic steps involved in the degradation of a complex substrate, such as the excess sludge from domestic sewage treatment plants, are shown in Figure 24.3. The percentages shown are based on COD conversion, valid only for the formation of methane from complex substrates, such as sludges from sewage treatment plants or others of similar composition.

For the complete fermentation of complex compounds into methane, each group of microorganisms has a specific function. Even if the contribution to the process as a whole is small, it is nevertheless necessary for the formation of the final product. Propionic acid results mainly from the fermentation of the carbohydrates and proteins present, and about 30% of the organic compounds are converted into this acid before they can be finally converted into methane. Acetic acid is the most abundant intermediate acid, formed from all the organic compounds. Concerning the degradation of complex substrates, such as sludge from sewage treatment plants, acetic acid is precursor of about 72% of the methane formed and, together with propionic acid, of about 85% of the total methane production. A large part of the remaining 15% results from the degradation of other acids, such as formic and butyric acids.

24.3.3 Thermodynamic aspects

Some of the conversion reactions of the products from fermentative bacteria into acetate, hydrogen and carbon dioxide are illustrated in Table 24.1. The last column

Table 24.1. Some important oxi-reduction reactions in anaerobic digestion

Nr	Oxidation reactions (electron donors)		ΔG_o (kJ/mole)
1	Propionate ⇒ acetate	$CH_3CH_2COO^- + 3H_2O \Rightarrow CH_3COO^- + HCO_3^- + H^+ + 3H_2$	+76.1
2	Butyrate ⇒ acetate	$CH_3CH_2CH_2COO^- + 2H_2O \Rightarrow 2CH_3COO^- + H^+ + 2H_2$	+48.1
3	Ethanol ⇒ acetate	$CH_3CH_2OH + H_2O \Rightarrow CH_3COO^- + H^+ + 2H_2$	+9.6
4	Lactate ⇒ acetate	$CH_3CHOHCOO^- + 2H_2O \Rightarrow CH_3COO^- + HCO_3^- + H^+ + 2H_2$	−4.2
	Reduction reactions (electron acceptors)		
5	Bicarbonate ⇒ acetate	$2HCO_3^- + 4H_2 + H^+ \Rightarrow CH_3COO^- + 4H_2O$	−104.6
6	Bicarbonate ⇒ methane	$HCO_3^- + 4H_2 + H^+ \Rightarrow CH_4 + 3H_2O$	−135.6
7	Sulfate ⇒ sulfide	$SO_4^{2-} + 4H_2 + H^+ \Rightarrow HS^- + 4H_2O$	−151,9

Source: Adapted from Foresti (1994) and Lettinga *et al.* (1996)

of the table shows the variation of standard free energy (pH equal to 7 and pressure of 1 atm), considering a temperature of 25 °C and the liquid being pure water. All the compounds present in the solution show a 1 mole/kg activity.

In accordance with the examples presented in Table 24.1, it can be clearly noticed that propionate, butyrate and ethanol (reactions 1, 2 and 3) are not degraded under the assumed standard conditions, as the thermodynamic aspects are unfavourable ($\Delta G_o > 0$). However, should the hydrogen concentration be low, the reactions can move to the right (product side). In practice, this is achieved by the continuous removal of H_2 from the medium, by means of electron acceptor reactions (e.g. reactions 5, 6 and 7). In a methanogenic digester operating in an appropriate manner, the partial H_2 pressure does not exceed 10^{-4} atm, and usually this pressure is close to 10^{-6} atm. Under these conditions of low partial hydrogen pressure, propionate, butyrate and ethanol start to degrade and release free energy to the medium. These low partial pressures can only be maintained if the hydrogen formed is quickly and effectively removed by the hydrogen-consuming microorganisms (Lettinga *et al.*, 1996).

24.3.4 Methane formation

Although the individual pathways involved in methane formation are not completely established yet, substantial progress in their understanding has been made in the past decades. Some methanogenic species are capable of using just hydrogen and carbon dioxide for their growth and methane formation, while others are capable of using formic acid, which is previously converted into hydrogen and carbon dioxide. At least two *Methanosarcina* species are capable of forming methane from methanol or acetic acid.

There are two basic mechanisms for methane formation: (i) cleavage of acetic acid and (ii) reduction of carbon dioxide. These mechanisms can be described as follows. In the absence of hydrogen, cleavage of acetic acid leads to the formation of methane and carbon dioxide. The methyl group of the acetic acid is reduced to methane, while the carboxylic group is oxidised to carbon dioxide:

$$C^*H_3COOH \Rightarrow C^*H_4 + CO_2 \qquad (24.1)$$

Microbial group involved: aceticlastic methanogenic organisms

When hydrogen is available, most of the remaining methane is formed from the reduction of carbon dioxide. CO_2 acts as an acceptor of the hydrogen atoms removed from the organic compounds by the enzymes. Since carbon dioxide is always present in excess in an anaerobic reactor, its reduction to methane is not the limiting factor in the process. The methane formation from the reduction of the carbon dioxide is shown below:

$$CO_2 + 4H_2 \Rightarrow CH_4 + 2H_2O \qquad (24.2)$$

Microbial group involved: hydrogenotrophic methanogenic organisms

The overall composition of the biogas produced during anaerobic digestion varies according to the environmental conditions prevailing in the reactor. The composition changes quickly during the initial start-up of the system and also when the digestion process is inhibited. For reactors operating in a stable manner, the composition of the biogas produced is reasonably uniform. However, the carbon dioxide/methane ratio can vary substantially, depending on the characteristics of the organic compound to be degraded. In the anaerobic treatment of domestic sewage, typical methane and carbon dioxide fractions present in the biogas are 70 to 80% and 20 to 30%, respectively.

The methane produced in anaerobic digestion processes is quickly separated from the liquid phase due to its low solubility in water. This results in a high degree of degradation of the liquid wastes, once this gas leaves the reactor to the gaseous phase. On the other hand, carbon dioxide is much more soluble in water than methane, and leaves the reactor partly as gas and partly dissolved in the liquid effluent.

24.3.5 Wastewater characteristics and COD balance

Although practical experience in the anaerobic treatment of liquid effluents is still recent, the potential application of the process can be evaluated from the knowledge of a few chemical characteristics of the waste to be treated. A preliminary evaluation of these characteristics will help choose the most suitable treatment process, allowing an estimation of biological solids production, nutrient requirements, methane production, etc.

Wastewater concentration in terms of biodegradable solids is of fundamental importance, and it can be reasonably estimated from the BOD and COD tests. Another important factor to be considered is the relative concentration of carbohydrates, proteins and lipids, in addition to other important chemical characteristics of the anaerobic biological treatment, especially pH, alkalinity, inorganic nutrients, temperature and the occasional presence of potentially toxic compounds.

(a) COD balance

Hulshoff Pol (1995) presented important and detailed considerations on the COD balance throughout the anaerobic degradation process. According to the author, the compounds present in the wastewater can be classified as of easy, difficult, or impossible degradation. Easily degradable compounds are those that are readily fermented by any type of anaerobic biomass (adapted or not to the waste type). The compounds of difficult degradation, named *complex substrates*, are not fermented by anaerobic microorganisms prior to their adaptation to the substrate. The period of adaptation to the substrate reflects the growth time of specialised microorganisms that can ferment the complex substrate. Lastly, certain organic compounds, known as *inert organic compounds*, are absolutely impossible to biologically degrade in anaerobic environments.

Biodegradable COD. Biodegradable COD (COD_{bd}) is a means of expressing the sewage treatability, which is defined as the total COD (COD_{tot}) portion present in the waste that can be biologically degraded in anaerobic conditions. The sewage biodegradability percentage is given by:

$$\%COD_{bd} = \frac{COD_{bd}}{COD_{tot}} \times 100 \qquad (24.3)$$

where:

$\%COD_{bd}$ = percentage of biodegradable COD (%)
COD_{bd} = concentration of biodegradable COD (mg/L)
COD_{tot} = concentration of total COD (mg/L)

Acidifiable COD. In an anaerobic reactor, the raw sewage provides the fermentative bacteria with non-acidified biodegradable substrate (COD_{bd}). This substrate is consumed by the fermentative microorganisms and converted into cells (COD_{cel}), hydrogen and volatile fatty acids. It is assumed that not all the COD_{bd} will be available for the methanogenic microorganisms, once part of it is converted into new bacterial cells. The COD_{bd} fraction that will be truly available for the methanogenic microorganisms is named acidified COD (COD_{acid}). Thus, the amount of influent biodegradable COD (COD_{inf}) that can be acidified is the sum of the fractions converted into VFA and methane (CH_4). The sewage acidification percentage can then be expressed as follows:

$$\%COD_{acid} = \frac{COD_{CH_4} + COD_{VFA}}{COD_{inf}} \times 100 \qquad (24.4)$$

where:

$\%COD_{acid}$ = percentage of acidified COD (%)
COD_{inf} = biodegradable COD contained in the influent (mg/L)

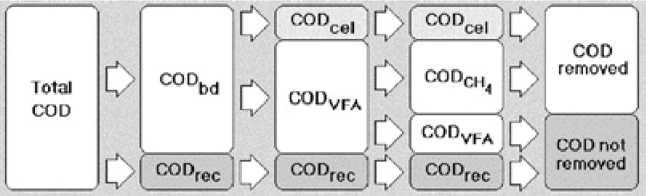


Figure 24.4. Diagram of the COD balance throughout the anaerobic degradation process

COD_{CH_4} = fraction of influent COD converted into methane (mg/L)
COD_{VFA} = fraction of COD still present as volatile fatty acids in the effluent (mg/L)

Recalcitrant COD. The recalcitrant COD (also named biologically resistant COD (COD_{rec})) refers to the portion of organic substrate that cannot be degraded by the fermentative microorganisms. The COD_{rec} is due to the complex substrate subjected to treatment in anaerobic reactors containing biomass not yet adapted to the complex substrate, or to the substrate considered biologically inert. Hence, the COD_{rec} is not fermented, and left biologically unaffected in the treated effluent. Figure 24.4 shows the COD balance throughout the anaerobic degradation process.

Soluble and particulate COD. Most of the compounds present in the raw sewage are not originally soluble and, added to the cells produced during the COD_{bd} degradation process, they form the portion of insoluble or particulate COD (COD_{part}). The COD solubility is usually known by means of laboratory analyses, and it may be presented in three types:

- **Filtered COD (COD_{filt})**. It is due to the presence of dissolved organic compounds in a sewage sample. The COD_{filt} is determined by using the portion of sample that passes through a paper filter of known pore size (1.5 µm). Alternatively to filtration, the sample can be centrifuged (5,000 rpm for 5 minutes), and the COD_{filt} from the supernatant liquid can be determined.
- **Particulate COD (COD_{part})**. It is due to the presence of suspended organic solids contained in a sewage sample. The COD_{part} is obtained as the difference between the total COD (sample neither filtered nor centrifuged) and the COD_{filt}, that is, the particulate COD is due to the solids which do not pass through the filter paper or that remain at the bottom of the recipients after the centrifugation stage.
- **Soluble COD (COD_{sol}):** The COD_{filt} of a sewage sample includes both the portion due to the dissolved particles (totally soluble) and the portion due to the presence of colloidal particles. The latter, responsible for the turbidity, is not removed by the conventional filtration or centrifugation

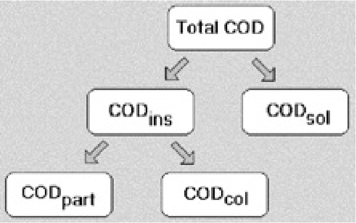


Figure 24.5. Classification of the sewage COD according to solubility

methods. This way, the real COD_{sol} consists of the portion of COD_{filt} that passes through a membrane filter.

Based on these considerations, the following relations can be established (see also Figure 24.5):

$$COD_{tot} = COD_{part} + COD_{col} + COD_{sol} \tag{24.5}$$

$$COD_{ins} = COD_{part} + COD_{col} \tag{24.6}$$

$$COD_{fil} = COD_{col} + COD_{sol} \tag{24.7}$$

Hydrolysable COD. Sewage usually contains organic polymers that need to be converted into simpler substrates (monomers) before being fermented. These organic compounds constitute the portion of hydrolysable COD, and the percentage of effectively hydrolysed insoluble COD is given by:

$$\%COD_{hid} = \frac{COD_{sol} + COD_{cel} + COD_{CH_4}}{COD_{ins}} \times 100 \tag{24.8}$$

where:

$\%COD_{hid}$ = percentage of hydrolysed COD (%)
COD_{sol} = fraction of soluble COD (including the volatile fatty acids) (mg/L)
COD_{cel} = fraction of COD converted into new fermentative bacteria cells (mg/L)
COD_{CH_4} = fraction of COD converted into methane (mg/L)
COD_{ins} = fraction of insoluble COD (particulate substrate) (mg/L)

(b) COD removal

The removal of COD in an anaerobic reactor may occur in two ways:

Biological COD removal

The elimination of soluble COD in the system refers to the difference between the influent COD and the effluent COD, and the COD removal percentage is expressed by:

$$\%COD_{remov} = \frac{COD_{inf} - COD_{eff}}{COD_{inf}} \times 100 \tag{24.9}$$

where:

$\%COD_{remov}$ = percentage of COD removed (%)
COD_{inf} = concentration of influent COD (mg/L)
COD_{eff} = concentration of effluent COD (mg/L)

Considering that the total COD of the effluent comprises the particulate COD due to the microorganism cells, there is generally a greater significance in working with the filtered COD of the effluent, which enables the identification of the COD fraction used for cellular growth as follows:

$$\%COD_{cel} = \frac{\%removalCOD_{fil} - \%COD_{CH_4}}{\%removalCOD_{fil} + \%COD_{VFA}} \times 100 \tag{24.10}$$

where:

$\%COD_{cel}$ = percentage of COD converted into new cells (%)
$\%removal\ COD_{fil}$ = percentage of removal of filtered COD related to the influent soluble COD (%)
$\%COD_{CH_4}$ = percentage of COD converted into methane (%)
$\%COD_{VFA}$ = percentage of influent COD still present as VFA in the effluent (%)

When the influent COD is already acidified, that is, already converted into volatile fatty acids, the elimination percentage of filtered COD is approximately equal to the percentage of COD converted into methane, since the yield coefficient of the methanogenic microorganisms is very low.

The preceding considerations refer to the biological removal of soluble COD. The evaluation of the biological removal of insoluble COD (particulate) is more difficult, since the portion of particulate COD non-hydrolysed and non-degraded in the system cannot be distinguished from the bacterial cells present in the effluent.

Non-biological removal of COD

Non-biological mechanisms of removal of soluble COD usually occur in biological sewage treatment systems, through their incorporation either in the sludge or in the particulate fraction lost with the effluent. In these cases, the percentage of removal of filtered COD will include a portion of COD eliminated by non-biological insolubility. Two main mechanisms contribute to that: precipitation and adsorption:

Precipitation usually results from changes in the pH or from the addition of calcium-based alkaline compounds, for pH control. The precipitates can settle, and then be incorporated into the sludge or be taken out from the system together with the effluent COD.

Adsorption consists in a reaction where the soluble COD is adsorbed on the surface of the biomass particles present in the system. The most important example in practice is the fat adsorption on the bacterial sludge.

In addition, a portion of insoluble COD (particulate) can be removed by non-biological mechanisms, by means of its retention in the sludge. Such retention occurs because the sludge bed can act as a “filter” or because the particulate material can have good settleability.

In the specific case of UASB reactors (see Chapter 27), or of any other anaerobic system that depends on the immobilisation of active biomass, the accumulation of insoluble COD on the sludge bed can be harmful to the process. This accumulation causes the formation of non-bacterial sludge which, if in excess, can cause dilution of the population of methanogenic microorganisms in the sludge, thus reducing the methanogenic activity.

24.3.6 Wastewater degradation and methane production

As described in Section 24.2, anaerobic digestion can be considered a two-phase process. In the first phase, a diversity of fermentative bacteria initially converts the complex organic compounds into soluble compounds and, at last, into short-chain volatile fatty acids. In the second phase, the methanogenic microorganisms use the products fermented in the first phase and convert them into methane. If hydrogen is not produced in the first phase, the fermentation stage results in an insignificant reduction of COD, once all the electrons released in the oxidation process of the organic compounds are transferred to organic acceptors, which remain in the medium. Hence, even though the fermentation stage enables the conversion of part of the energy source into carbon dioxide and of part of the organic matter into new cells, it is considered an inadequate process for both the return of organic carbon to the atmosphere and its removal from the wastewater. However, when hydrogen is formed, it represents a gaseous product that escapes from the medium, causing, therefore, a reduction in the energy content of the wastewater.

Many of the acids and alcohols produced in the initial fermentation phase are converted into a highly insoluble gas, methane, that escapes from the medium, thus favouring the main mechanism for recycling of the organic carbon under anaerobic conditions. Except for the losses caused by microbial inefficiency, almost all the energy removed from the system is recovered in the form of methane gas. However, the formation of methane does not complete the carbon cycle, unless it is oxidised into carbon dioxide, either biologically or by combustion, to become available for recycling by photosynthesis.

(a) Estimation of methane production considering the chemical composition of the waste

Knowing the chemical composition of the wastewater enables an estimation of the amount of methane to be produced and, consequently, of the amount of degraded organic matter. The Buswell stoichiometric equation is used to estimate the production of methane from a given chemical composition of the wastewater:

$$C_n\,H_a\,O_b\,N_d + \left(n - \frac{a}{4} - \frac{b}{2} + \frac{3d}{4}\right) H_2O$$
$$\Rightarrow CH_4 + \left(\frac{n}{2} - \frac{a}{8} + \frac{b}{4} + \frac{3d}{8}\right) CO_2 + (d)\,NH_3 \tag{24.11}$$

In this equation, $C_nH_aO_bN_d$ represents the chemical formula of the biodegradable organic compound subjected to the anaerobic degradation process, and the production of methane considered herein is the maximum stoichiometrically possible. Neither the use of substrate nor other routes of conversion of organic matter are taken into consideration for the production of bacterial biomass.

In the presence of oxygen (less probable) or of specific inorganic donors (such as nitrate, sulfate or sulfite), the production of methane will decrease, according to the following equations (Lettinga *et al.*, 1996):

$$10H + 2H^+ + 2NO_3{}^- \Leftrightarrow N_2 + 6H_2O \tag{24.12}$$

(considering the presence of nitrate in the wastewater)

$$8H + SO_4{}^{2-} \Leftrightarrow H_2S + 2H_2O + 2OH^- \tag{24.13}$$

(considering the presence of sulfate in the wastewater)

Equation 24.13 shows that the reduced sulfate in an anaerobic reactor leads to the formation of H_2S, a gas that dissolves much more in water than does CH_4. Therefore, the partial permanence of H_2S in the liquid phase will imply a smaller reduction of the influent COD, when compared to the treatment of wastewaters not containing sulfate (see Section 24.3.7). According to the Buswell equation, the amount of CO_2 in the biogas can also be much smaller than expected, due to the high solubility of this gas in water.

(b) Estimation of methane production considering the degraded COD

Another method of evaluating the production of methane is from the estimation of the COD degradation in the reactor, according to the following equation:

$$CH_4 + 2O_2 \Rightarrow CO_2 + 2H_2O \tag{24.14}$$
$$(16\text{ g}) + (64\text{ g}) \Rightarrow (44\text{ g}) + (36\text{ g})$$

It can be concluded that one mole of methane requires two moles of oxygen for its complete oxidation to carbon dioxide and water. Therefore, every 16 grams of CH_4 produced and lost to the atmosphere corresponds to the removal of 64 grams of COD from the waste. Under normal temperature and pressure conditions, this corresponds to 350 mL of CH_4 for each gram of degraded COD. The general expression that determines the theoretical production of methane per gram of COD removed from the waste is as follows:

$$V_{CH_4} = \frac{COD_{CH_4}}{K(t)} \tag{24.15}$$

where:

V_{CH_4} = volume of methane produced (L)

COD_{CH_4} = load of COD removed from the reactor and converted into methane (gCOD)

K(t) = correction factor for the operational temperature of the reactor (gCOD/L)

$$K(t) = \frac{P \times K}{R \times (273 + T)} \tag{24.16}$$

where:

P = atmospheric pressure (1 atm)
K = COD corresponding to one mole of CH_4 (64 gCOD/mole)
R = gas constant (0.08206 atm·L/mole·°K)
T = operational temperature of the reactor (°C)

Considering that the production of methane can be easily determined in an anaerobic reactor, this is a fast, direct measurement of the conversion degree of the waste and of the efficiency of the treatment system.

Example 24.1

Consider the treatment of a wastewater with the following characteristics:

- temperature: 26 °C
- flow: 500 m^3/d
- composition of the wastewater:
  sucrose ($C_{12}H_{22}O_{11}$) : C = 380 mg/L, Q = 250 m^3/d
  formic acid (CH_2O_2) : C = 430 mg/L, Q = 100 m^3/d
  acetic acid ($C_2H_4O_2$) : C = 980 mg/L, Q = 150 m^3/d

Determine:

(a) The final concentration of the wastewater in terms of COD:

By balancing the oxidation reactions of each of the compounds of the wastewater:

- concentration of COD in the sucrose
  $C_{12}H_{22}O_{11} + 12O_2 \Rightarrow 12CO_2 + 11\,H_2O$
  342 g..........384 gCOD
  380 mg/L.........x gCOD $\Rightarrow$ x = 427 mgCOD/L
- COD load due to the sucrose
  $250\ m^3/d \times 0.427\ kgCOD/m^3 = 106.8\ kgCOD/d$

Example 24.1 (_Continued_)

- concentration of COD in the formic acid
  $CH_2O_2 + 0.5O_2 \Rightarrow CO_2 + H_2O$
  46 g..........16 gCOD
  430 mg/L......x gCOD ⇒ x = 150 mgCOD/L

- COD load due to the formic acid
  $100\ m^3/d \times 0.150\ kgCOD/m^3 = 15.0\ kgCOD/d$

- concentration of COD in the acetic acid
  $C_2H_4O_2 + 2O_2 \Rightarrow 2CO_2 + 2H_2O$
  60 g............64 gCOD
  980 mg/L........x gCOD ⇒ x = 1.045 mgCOD/L

- COD load due to the acetic acid
  $150\ m^3/d \times 1.045\ kgCOD/m^3 = 156.8\ kgCOD/d$

- final concentration of the waste in terms of COD
  Final concentration = Total load/total flow = (106.8 + 15.0 + 156.8 kgCOD/d)/500 m^3/d
  Final concentration = (278.6 kgCOD/d)/(500 m^3/d) = 0.557 kgCOD/m^3 (**557 mgCOD/L**)

(b) The maximum theoretical methane production, assuming the following yield coefficients for acidogenic and methanogenic organisms: $Y_{acid} = 0.15$ and $Y_{methan} = 0.03\ gCOD_{cel}/gCOD_{remov}$.

The maximum theoretical production occurs when the removal efficiency of COD is 100%, and there is no sulphate reduction in the system.

- COD load removed in the treatment system:
  278.6 kgCOD/d (100% efficiency)

- COD load converted into acidogenic biomass:
  $COD_{acid} = Y_{acid} \times 278.6 = 0.15 \times 278.6 = 41.2$ kgCOD/d

- COD load converted into methanogenic biomass:
  $COD_{methan} = Y_{methan} \times (278.6 - 41.2) = 0.03 \times 237.4 = 7.1$ kgCOD/d

- COD load converted into methane:
  COD_{CH_4} = total load − load converted into biomass = 278.6 − 41.2 − 7.1 = 230.3 kgCOD/d

- Estimated production of methane:
  The value of K(t) is determined from Equation 24.16.
  $K(t) = (P \cdot K)/[R \cdot (273 + t)] = (1\ atm \times 64\ gCOD/mole)/[0.0821\ atm \cdot L/mole \cdot K \times (273 + 26\ °C)]$
  K(t) = 2.61 gCOD/L

***Example 24.1* (*Continued*)**

The theoretical production of methane is determined from Equation 24.15.
$V_{CH_4} = COD_{CH_4}/K(t) = (230.3\ \text{kgCOD/d})/(2.61\ \text{kgCOD/m}^3)$
$\mathbf{V_{CH_4} = 88.2\ m^3/d}$

Note: The theoretical production of methane can also be calculated from Equation 24.11. In this case, the theoretical production should be calculated separately for each of the three compounds present in the wastewater, in terms of their concentrations and individual loads removed (not in terms of COD). After that, the following should be done:

- convert the methane load produced into the equivalent COD load (Equation 24.14)
- deduct the COD load converted into acidogenic and methanogenic biomass (as above)
- estimate the volumetric production of methane (Equations 24.15 and 24.16).

24.3.7 Sulfate reduction and methane production

As analysed in Section 24.2, the presence of sulfate in wastewater causes a change in the metabolic pathways in the anaerobic digester (Figure 24.2), in view of a competition for substrate established between the sulfate-reducing bacteria and the fermentative, acetogenic and methanogenic microorganisms. Hence, two final products are formed: methane (by methanogenesis) and sulfide (by sulfate reduction). The magnitude of this competition is related to several aspects, particularly the pH and the COD/SO_4^{2-} ratio in the wastewater. The production of sulfides may cause serious problems during the treatment of these wastewaters (adapted from Lettinga, 1995; Visser, 1995):

- The reduced SO_4^{2-} results in the formation of H_2S, an inhibiting compound for the methanogenic microorganisms that can reduce their activity and the capacity of the anaerobic reactor. In practice, the methanogenic microorganisms become more inhibited only when the COD/SO_4^{2-} ratio is less than 7, but are strongly dependent on the pH. For high COD/SO_4^{2-} ratios (>10), a large portion of the H_2S produced will be removed from the liquid phase, in view of a higher production of biogas, thus reducing its inhibiting effect on the liquid phase.
- Part of the hydrogen sulfide produced passes to the gaseous phase (biogas), which may cause corrosion and bad odour problems. If the biogas is intended to be used, an additional cost should be estimated for its purification.
- The presence of sulfide causes a high demand for oxygen in the effluent, as well as bad odour problems. A post-treatment phase for sulfide removal may be necessary.
- For the same amount of organic material present in the waste, the sulfate reduction decreases the amount of methane produced. A reduction of 1.5 g

of SO_4^{2-} corresponds to the use of 1.0 g of COD, which means a smaller availability for conversion into CH_4 (see Equation 24.17).

The COD used for reduction of the sulfate present in the wastewater can be estimated by the following equation:

$$\begin{array}{l} S^{2-} + 2O_2 \Leftrightarrow SO_4^{2-} \\ (32\text{ g}) + (64\text{ g}) \Rightarrow (96\text{ g}) \end{array} \qquad (24.17)$$

It is noted that 1 mole of SO_4^{2-} requires two moles of oxygen for its reduction to sulfide. Therefore, every 96 g of SO_4^{2-} present in the waste consume 64 g of COD (1.5 SO_4^{2-}:1.0 COD ratio).

24.4 ENVIRONMENTAL REQUIREMENTS

24.4.1 Preliminaries

A natural habitat does not imply an environment unaffected by human activities, but an environment where the species that make up the microbial population are those selected by interaction with the environment and among themselves. Nutritional and physical conditions enable the selection of the organisms better adapted to the environment, which may vary quickly and frequently due to changes in the supply of nutrients or in the physical conditions.

Both physical and chemical characteristics of the environment influence microbial growth. Physical factors usually act as selective agents, while chemical factors can or cannot be selective. Some elements, such as carbon and nitrogen, which are usually required in relatively large amounts, can be very important in the selection of the prevailing species. Micronutrients, which are required in very small amounts, generally have little or no selective influence (Speece, 1986).

Anaerobic digestion is particularly susceptible to the strict control of the environmental conditions, as the process requires an interaction between fermentative and methanogenic organisms. A successful process depends on an accurate balance of the ecological system. Special attention should be given to the methanogenic microorganisms, as they are considered highly vulnerable to changes in the environmental conditions. The main environmental requirements of anaerobic digestion are commented below (Speece, 1983).

24.4.2 Nutrients

The nutritional needs of the microbial populations involved in biological wastewater treatment processes are usually established from the chemical composition of the microbial cells. As the precise composition is rarely known, the nutrient requirements are determined based on the empirical composition of the microbial cells. Such consideration is based on the fact that almost all living cells are formed by similar types of compounds, and that such cells present similar chemical composition, requiring therefore the same elements in the same relative proportions.

Table 24.2. Chemical composition of the methanogenic microorganisms

Macronutrients		Micronutrients	
Element	Concentration (g/kg TSS)	Element	Concentration (mg/kg TSS)
Nitrogen	65	Iron	1,800
Phosphorus	15	Nickel	100
Potassium	10	Cobalt	75
Sulfur	10	Molybdenum	60
Calcium	4	Zinc	60
Magnesium	3	Manganese	20
		Copper	10

Source: Lettinga *et al.* (1996)

The chemical composition of the methanogenic microorganisms is presented in Table 24.2.

According to Lettinga *et al.* (1996), the minimum nutrient requirements can be calculated by the following expression:

$$\mathrm{Nr} = \mathrm{S_0}\cdot\mathrm{Y}\cdot\mathrm{N_{bac}}\cdot\frac{\mathrm{TSS}}{\mathrm{VSS}} \tag{24.18}$$

where:

Nr = nutrient requirement (g/L)
$\mathrm{S_0}$ = concentration of influent COD (g/L)
Y = yield coefficient (gVSS/gCOD)
$\mathrm{N_{bac}}$ = concentration of nutrient in the bacterial cell (g/gVSS)
TSS/VSS = total solids/volatile solids ratio for the bacterial cell (usually 1.14)

For biological treatment processes to be successful, the inorganic nutrients necessary for the growth of microorganisms should be supplied in sufficient amounts. If the ideal concentration of nutrients is not supplied, there should be some form of compensation, either by applying smaller loads to the treatment system, or by allowing a reduced efficiency of the system. The presence or absence of micronutrients in the wastewater is generally evaluated by a laboratory survey. Sometimes, the combined treatment of several types of wastewater can compensate for the lack of micronutrients in some wastes.

Domestic sewage generally presents all appropriate types of nutrients in suitable concentrations, thus providing an ideal environment for the growth of microorganisms, with no limitations for the anaerobic digestion process. A possible exception is the availability of sufficient iron in sludge generated in domestic sewage treatment, which may limit the methanogenic activity. On the other hand, industrial effluents are more specific in composition and may require a nutrient supplementation for an ideal degradation.

The following nutrients, in decreasing order of importance, are necessary for the nutritional stimulation of methanogenic microorganisms: nitrogen, sulfur, phosphorus, iron, cobalt, nickel, molybdenum, selenium, riboflavin and vitamin B12.

(a) Nitrogen

Generally, nitrogen is the inorganic nutrient required in larger concentrations for the growth of microorganisms. Under anaerobic conditions, nitrogen in the forms of nitrite and nitrate is not available for bacterial growth, as it is reduced to nitrogen gas and released to the atmosphere. Ammonia and the fraction of organic nitrogen released during degradation are the main sources of nitrogen used by microorganisms.

As bacteria grow much more in wastes containing large amounts of carbohydrates than they do in wastes containing proteins and volatile acids, the nitrogen needs for the first type of waste may be about six times larger than those for the volatile acid-based wastes due to an increased synthesis of the fermentative bacteria.

Nitrogen requirements are based on the empirical chemical composition of the microbial cell, according to Table 24.2. Although an estimation of the nutrient requirements based on the sewage concentration is not the most suitable one, most of the guidelines contained in the specialised literature refer to a COD-based supplementation of nutrients. According to Lettinga *et al.* (1996), assuming that the nutrients present in sewage are in a form available to the bacteria, the following relations can be used:

- *Biomass with low yield coefficient ($Y \sim 0.05$ gVSS/gCOD)*
  e.g. degradation of volatile fatty acids
  COD:N:P = 1000:5:1
  C:N:P = 330:5:1

- *Biomass with high yield coefficient ($Y \sim 0.15$ gVSS/gCOD)*
  e.g. degradation of carbohydrates
  COD:N:P = 350:5:1
  C:N:P = 130:5:1

(b) Phosphorus

Microbial incorporation of phosphorus in anaerobic digestion has been reported as being approximately 1/5 to 1/7 of that established for nitrogen. Most of the microorganisms are capable of using inorganic orthophosphate, which can be incorporated by the growing cells by means of the mediation of enzymes named phosphatases.

(c) Sulfur

Most of the methanogenic microorganisms use sulfide as a source of sulfur, although some of them can use cysteine. If inorganic sulfate is present, it is reduced to sulfide, which then reacts with the serine amino acid to form sulfur containing the cysteine amino acid. Sulfur is necessary for the synthesis of proteins.

In general, the concentration of sulfate in domestic sewage is sufficient to provide the sulfur necessary for the bacterial growth, which is required in relatively small amounts. This is due to the low sulfur content in the microbial cells. Other reasons to disregard the need for sulfides in anaerobic digestion include: (i) presence of H_2S in the biogas; (ii) microbial synthesis of sulfide and (iii) precipitation of sulfides by metals.

Sulfur and phosphorus seem to be required in the same amount. It should be emphasised that sulfur requirements for methanogenic microorganisms are part of a complex process. On one hand, the presence of sulfates can limit the methanogenesis, because the sulfate-reducing bacteria compete for substrates such as hydrogen and acetate. On the other hand, the methanogenic organisms depend on the production of sulfides for their growth. This illustrates the relatively narrow ecological environment occupied by the methanogenic organisms, where some inorganic compounds pass from ideal to toxic concentrations within a small range.

Example 24.2

Calculate the nitrogen and phosphorus requirements of an anaerobic treatment system with the following characteristics:

- type of substrate: carbohydrate
- concentration of the influent substrate: $S_0 = 0.350$ gCOD/L
- yield coefficient: $Y = 0.15$ gVSS/gCOD
- TSS/VSS ratio of the bacterial cell: 1.14
- concentration of nutrients in the bacterial cell: 0.065 gN/gTSS; 0.015 gP/gTSS (Table 24.2)

Solution:

- Calculation of the nitrogen requirement

  Using Equation 24.18:
  $Nr = 0.350$ gCOD/L × 0.15 gVSS/gCOD × 0.065 gN/gTSS
  × 1.14 gTSS/gVSS
  $Nr = 0.0039$ gN/L (3.9 mgN/L)

- Calculation of the phosphorus requirement

  Using Equation 24.18:
  $Nr = 0.350$ gCOD/L × 0.15 gVSS/gCOD × 0.015 gP/gTSS
  × 1.14 gTSS/gVSS
  $Nr = 0.0009$ gP/L (0.9 mgP/L)

- Determination of the COD:N:P ratio

  0.350 gCOD/L:0.0039 gN/L:0.0009 gP/L
  350:3.9:0.9 or (350:4:1)

(d) Micronutrients

Besides nitrogen, phosphorus and sulfur, which, together with carbon and oxygen, constitute the macromolecules of the microbial cells, a large number of other elements are necessary for the anaerobic digestion process. These elements are named micronutrients and comprise the micromolecules of the cells. They represent about 4% of the dry weight of the cells.

It is difficult to determine in practice the exact demand of these micronutrients, once the presence and need for sulfides by the methanogenic organisms lead to the precipitation of these elements from the solution, making the concentration of metals in equilibrium very low. To solve this situation, a pulse application of acidified influent can be performed to disturb the chemical equilibrium and make the metals momentarily available for the methanogenic microorganisms.

Iron, cobalt, nickel and molybdenum are the main micronutrients required by the microorganisms that form methane from acetate.

24.4.3 Temperature

Among the physical factors that affect microbial growth, temperature is one of the most important in the selection of species. Microorganisms are not capable of controlling their internal temperature and, consequently, the temperature inside the cell is determined by the external ambient temperature.

Three temperature ranges can be associated with microbial growth in most of the biological processes (Batstone *et al*., 2002):

- *psycrophilic range*: between 4 and approximately 15 °C
- *mesophilic range*: between 20 and approximately 40 °C
- *thermophilic range*: between 45 and 70 °C, and above

In each of these three ranges, where microbial growth is possible, three temperature values are usually used to characterise the growth of the microorganism species (see Figure 24.6):

- *minimum* temperature, below which growth is not possible
- *optimum* temperature, in which growth is maximum
- *maximum* temperature, above which growth is also not possible

The *maximum* and *minimum* temperatures define the limits of the range in which growth is possible, and the *optimum* temperature is that in which growth rate is maximum. The microbial growth rate at temperatures close to the minimum is typically low, but it increases exponentially as the temperature rises, reaching its maximum close to the ideal temperature. From the ideal growth rate, the increase of a few degrees causes an abrupt drop in the growth rate, to zero value.

The microbial formation of methane may occur in a wide temperature range (0 to 97 °C). Two ideal temperature levels have been associated with the anaerobic digestion, one in the mesophilic range (30 to 35 °C), and another in the thermophilic

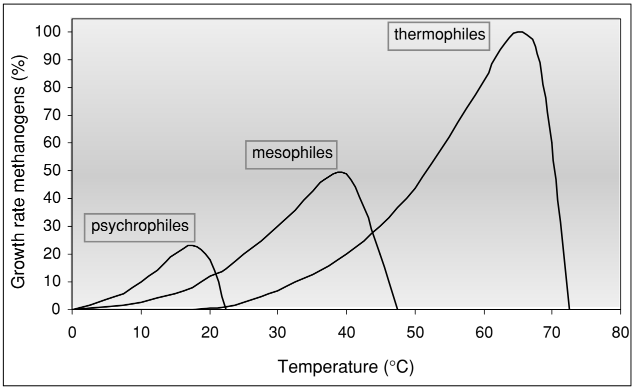


Figure 24.6. Influence of the temperature on the biomass growth rate. Relative growth rate of psychrophilic, mesophilic and thermophilic methanogens (source: adapted from van Lier *et al.*, 1997)

range (50 to 55 °C). Most of the anaerobic digesters have been designed in the mesophilic range, although their operation is also possible in the thermophilic range. However, the operational experience of anaerobic digesters in this range has not been very satisfactory, and many questions are still pending, such as whether the resulting benefits overcome the disadvantages, including the necessary additional energy to heat up the digesters, the poor quality of the supernatant and the instability of the process.

The external effects of temperature on bacterial cells are also important. For example, the degree of dissociation of several compounds depends strongly on the temperature, such as the specific case of ammonia. The thermodynamics of several reactions is also affected by temperature, such as the dependence of the hydrogen pressure in anaerobic digesters where fermentation occurs in an appropriate manner.

The importance of the quantitative data on the effects of the temperature on the microbial population is that a considerable reduction may be achieved in the reactor volume, if it is operated close to the ideal temperature, once the maximum specific growth rate of the microbial population rises as the temperature increases. Although high temperatures are desired, maintaining a uniform temperature in the reactor may be more important, once the anaerobic process is considered very sensitive to abrupt temperature changes, which may cause an unbalance between the two largest microbial populations and the consequent failure of the process (the usual limit is about 2 °C per day).

The methane-forming microorganisms prevailing in anaerobic digesters operated in the mesophilic temperature range belong to the genera *Methanobacterium*, *Methanobrevibacter* and *Methanospirillum*, which are hydrogen-using organisms, and to the genera *Methanosarcina* and *Methanosaeta*, which are organisms that use acetate to form methane.

The temperature affects the biological processes in two ways: (i) influencing the enzymatic reaction rates; and (ii) influencing the substrate diffusion rates. Although diffusion is an important factor, particularly in full-scale reactors, only the effects of temperature related to the reaction rates are discussed in this item.

The data found in the specialised bibliography indicate that K_s and Y decrease as the temperature increases, while the K_d coefficient of low-growth-rate cultures is little affected by temperature (Grady and Lim, 1980).

The Arrhenius equation is frequently used to quantify the effects of temperature on biochemical reactions:

$$K = K_o \cdot e^{\left(\frac{-E}{R \cdot T_{abs}}\right)} \tag{24.19}$$

where:

K = reaction rate
K_o = constant
E = activation energy (cal/mole)
R = gas constant (1.98 cal/mole · K)
T_{abs} = absolute temperature (K)

According to the experimental data available, μ_{max} increases as the temperature rises, until a maximum growth value is reached. From this maximum value, μ_{max} decreases quickly. This decrease results from two competitive processes: (i) bacterial synthesis; and (ii) bacterial decay, each represented by the Arrhenius equation, so that the net growth rate can be expressed as follows:

$$K_{net} = K_1 \cdot e^{\left(\frac{-E_1}{R \cdot T_{abs}}\right)} - K_2 \cdot e^{\left(\frac{-E_2}{R \cdot T_{abs}}\right)} \tag{24.20}$$

where:

K_{net} = net growth rate
K_1 = bacterial synthesis rate
K_2 = bacterial decay rate

As the temperature increases, the inactivation and denaturation of enzymes and proteins become very important, until the net growth rate reaches a maximum. Above the ideal temperature, the growth rate falls suddenly, when the decay begins to prevail over synthesis.

According to Henze and Harremoës (1983), the maximum bacterial growth rate decreases 11% per °C, for anaerobic digesters operated at temperatures below 30 °C, as shown by the following expression (van Haandel and Lettinga, 1994):

$$K(t) = K_{30} \times 1.11^{(T-30)} \tag{24.21}$$

where:

$K(t)$ = growth rate for the temperature (T)
K_{30} = growth rate for t = 30 °C
T = temperature (°C)

24.4.4 pH, alkalinity and volatile acids

These three environmental factors are closely related to each other, being equally important to the control and suitable operation of anaerobic processes. The pH affects the process in two main ways (Lettinga *et al.*, 1996):

- *directly*: affecting, for example, the enzymes' activity by changing their proteic structure, which may occur drastically as a result of changes in the pH
- *indirectly*: affecting the toxicity of a number of compounds (see Section 24.5.5)

The methane-producing microorganisms have optimum growth in the pH range between 6.6 and 7.4, although stability may be achieved in the formation of methane in a wider pH range, between 6.0 and 8.0. pH values below 6.0 and above 8.3 should be avoided, as they can inhibit the methane-forming microorganisms. The optimum pH depends on the type of microorganisms involved in the digestion process, as well as on the type of substrate. Table 24.3 presents values of optimum pH ranges for the degradation of different substrates.

Regarding the stability of the process, the fact that the acid-producing bacteria are much less sensitive to pH than the methanogenic microorganisms is particularly important, as the acidogenic bacteria can still be very active, even for pH values as low as 4.5. In practice, this means that the production of acids in a reactor can continue freely, although the methane production has been practically interrupted due to the low pH values. As a result, the reactor contents will become "sour".

The acid-producing bacteria have an optimum growth rate in the pH range between 5.0 and 6.0, with a higher tolerance to lower pH values. Therefore, pH control aims mainly at eliminating the risk of inhibition of the methanogenic microorganisms by the low pH values, thus avoiding the failure of the process.

The operation of an anaerobic reactor with the pH constantly below 6.5 or above 8.0 can cause a significant decrease in the methane production rate. In addition, sudden pH changes (pH shocks) can adversely affect the process, and recovery will depend on a series of factors, related to the type of damage caused to the microorganisms (either permanent or temporary). According to Lettinga *et al.*

Table 24.3. Optimum pH ranges for the degradation of different substrates

Substrate	Optimum pH
Formiate	6.8 to 7.3
Acetate	6.5 to 7.1
Propionate	7.2 to 7.5

Source: Lettinga *et al.* (1996)

(1996), the recovery will be quicker if:

Acid pH shock	*Alkaline pH shock*
1. The pH drop was not significant.	1. The pH rise was not significant.
2. The pH shock had a short duration.	2. The pH shock had a short duration.
3. The VFA concentration during the pH shock remained low.	

(a) Alkalinity and buffer capacity

The buffer capacity can be understood as the capacity of a solution to avoid changes in the pH. A buffer solution consists of a mixture of a weak acid and its corresponding salt, thus enabling the grouping of the ions H^+ and OH^- and avoiding both the increase and the decrease of the pH. The following generic equations are applied:

$$HA + H_2O \Leftrightarrow H_3O^+ + A^- \tag{24.22}$$

$$K_A = \frac{\left[H_3O^-\right].\left[A^-\right]}{[HA]} \tag{24.23}$$

$$pH = pK_A + \log \frac{\left[A^-\right]}{[HA]} \tag{24.24}$$

The buffer capacity reaches its maximum when $pH = pK_A$, that is, when $[A^-] = [HA]$.

The two main factors that affect the pH in anaerobic processes are carbonic acid and volatile acids. In the pH range between 6.0 and 7.5, the buffer capacity of the anaerobic system depends almost completely on the carbon dioxide/alkalinity system, which, in equilibrium with the dissociation of the carbonic acid, tends to regulate the concentration of the hydrogen ion, as explained below.

The amount of carbonic acid in solution is directly related to the amount of CO_2 in the gaseous phase, once a balance is established between the amounts of CO_2 in the liquid phase and in the gaseous phase. The portion of CO_2 dissolved in the liquid phase can be established by Henry's law:

$$[CO_2] = K_H \cdot P_{CO_2} \tag{24.25}$$

where:

$[CO_2]$ = saturation concentration of CO_2 in water (mole)
K_H = constant of Henry's law related to the CO_2-water balance (mole/atm·L)
P_{CO_2} = CO_2 partial pressure (atm)

The relation between alkalinity and pH is then given by the following expression (Foresti, 1994; Lettinga *et al.*, 1996):

$$pH = pK_1 + \log \frac{\left[HCO_3^-\right]}{[H_2CO_3^*]} \tag{24.26}$$

where:

$pK_1 = \log (1/K_1)$

K_1 = constant of apparent ionisation (4.45×10^{-7}, at 25 °C), that is related to all the CO_2 dissolved in the liquid

$$\left[H_2CO_3^*\right] = [CO_2] + [H_2CO_3] \cong [\sim CO_2(\text{líq})] \tag{24.27}$$

Hence, the portion of $H_2CO_3^*$ can be obtained by calculating the partial carbon dioxide gas pressure, according to Equation 24.25.

(b) Interaction between alkalinity and volatile acids

The interaction between alkalinity and volatile acids during anaerobic digestion is based on whether the alkalinity of the system is able to neutralise the acids formed in the process and buffer the pH in case of accumulation of volatile acids. Both the alkalinity and the volatile acids derive primarily from the decomposition of organic compounds during digestion, as follows:

- Conversion of intermediate volatile fatty acids. The digestion of sodium acetate, for example, can lead to the formation of sodium bicarbonate

$$CH_3COONa + H_2O \Rightarrow CH_4 + CO_2 + NaOH \Rightarrow CH_4 + NaHCO_3 \tag{24.28}$$

- Conversion of proteins and amino acids, with formation of ammonia (NH_4^-). The combination between ammonia and carbonic acid in solution leads to the formation of ammonia bicarbonate

$$NH_3 + H_2O + CO_2 \Rightarrow NH_4^+ + HCO_3^- \tag{24.29}$$

Digestion of other organic compounds that do not lead to a cation as final product does not produce alkalinity. This occurs, for example, in the degradation of carbohydrates and alcohols. This is particularly important due to the high microbial synthesis during the degradation of carbohydrates, which could result in the depression of alkalinity, should the present ammonia bicarbonate be used as source of nitrogen for biological synthesis.

(c) Alkalinity of the volatile acids

As a result of the reaction of the alkalinity with the volatile fatty acids produced in the system, the bicarbonate alkalinity is converted into alkalinity of volatile acids, because volatile acids are stronger than bicarbonates. However, the alkalinity buffering capacity of the volatile acids is situated in the pH range between 3.75 and 5.75, being, therefore, of little importance in anaerobic digestion. Consequently, a supplementation of the bicarbonate alkalinity lost in the reaction with the volatile acids should be provided.

In practice, for calculation of the bicarbonate alkalinity, the portion corresponding to the alkalinity of the volatile acids should be discounted from the total alkalinity, as follows (Foresti, 1994):

$$BA = TA - 0.85 \times 0.83 \times VFA = TA - 0.71 \times VFA \qquad (24.30)$$

where:

BA = bicarbonate alkalinity (as $mgCaCO_3/L$)
TA = total alkalinity (as $mgCaCO_3/L$)
VFA = concentration of volatile fatty acids (as mg acetic acid/L)
0.85 = correction factor that considers 85% of ionisation of the acids to the titration end point
0.83 = conversion factor from acetic acid into alkalinity

(d) Monitoring of alkalinity

In the monitoring of anaerobic reactors, the systematic verification of the alkalinity becomes more important than the evaluation of the pH. This is due to the logarithmic scale of pH, meaning that small pH decreases imply the consumption of a large amount of alkalinity, thus reducing the buffering capacity of the medium.

To determine separately the portions of bicarbonate alkalinity and of alkalinity of the volatile acids, the titration of the sample can be performed in two stages, according to the methodology proposed by Ripley *et al.* (1986):

- *titration up to pH 5.75*: the first stage of titration provides the *partial alkalinity* (PA), practically equivalent to the bicarbonate alkalinity
- *titration up to pH 4.3*: the second stage of titration provides the *intermediate alkalinity* (IA), practically equivalent to the alkalinity of the volatile acids

An important aspect of determining the alkalinity in two stages refers to the significance of the IA/PA ratio. According to Ripley *et al.* (1986), IA/PA values higher than 0.3 indicate the occurrence of disturbances in the anaerobic digestion process. The stability of the process is possible for IA/PA values different from 0.3, and the verification of each particular case is recommended (Foresti, 1994).

(e) Alkalinity necessary for the process

From the operational point of view, if the alkalinity is generated from the influent sewage, the maintenance of high levels of alkalinity in the system is desirable because high concentrations of volatile acids could be buffered without causing a substantial drop in pH. However, if an alkalinity supplementation is necessary, then the selection of chemical compounds shall be evaluated in terms of applicability and economy. The minimum acceptable alkalinity requirement depends on the concentration of the sewage, a decisive factor to determine the potential generation of acids in the system.

According to van Haandel and Lettinga (1994), the most important issue related to the pH value and stability is whether the alkalinity of the medium (influent alkalinity+generated alkalinity) is sufficient to keep itself at levels considered safe. The authors present a complete methodology, relating the determination of the pH and alkalinity in anaerobic digesters.

(f) Chemical products for alkalinity supplementation

Several chemical products can be used to control the pH of anaerobic processes, including hydrated lime ($Ca(OH)_2$), quicklime (CaO), sodium carbonate (Na_2CO_3), sodium bicarbonate ($NaHCO_3$), sodium hydroxide (NaOH) and ammonia bicarbonate (NH_4HCO_3). These chemical products can be separated into two groups:

- those that provide bicarbonate alkalinity directly (NaOH, $NaHCO_3$, NH_4HCO_3)
- those that react with carbon dioxide to form bicarbonate alkalinity (CaO, $Ca(OH)_2$, NH_3)

Lime is usually the cheapest source of alkalinity but, as it is a very insoluble product, it can cause serious operational problems. Carbon dioxide reacts with lime to form calcium bicarbonate, which can cause vacuum in closed digesters. If the carbon dioxide present is insufficient to react entirely with lime, the final pH may be very high, which can be as harmful as a very low pH. The formation of undesirable precipitates and fouling can cause serious operational problems.

Sodium bicarbonate is easy to handle, is very soluble and, unlike lime, it neither requires carbon dioxide nor increases pH substantially when excessively dosed. However, the cost of the product is very high.

The use of ammonia as a source of alkalinity depends substantially on the local conditions. For example, the use of anhydrous ammonia, in spite of it being cheap, may be prohibitive because the effluent will contain an excessive amount of ammonia. Besides that, care should be taken to prevent biomass toxicity by ammonia.

24.4.5 Toxic materials and their control

The appropriate degradation of organic sewage by any biological process depends on the maintenance of a favourable environment for microorganisms, including either the control or the elimination of toxic materials. Since any compound present in sufficiently high concentrations can be toxic, the toxicity should be discussed in terms of toxic levels, instead of toxic materials. In this aspect, according to Speece *et al.* (1986), the following considerations are pertinent:

- What are the required concentrations that cause toxicity?
- Is the toxic effect reversible or bactericide?
- What is the acclimatisation potential of the microorganisms?

Toxicity has been considered one of the main reasons for a non-generalised use of anaerobic digestion, once there is a widespread understanding that anaerobic processes are not capable of tolerating toxicity. It is true that methanogenic microorganisms can be more easily inhibited by toxins, due to the relatively small fraction of substrate converted into cells and to the long generation period of these microorganisms. However, microorganisms usually have a certain capacity of adaptation to the inhibiting concentrations of most of the compounds, provided that the toxicity impact minimised by some design measures, such as long solids retention time and minimised residence time of toxins in the system. The following control methods for toxic materials were suggested by McCarty (1964):

- removal of the toxic materials present in the sewage
- dilution below the toxic limit
- formation of insoluble complexes or precipitation
- antagonism of toxicity by means of the use of another compound

Several organic and inorganic compounds can be toxic or inhibitors to the anaerobic process, although the general effect resulting from the addition of most of them may vary from stimulating to toxic. Microbial activity is usually stimulated at low concentrations, but it also depends on the type of compound present. As the concentration is increased, inhibition may become high, and the rate of microbial activity may fall to zero.

(a) Toxicity by salts

Toxicity by salts is usually associated with the cation, and not with the anion of the salt. Cation toxicity assessments carried out by Kugelman and McCarty (1965) indicated the following increasing order of inhibition, based on the molar concentration: Na^+ (0.32 M), $NH_4{}^+$(0.25 M), K^+ (0.15 M), Ca^{2+} (0.11 M) and Mg^{2+} (0.08 M). However, more recent studies have shown that the inhibiting concentrations can be higher, provided that the biomass undergoes an adaptation stage (Lettinga *et al.*, 1996).

If some cation is found at an inhibiting concentration in the influent sewage, inhibition can be reduced if an antagonistic ion is either present or added to the system. Sodium and potassium are the best antagonists for that purpose, provided that they are used in stimulating concentrations, as indicated in Table 24.4. Antagonistic elements are usually added by means of chloride salts.

Table 24.4. Stimulating and inhibiting concentrations of some cations

	Concentration (mg/L)		
Cation	Stimulating	Moderately inhibiting	Strongly inhibiting
Calcium	100 to 200	2,500 to 4,500	8,000
Magnesium	75 to 150	1,000 to 1,500	3,000
Potassium	200 to 400	2,500 to 4,500	12,000
Sodium	100 to 200	3,500 to 5,500	8,000

Source: McCarty (1964)

(b) Toxicity by ammonia

Usually, the presence of ammonia bicarbonate, resulting from the digestion of sewage rich in urea- or protein-based compounds, is beneficial to the digester as a source of nitrogen and as a buffer for pH changes. However, both the ammonium ion (NH_4^+) and the free ammonia (NH_3) can become inhibitors when present in high concentrations. These two forms of ammonia are balanced, with the relative concentration of each depending on the pH of the medium, as indicated in the following equation:

$$NH_4^+ \Leftrightarrow NH_3 + H^+ \tag{24.31}$$

For high concentrations of hydrogen ion (pH equal to or lower than 7.2), the balance shifts to the left, so that inhibition becomes related to the concentration of the ammonium ion (see Section 2.2.3.4). For higher pH levels, the concentration of hydrogen ion decreases, and the balance shifts to the right. In this situation, free ammonia may become the inhibiting agent. Studies have shown that concentrations of free ammonia above 150 mg/L are toxic to the methanogenic microorganisms, while the maximum safety limit for the ammonium ion is approximately 3,000 mg/L. The concentrations of free ammonia that can have either a beneficial or an adverse effect on anaerobic processes are presented in Table 24.5.

(c) Toxicity by sulfide

Toxicity by sulfide is a potential problem in anaerobic treatment, firstly due to the biological reduction of sulfates and organic sulfur-containing compounds, and also for the anaerobic degradation of protein-rich compounds. As covered in Sections 24.3.6 (Equation 24.13) and 24.3.7, the reduced sulfate leads to the formation of H_2S, which dissociates in water, in accordance with the following equations (Jansen, 1995):

$$H_2S \Leftrightarrow H^+ + HS^- \tag{24.32}$$

$$HS^- \Leftrightarrow H^+ + S^{2-} \tag{24.33}$$

The dissociation of species is related to the temperature and to the pH of the medium, in accordance with the distribution diagram shown in Figure 24.7,

Table 24.5. Effects of free ammonia on anaerobic processes

Concentration (as N, mg/L)	Effect
50 to 200	Beneficial
200 to 1,000	No adverse effect
1,500 to 3,000	Inhibitor for pH > 7.4 to 7.6
Above 3,000	Toxic

Source: McCarty (1964)

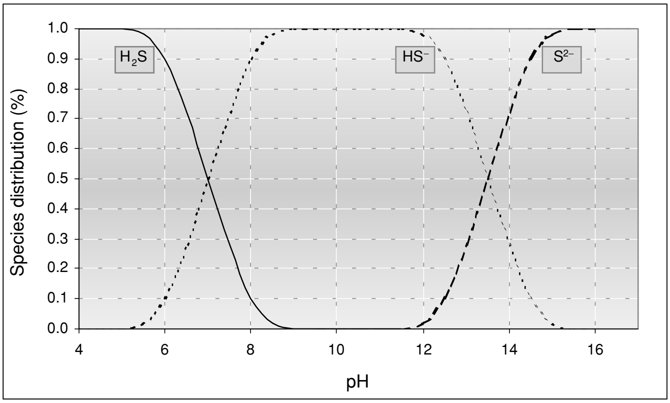


Figure 24.7. Distribution diagram for H_2S (T = 25 °C)

developed for a temperature of 25 °C. From the analysis of the diagram, it can be concluded that:

- the un-ionised form (H_2S) is the main dissolved component for pH values lower than 7
- the ionised form (HS^-) prevails for pH values between 7 and 14
- the concentration of free sulfide (S^{2-}) is negligible in the pH range associated with sewage treatment

Inhibition by sulfide is dependent on the concentration of non-dissociated hydrogen sulfide (H_2S) in the medium, which indicates that the inhibition by sulfide is strongly dependent on pH, within the pH range usually associated with anaerobic digestion (6.5 to 8). The distribution diagram shows that, for a pH value equal to 7, around 50% of the sulfide will be present in the most toxic, non-dissociated form (H_2S) and the other 50% in the less toxic, dissociated form (HS^-). On the other hand, H_2S can still be either present in the gaseous phase (H_2S_{gas}) or dissolved in the liquid phase (H_2S_{liq}). The higher or lower presence of sulfides in the gaseous phase will strongly depend on the gas production in the system. The greater the production of CH_4 in the reactor, the larger the amount of sulfides in the gaseous form removed from the liquid phase. Consequently, the toxicity of H_2S will decrease as the concentration of influent COD increases (larger production of CH_4). It is generally assumed that, for a COD/SO_4^{2-} ratio higher than 10, toxicity problems will not occur in the anaerobic reactor.

From the practical point of view, it is important to determine the sensitivity of the biomass to sulfide. The amount of sulfides produced in the anaerobic treatment depends on the following main factors:

- COD/SO_4^{2-} ratio in the influent (a low ratio results in a high sulfide production)
- composition of the organic substrate

- pH and temperature of the medium
- result of the competition between sulfate-reducing and methanogenic microorganisms

For the design and operation of anaerobic reactors, it is important to know the maximum allowable concentration of non-dissociated H_2S. According to the literature, anaerobic reactors with a high biomass retention capacity (e.g. UASB reactors and anaerobic filters) can tolerate higher levels of sulfide, amounting approximately to 170 mg H_2S/L (Speece, 1986). Sulfides in the form of H_2S become very toxic when present in concentrations above 200 mg/L, but they can be tolerated up to this concentration if the operation of the system is continuous and if the biomass undergoes some acclimatisation. Sulfide concentrations amounting to 50 to 100 mg/L can be tolerated with little or no system acclimatisation.

If the sulfide concentration in the reactor exceeds the maximum tolerable values, special measures should be taken to ensure a good performance of the system:

- increase pH in the reactor, so that the dissociation of H_2S in the liquid phase favours the formation of HS^-. From Figure 24.7, only 10% of the sulfide will be present in non-dissociated form if the pH in the reactor is equal to 8
- dilute the influent, aiming at reducing the concentration of sulfides in the reactor
- precipitate sulfides by using iron salts
- increase COD/SO_4^{2-} ratio, to favour the release of H_2S from the liquid phase to the gaseous phase

(d) Toxicity by metals

Toxic elements and compounds such as chromium, chromates, nickel, zinc, copper, arsenic and cyanides, among others, are classified as highly toxic inorganic toxins. In particular, the presence of low concentrations of copper, zinc and nickel in soluble state is considered highly toxic, and these salts are associated with most of the toxicity problems caused by metals in anaerobic treatment.

The concentrations of the most toxic metals that can be tolerated in anaerobic treatment are related to the concentrations of sulfide available to be combined with the metals and then form insoluble sulfide salts. Sulfides by themselves are very toxic to anaerobic treatment but, when combined with metals, they form insoluble salts that have no adverse effect.

One of the most effective procedures to control toxicity by metals is the addition of sufficient amounts of sulfide to precipitate the metals. Approximately 1.8 to 2.0 mg/L of metals is precipitated as metallic sulfides by the addition of 1.0 mg/L of sulfide (S^{2-}). This phenomenon is a good alternative for the treatment of industrial effluents containing metals. If this ratio (1 mg/L of sulfide:2 mg/L of metals) is not verified during the treatment, the addition of sodium sulfide or of a sulfate salt is recommended.

25

Biomass in anaerobic systems

25.1 PRELIMINARIES

A biological treatment process tends to be economical if it can be operated at low hydraulic detention times and at sufficiently long solids retention times to allow microorganism growth. This was for many years the greatest problem of anaerobic digestion, as the solids retention time could not be controlled independently of the hydraulic detention time. Thus, the microorganisms involved in the process, which have low growth rates, needed extremely long retention times and consequently reactors of large volumes. The development of high-rate anaerobic processes solved this problem, since these processes are capable of allowing the presence of a large amount of high-activity biomass, which can be maintained in the reactor even when operated at low hydraulic detention times. If sufficient contact can be guaranteed between the biomass and the organic compounds, high volumetric loads can then be applied to the system.

25.2 BIOMASS RETENTION IN ANAEROBIC SYSTEMS

25.2.1 Preliminaries

Microbial cells exist in a wide range of sizes, forms and growth phases, individually or aggregated in several microstructures. These conditions have a practical meaning in anaerobic digestion, as it is probable that the biomass form has a significant effect on the survival of the organisms and on the transfer of nutrients and, consequently, on the global efficiency of the anaerobic digestion process.

Biological Wastewater Treatment in Warm Climate Regions by Marcos von Sperling and Carlos Augusto de Lemos Chernicharo. ISBN: 1 84339 002 7. Published by IWA Publishing, London, UK.

The formation of a certain structure of aggregated cells depends on several factors, including the size range of the cells inside the microbial population and the location of each individual cell in relation to the others and to the growth medium, for example in the gas/liquid interface. The retention of high-activity biomass in high-rate anaerobic processes depends on a series of factors and mechanisms, as discussed in the following items (adapted from Stronach *et al.*, 1986).

25.2.2 Retention by attachment

The habitats of microorganisms in aqueous systems, such as anaerobic digesters, are very diverse, and their survival and growth depend on factors such as temperature, nutrient availability and stratification. The organisms often overcome the instability of the environment where they live by attachment to a surface. The attachment capability of bacteria is impressive. Their superficial structures seem to allow some form of control of the adhesion, while their microscopic dimensions guarantee that they are hardly subjected to the shearing forces that happen naturally in the medium.

This form of immobilisation of microorganisms, through attachment, is possible on fixed surfaces, such as in anaerobic processes with a stationary bed (e.g. anaerobic filter), or on moving surfaces, such as in anaerobic processes of expanded and fluidised beds. Figure 25.1 illustrates the biofilm formation attached to a support medium.

25.2.3 Retention by flocculation

Flocculation has a practical meaning in sewage treatment, since the flocculating microstructures can be easily separated from the liquid phase by sedimentation. The phenomenon of flocculation is particularly important in two-stage processes and in upflow anaerobic sludge blanket (UASB) reactors. Bacterial growth in flocs is not necessary for an efficient substrate removal, but it is essential to guarantee an effluent with low concentrations of suspended solids.

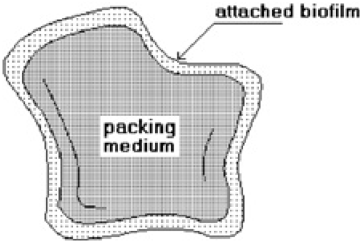


Figure 25.1. Biomass retention by attachment

25.2.4 Retention by granulation

In terms of wastewater treatment, the phenomenon of granulation (formation of granules) seems to be restricted to UASB reactors (and its variants) and, to a lesser extent, to anaerobic filters. This is usually associated with the treatment of wastewaters rich in carbohydrates and volatile acids.

The mechanisms that control the selection and formation of granules are related to physical, chemical and biological factors, including (Lettinga *et al.*, 1980; Hulshoff Pol *et al.*, 1984; Wiegant and Lettinga, 1985):

- the characteristics of the substrate (concentration and composition)
- the gravitational compression of the sludge particles and the superficial rate of biogas liberation
- the ideal conditions for the growth of the methanogenic archaea, such as the presence of bivalent cations
- the upflow velocity of the liquid through the sludge bed

Particularly important is the upflow velocity of the liquid, which provides a constant selective pressure on the microorganisms that start adhering to each other and thereby leads to the formation of granules that present good settleability. The granules usually have a well-defined appearance and they can be several millimetres in diameter and accumulate in large amounts in the reactor. The granular configuration presents several advantages from an engineering point of view (Guiot *et al.*, 1992):

- the microorganisms are usually densely grouped
- the non-use of inert support mediums enables the maximum use of the reaction volume of the reactor
- the spherical form of the granules provides a maximum microorganism/volume ratio
- the granules present excellent settleability

In the arrangement of biomass in granules, the different bacterial populations seem to selectively group in layers on top of each other, for example like the model proposed by Guiot *et al.* (1992) for the substrate and product diffusion (Figure 25.2).

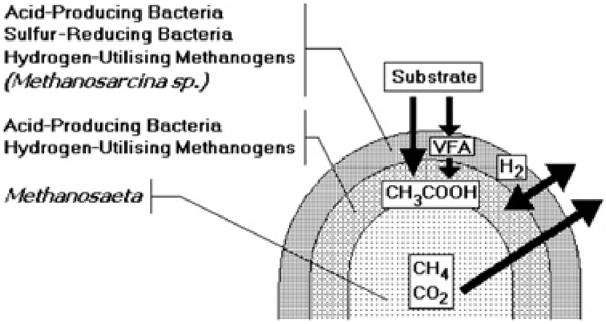


Figure 25.2. Microorganism structure in a granule (after Guiot *et al.*, 1992)

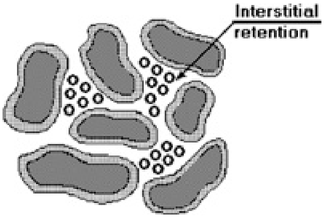


Figure 25.3. Interstitial biomass retention

25.2.5 Interstitial retention

This type of biomass immobilisation occurs in the interstices (Figure 25.3) of stationary support mediums, as is the case of fixed bed anaerobic reactors. The surfaces of the medium serve as support for the attached bacterial growth (formation of the biofilm), while the empty spaces in the packing material are occupied by microorganisms that grow dispersely.

25.3 EVALUATION OF THE MICROBIAL MASS

The determination of the biomass in anaerobic digesters presents two main difficulties: (i) in some systems, the microorganisms are attached to small inert particles; and (ii) the biomass is usually present as a consortium of different morphologic and physiologic types.

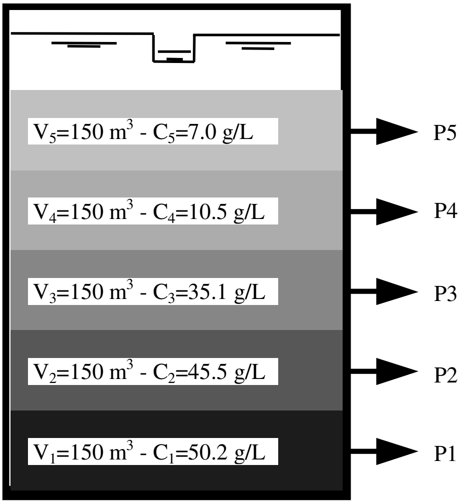
The determination of the biomass and the microbial composition usually requires the extraction, isolation and separation of the biochemical constituents that are specific to a certain group of microorganisms. The cellular components that change quickly in nature, after the death of a cell, can be used, for example, for the estimation of the viable biomass.

Although there are several methodologies to evaluate the amount and activity of the biomass in anaerobic digesters, most of them are sophisticated and cannot be adopted as control and monitoring parameters for reactors operating in full scale, especially if considering the existing laboratory resources in many developing countries. Hence, the evaluation of the amount of biomass is usually made through the determination of the vertical solids profile, considering that the volatile solids are a measure of the biomass present in the reactor (mass of cellular material). Sludge samples collected at different levels of the reactor height are gravimetrically analysed and the results are expressed in terms of grams of volatile solids per litre (gVS/L). These concentration values of volatile solids (made for each of the sludge sampling points along the reactor height), multiplied by the volumes corresponding to each sampled zone, provide the mass of microorganisms along the reactor profile. The sum of the biomass quantities in each zone is equal to the total mass of solids in the reactor, as shown in Example 25.1.

Example 25.1

Determine the amount and the average concentration of the biomass in an anaerobic reactor. Data are:

- total reactor volume: $V = 1{,}003.5\ m^3$
- volume of the digestion compartment: $V_{dc} = 752.6\ m^3$
- volume of the sedimentation compartment: $V_{sc} = 250.9\ m^3$
- volumes corresponding to each sampled zone, as indicated in the illustration below (V_1 to V_5)
- sludge concentration in each sampled zone, as indicated in the illustration below (C_1 to C_5)



Solution:

- Calculation of the amount of biomass (M) in each zone of the reactor:

$$\begin{aligned}
&\text{Zone 1: } M_1 = C_1 \times V_1 = 50.2\ \text{kgVS/m}^3 \times 150\ \text{m}^3 = 7{,}530\ \text{kgVS}\\
&\text{Zone 2: } M_2 = C_2 \times V_2 = 45.5\ \text{kgVS/m}^3 \times 150\ \text{m}^3 = 6{,}750\ \text{kgVS}\\
&\text{Zone 3: } M_3 = C_3 \times V_3 = 35.1\ \text{kgVS/m}^3 \times 150\ \text{m}^3 = 5{,}265\ \text{kgVS}\\
&\text{Zone 4: } M_4 = C_4 \times V_4 = 10.5\ \text{kgVS/m}^3 \times 150\ \text{m}^3 = 1{,}575\ \text{kgVS}\\
&\text{Zone 5: } M_5 = C_5 \times V_5 = 7.0\ \text{kgVS/m}^3 \times 150\ \text{m}^3 = 1{,}050\ \text{kgVS}
\end{aligned}$$

- Calculation of the amount of biomass in the digestion compartment (M_{dc}):

$$M_{dc} = M_1 + M_2 + M_3 + M_4 + M_5 = 22{,}170\ \text{kgVS}$$

***Example 25.1* (*Continued*)**

- Calculation of the average biomass concentration in the digestion compartment (C_{dc})

$$C_{dc} = M_{dc}/V_{dc} = 22{,}170\ \text{kgVS}/750\ \text{m}^3 = 29.6\ \text{kgVS/m}^3$$
$$= 29.6\ \text{gVS/L} = 29{,}600\ \text{mgVS/L} \approx 3.0\%$$

- Calculation of the average biomass concentration in the reactor (C_r):

Assuming that the amount of biomass in the settling compartment is negligible when compared to the digestion compartment, it can be stated that $M_r = M_{dc}$

$$C_r = M_r/V = 22{,}170\ \text{kgVS}/1{,}003.5\ \text{m}^3 = 22.1\ \text{kgVS/m}^3$$
$$= 22.1\ \text{gVS/L} = 22{,}100\ \text{mgVS/L} \approx 2.2\%$$

25.4 EVALUATION OF THE MICROBIAL ACTIVITY

25.4.1 Preliminaries

In the last few years, with the development of high-rate anaerobic processes and the increased knowledge of the microbiology and biochemistry of the process, a growing use of anaerobic digestion has been observed for the treatment of a diverse number of liquid effluents. However, the success of any anaerobic process, especially the high-rate ones, depends fundamentally on the maintenance (inside the reactors) of an adapted biomass with a high microbiological activity that is resistant to shock loads. The development of techniques for the evaluation of the microbial activity in anaerobic reactors is very important, especially of the methanogenic archaea, so that the biomass can be preserved and monitored.

In this respect, several methods have been proposed to evaluate the anaerobic microbial activity, considering the assessment of the **specific methanogenic activity** (**SMA**). However, the precision of several methodologies was considered doubtful or too sophisticated for reproduction in laboratories. Another problem identified refers to the difficulty, or even impossibility, in obtaining anaerobic sludge in sufficient amounts, from reactors in laboratory scale, for the development of conventional tests.

A preliminary analysis of the studies already developed in the area indicates that some methods used for the evaluation of the SMA are crude or imprecise, whilst others are too expensive or sophisticated. The simplified method developed by James *et al.* (1990), from an adaptation of the operation of the Warburg respirometer, was undoubtedly a valuable contribution, but as the authors themselves stated, greater success was dependent on the automation of the gas measurement system and on the optimisation of the monitoring system of the test as a whole.

In this regard, the work developed by Monteggia (1991), incorporating manometers with electric sensors for the continuous monitoring of the biogas production, constituted an important improvement on the SMA test.

Recently, some innovations have been presented in relation to the gas measurement system, which replaced the conventional manometers with pressure transducers. The incorporation of these devices facilitated significantly the detection of the pressure differential inside the reaction and control flasks, besides allowing the transmission of electric pulses to a computer terminal.

25.4.2 Importance of the SMA test

The evaluation of the specific methanogenic activity of anaerobic sludge has proved important in the effort to classify the biomass potential in the conversion of soluble substrate into methane and carbon dioxide. The microbial activity test can be used as a routine analysis to quantify the methanogenic activity of anaerobic sludge or, also, in a series of other applications, as listed below:

- to evaluate the behaviour of biomass under the effect of potentially inhibiting compounds
- to determine the relative toxicity of chemical compounds present in liquid effluents and solid residues
- to establish the degree of degradability of several substrates, especially of industrial wastewater
- to monitor the changes of activity of the sludge, because of a possible accumulation of inert materials after long periods of reactor operation
- to determine the maximum organic load that can be applied to a certain sludge type, providing an acceleration of the start-up stage of treatment systems
- to evaluate kinetic parameters

25.4.3 Brief description of the SMA test

In practice, the SMA test consists in the evaluation of the capacity of the methanogenic archaea to convert organic substrate into methane and carbon dioxide gas. Thus, from known amounts of biomass (gVS) and substrate (gCOD), and under established conditions, the production of methane can be evaluated during the test period. The SMA is calculated based on the maximum methane productivity rates ($mLCH_4/gVS{\cdot}h$ or $gCOD\text{-}CH_4/gVS{\cdot}d$). The conversion of $mLCH_4$ into $gCOD\text{-}CH_4$ is done according to Equations 24.15 and 24.16 (Chapter 24). For the development of the test, the following are necessary:

- anaerobic sludge, for which the SMA is to be evaluated
- organic substrate (usually sodium acetate is used)
- buffer and nutrient solution (see Table 25.1)
- reaction flasks

Table 25.1. Buffer and nutrient solution

Solution	Reagent	Concentration	Purpose
1	KH_2PO_4	1,500 mg/L	Buffer
	K_2HPO_4	1,500 mg/L	
	NH_4Cl	500 mg/L	Macronutrient
	$Na_2S{\cdot}7H_2O$	50 mg/L	
2	$FeCl_3{\cdot}6H_2O$	2,000 mg/L	Micronutrient
	$ZnCl_2$	50 mg/L	
	$CuCl_2{\cdot}4H_2O$	30 mg/L	
	$MnCl_2{\cdot}2H_2O$	500 mg/L	
	$(NH_4)_6{\cdot}Mo_7O_{24}4H_2O$	50 mg/L	
	$AlCl_3$	50 mg/L	
	$CoCl_3{\cdot}6H_2O$	2,000 mg/L	
	HCl (concentrated)	1 mL	

Note: At the time solutions are used, add 1 mL of solution 2 per litre of solution 1 to obtain a single solution that shall be added to the reaction flask.
Source: Monteggia (1991)

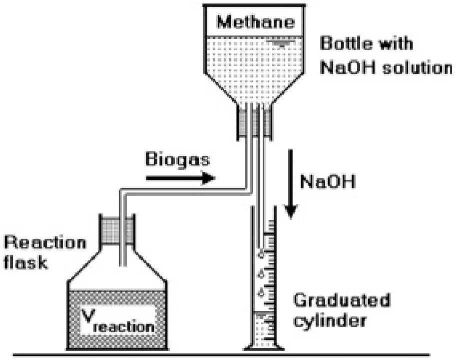


Figure 25.4. Apparatus for biogas measurement (adapted from van Haandel and Lettinga, 1984)

- temperature controlling device (water bath, incubator, heat apparatus, acclimatised room, etc.)
- mixing device for the sludge sample
- device for measuring gas production over a certain period of time. The measurement of the production of gases can be evaluated in different ways, each with its advantages and disadvantages:
  - through water displacement (see Figure 25.4)
  - through mini-manometers (visual reading or with an electric sensor)
  - through pressure transducers etc.

Although there are different methods to follow in the development of SMA tests, the following protocol for the test was recently adopted by PROSAB (Brazilian

Research Programme on Basic Sanitation):

- determine the concentration of volatile solids present in the sludge to be analysed (gVS/L)
- place the pre-established amounts of sludge into the reaction flasks, preferably 12 to 24 hours before the beginning of the test, seeking to adapt them to the test conditions. Reaction flasks of 250 to 500 mL have usually been used at a temperature of 30 °C for the development of the test
- add to the reaction flasks certain amounts of the buffer and nutrient solution, to obtain final concentrations of the mixture (sludge+solution+substrate) of around 2.5 gVS/L. The final volume of the mixture should occupy between 70 and 90% of the volume of the reaction flask
- before adding the substrate, the oxygen present in the head space of the flask should be removed using gaseous nitrogen (pressure of 5 psi, for 5 minutes)
- add the substrate to the reaction flasks, in the concentrations desired (usually with concentrations varying from 1.0 to 2.5 gCOD/L)
- turn on the mixing device in the reaction flasks
- record the volumes of biogas produced at each time interval, during the test period (mL/hour). The determination of the methane concentration in the biogas can be made by chromatography or, alternatively, by the absorption of the carbon dioxide gas present in the biogas, through its passage in an alkaline solution (e.g. NaOH 5%)

Example 25.2

Determine the main parameters necessary for the development of a SMA test of an anaerobic sludge, considering:

- number of reaction flasks: 4
- test temperature: T = 30 °C
- volume of each reaction flask: 250 mL
- total volume of the mixture (sludge+solution+substrate): 200 mL (20% head space)
- concentration of the anaerobic sludge to be tested: 3% (30 gVS/L)
- sludge concentration in the mixture (sludge+solution+substrate): 2.5 gVS/L
- COD concentrations tested (gCOD/L): 1.0 (flask 1), 1.5 (flask 2), 2.0 (flask 3) and 2.5 (flask 4)

Solution:

- Determination of the sludge volume to be added to each flask, to obtain the final concentration in the mixture (sludge+solution+substrate)

Example 25.2 (Continued)

equal to 2.5 gVS/L:

$$V_{sludge} = (V_{mixture} \times C_{mixture})/C_{sludge} = (200\,mL \times 2.5\,gVS/L)/30\,gVS/L$$
$$= 16.7\,mL$$

- Determination of the mass of microorganisms in each flask:

$$M_{sludge} = V_{sludge} \times C_{sludge} = 16.7\,mL \times 0.030\,gVS/mL = 0.501\,gVS$$

- Determination of the substrate volume to be added to each flask, to obtain the final concentrations of 1.0, 1.5, 2.0 and 2.5 gCOD/L

  Considering the application of the sodium acetate solution with a concentration of 100 gCOD/L:
  - flask 1 (1.0 gCOD/L): $V_{substrate} = (C_{mixture} \times V_{mixture}) / C_{solution} =$ (1.0 mgCOD/mL × 200 mL)/100 mgCOD/mL = 2 mL
  - flask 2 (1.5 gCOD/L): $V_{substrate} =$ (1.5 mgCOD/mL × 200 mL)/ 100 mgCOD/mL = 3 mL
  - flask 3 (2.0 gCOD/L): $V_{substrate} =$ (2.0 mgCOD/mL × 200 mL)/ 100 mgCOD/mL = 4 mL
  - flask 4 (2.5 gCOD/L): $V_{substrate} =$ (2.5 mgCOD/mL × 200 mL)/ 100 mgCOD/mL = 5 mL

- Determination of the volume of buffer and nutrient solution:

Knowing that the total volume of the mixture was established at 200 ml, the volume of buffer and nutrient solution can be obtained by subtracting the sludge and substrate volumes already calculated from the total volume (see the following table).

Flask	Sludge concentration (gVS/L)	Volume (mL)				Quantity of biomass (gVS)	Final concentration	
		Sludge	Substrate	Solution	Mixture		Sludge (gVS/L)	Substrate (gCOD/L)
1	30	16.7	2	181.3	200	0.501	2.5	1.0
2	30	16.7	3	180.3	200	0.501	2.5	1.5
3	30	16.7	4	179.3	200	0.501	2.5	2.0
4	30	16.7	5	178.3	200	0.501	2.5	2.5

Once the preparatory parameters for the test have been defined, as shown in the above table, one should proceed according to the test protocol described in Section 25.4.3. The continuous monitoring of the methane production in the reaction flasks makes it possible to obtain data that correlate time with cumulative CH_4 production. The graphic representation of these data allows obtaining curves similar to those presented in Figure 25.5, one for each of the reaction flasks (1 to 4).

The determination of the specific methanogenic activity is done based on the evaluation of the slope of the line of best fit of the methane production curve (steepest reach). The slope gives the methane production rate (e.g. $mLCH_4$/hour) which, divided by the initial amount of biomass present

Example 25.2 (*Continued*)

in the reaction flask (in the example, $M_{sludge} = 0.501$ gVS), gives the specific methanogenic activity of the sludge (mLCH$_4$/gVS.hour). The correspondence of the volume of methane in mass of COD converted into CH$_4$ (COD-CH$_4$) is usually done, as detailed in Chapter 24 (Equations 24.15 and 24.16), so as to enable the SMA to be expressed in terms of gCOD-CH$_4$/gVS·d.

Figure 25.6 shows the methanogenic activity curves for each of the flasks, obtained by calculating the activity for each time interval and not just for the parts where the methane production rate is maximum.

According to Figure 25.6, the maximum activities were approximately 0.50, 0.55, 0.75 and 0.68 gCOD-CH$_4$/gVS·d, for flasks 1, 2, 3 and 4, respectively. In this example, the anaerobic sludge showed its largest activity for a substrate concentration equal to 2.0 gCOD/L (flask 3). This is the specific methanogenic activity of the sludge that should be considered. The most accurate calculation of the activities should be done with the reaches of maximum slope (Figure 25.5), as explained previously.

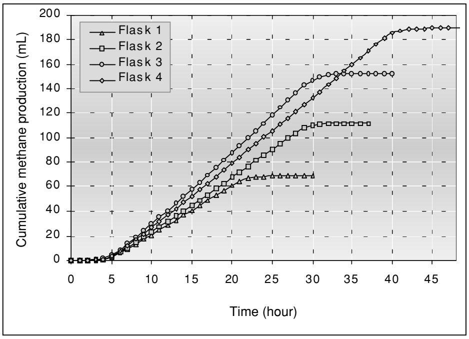


Figure 25.5. SMA test. Cumulative CH$_4$ production results

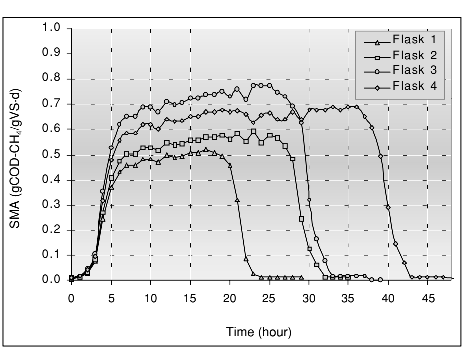


Figure 25.6. SMA test. Methanogenic activity results

***Example 25.2* (*Continued*)**

- Determination of the amount of substrate converted into methane: According to the curves of Figure 25.5, the total CH_4 production, at the end of the test for each of the flasks, was:
  - flask 1: $V_{CH_4} \cong 70$ mL
  - flask 2: $V_{CH_4} \cong 112$ mL
  - flask 3: $V_{CH_4} \cong 152$ mL
  - flask 4: $V_{CH_4} \cong 190$ mL
- Determination of the theoretical methane production, from the amount of substrate (gCOD) added to each flask:

  According to Equations 24.15 and 24.16 (Chapter 24):

$$K(t) = (P \cdot K)/[R \cdot (273 + T)] = (1 \times 64)/[0.08206 \times (273 + 30)]$$

$$= 2.57 \text{ gCOD/L}$$

$$V_{CH_4} = \text{COD–CH}_4/K(t) =$$

  - flask 1: 2 mL × 100 mgCOD/mL = 200 mgCOD ⇒ VCH_4 = 200 mgCOD/2.57 mgCOD/mL = 77.8 mL
  - flask 2: 3 mL×100 mgCOD/mL = 300 mgCOD ⇒ VCH_4 = 300 mgCOD/2.57 mgCOD/mL = 116.7 mL
  - flask 3: 4 mL×100 mgCOD/mL = 400 mgCOD ⇒ VCH_4 = 400 mgCOD/2.57 mgCOD/mL = 155.6 mL
  - flask 4: 5 mL×100 mgCOD/mL = 500 mgCOD ⇒ VCH_4 = 500 mgCOD/2.57 mgCOD/mL = 194.6 mL
- Determination of the percentage substrate converted into methane:
  - flask 1: 70 mL/77.8 mL = 90%
  - flask 2: 112 mL/116.7 mL = 96%
  - flask 3: 152 mL/155.6 mL = 98%
  - flask 4: 190 mL/194.6 mL = 98%

25.4.4 Final considerations about the SMA test

Although the SMA test constitutes a very useful tool, the results should still be used with caution, as there is no accepted international standard as yet. The efforts of the *IWA Task Group on anaerobic biodegradability and activity tests* in establishing such standard should be acknowledged. So far, the different methodologies and experimental conditions can lead to different SMA results, which are difficult to be compared amongst themselves. In this respect, it is understood that the results obtained with the test reflect much more the relative specific methanogenic activities, and not the absolute ones. However, even if the results are relative for certain test conditions, they are very important for the follow-up and evaluation of anaerobic reactors.

26

Anaerobic treatment systems

26.1 PRELIMINARIES

The essence of biological wastewater treatment processes resides in the capacity of the microorganisms involved to use the biodegradable organic compounds and transform them into by-products that can be removed from the treatment system. The by-products formed can be in solid (biological sludge), liquid (water) or gaseous (carbon dioxide, methane, etc.) form. In any process used, aerobic or anaerobic, the capacity for using the organic compounds will depend on the microbial activity of the biomass present in the system.

Until recently, the use of anaerobic processes for the treatment of liquid effluents was considered uneconomical and problematic. The reduced growth rate of the anaerobic biomass, especially the methanogenic Archaea, makes the control of the process delicate, since the recovery of the system is very slow when the anaerobic biomass is exposed to adverse environmental conditions.

With the expansion of research in the area of anaerobic treatment, "high-rate systems" have been developed. Essentially, these are characterised by their ability to retain large amounts of high-activity biomass, even with the application of low hydraulic detention times. Thus, a high solids retention time is maintained, even with the application of high hydraulic loads to the system. The result is compact reactors with volumes inferior to conventional anaerobic digesters, however maintaining the high degree of sludge stabilisation. The concepts of hydraulic detention time and solids retention time are covered in Section 9.5.3. In this chapter, the main anaerobic

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systems used for wastewater treatment are described. For convenience, they are classified into two large groups, as shown below:

1. Conventional systems
    - Sludge digesters
    - Septic tanks
    - Anaerobic ponds
2. High-rate systems
    - With attached growth
        - Fixed bed reactors
        - Rotating bed reactors
        - Expanded/fluidised bed reactors
    - With dispersed growth
        - Two-stage reactors
        - Baffled reactors
        - Upflow sludge blanket reactors
        - Expanded granular bed reactors
        - Reactors with internal recirculation

26.2 CONVENTIONAL SYSTEMS

26.2.1 Preliminaries

In this chapter, the designation *conventional systems* is used to classify reactors that are operated with low volumetric organic loads, as they do not have retention mechanisms for large quantities of high-activity biomass. Obviously, a well-defined separation line does not exist between the conventional and the high-rate systems. The examples presented here are only for the purpose of classifying some types of reactors, based on the main aspects that differentiate them from high-rate reactors, which are:

- Absence of solids retention mechanisms in the system: as discussed in Chapter 25, biomass retention in anaerobic systems is improved in a significant way through mechanisms that favour the immobilisation of the microorganisms inside the digestion compartment, as attachment and granulation. The absence of such mechanisms hinders the retention of great amounts of biomass in the treatment system.
- Long hydraulic detention times and low volumetric loads: the absence of solids retention mechanisms in the system implies the need for the conventional reactors to be designed and operated with long hydraulic detention times, to guarantee that the biomass will stay in the system long enough for its growth (see Section 9.5).
- Low volumetric loads: the design of reactors with long hydraulic detention times implies having tanks with large volumes and, as a result, low volumetric loads applied to the system (kgCOD/m^3reactor·d or kgVS/m^3reactor·d).

From the following discussion, it will become clear that some aspects that are used to classify conventional systems can be found in a more or less pronounced way in a certain reactor type. It can be inferred that conventional systems are evolving towards high-rate systems.

26.2.2 Anaerobic sludge digesters

Conventional digesters are mainly used for the stabilisation of primary and secondary *sludge*, originating from sewage treatment, and for the treatment of *industrial effluents* with a high concentration of suspended solids. They usually consist of covered circular or egg-shaped tanks of reinforced concrete. The bottom walls are usually inclined, so as to favour the sedimentation and removal of the most concentrated solids. The covering of the reactor can be fixed or floating (mobile).

Since conventional digesters are preferably used for the stabilisation of wastes with a high concentration of particulate material, the hydrolysis of these solids can become the limiting stage of the anaerobic digestion process. The hydrolysis rate, in turn, is affected by several factors, such as: (i) temperature; (ii) residence time; (iii) substrate composition and (iv) particle size.

Thus, with the aim to optimise the hydrolysis of the particulate material, conventional digesters may be heated up, with operation temperatures usually ranging from 25 to 35 °C. The hydrolysis phase evolves very slowly when the digesters are operated at temperatures below 20 °C.

As the conventional digesters do not have specific means for biomass retention in the system, the hydraulic detention time should be long enough to guarantee the permanence and multiplication of the microorganisms in the system, while enabling all the phases of the anaerobic digestion to be processed appropriately.

Depending on the existence of mixing devices and on the number of stages, three main digester configurations have been applied:

- low-rate anaerobic sludge digester
- one-stage high-rate anaerobic sludge digester
- two-stage high-rate anaerobic sludge digester

Anaerobic sludge digesters are covered in detail in Chapter 49.

(a) Low-rate anaerobic sludge digester

The low-rate digester does not have mixing devices and usually comprises a single tank, where the digestion, sludge thickening and supernatant formation occur simultaneously. Raw sludge is added to the part of the digester where the sludge is undergoing active digestion and the biogas is being released. With the upflow movement of the biogas, particles of sludge and other flotation materials are taken to the surface, forming a scum layer. As a result of the digestion, the sludge stratifies below the scum layer, and four different zones are formed inside the reactor, as characterised (see Figure 26.1): scum zone, supernatant zone, active digestion zone and stabilised sludge zone.

The supernatant and stabilised sludge are periodically removed from the digester. Because of the sludge stratification and the absence of mixing, no more than 50% of the digester volume are actually used in the digestion process, with large reactor volumes being required to achieve good sludge stabilisation. In view of these limitations, low-rate digesters are mainly used in small treatment plants.

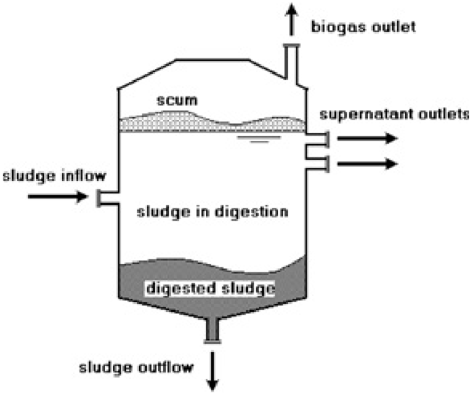


Figure 26.1. Schematic representation of a low-rate anaerobic sludge digester

(b) One-stage high-rate anaerobic sludge digester

The one-stage high-rate digester incorporates supplemental heating and mixing mechanisms, besides being operated at uniform feeding rates and with the previous thickening of the raw sludge, to guarantee more uniform conditions in the whole digester. As a result, the tank volume can be reduced and the stability of the process is improved. Figure 26.2 presents a schematic representation of a one-stage high-rate digester.

The solids retention times recommended for the design of complete-mix digesters are illustrated in Figure 26.3, and the high dependence of these in relation to the operational temperature of the digester can be observed. When sizing the reactor, the hydraulic detention time shall be equal to the solids retention time, as the system does not have a solids retention mechanism (see Sections 9.5 and 25.2).

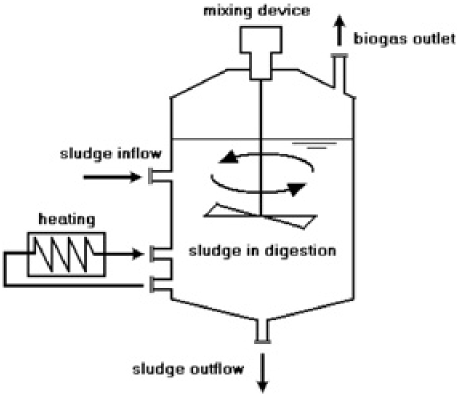


Figure 26.2. Schematic representation of a one-stage high-rate anaerobic sludge digester

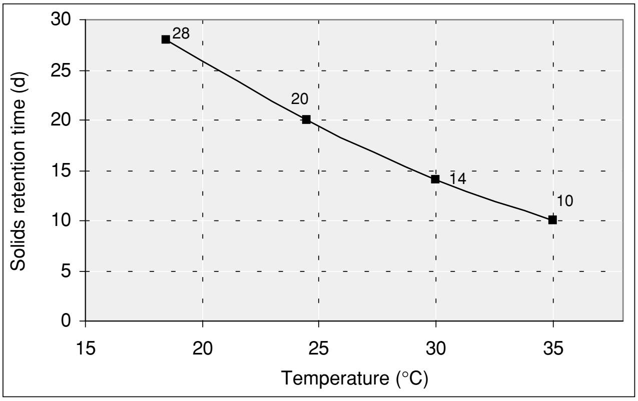


Figure 26.3. Design recommendations for completely mixed anaerobic digesters (adapted from Metcalf and Eddy, 1991)

Different techniques such as gas recirculation, sludge recirculation or mechanical mixers of various configurations can be used to obtain the mixture of the sludge inside the digester.

(c) Two-stage high-rate anaerobic sludge digester

Basically, the two-stage digester consists in the incorporation of a second tank, operating in series with a high-rate primary digester, as illustrated in Figure 26.4. In this configuration, the first tank is used for the digestion of the sludge, and may therefore be equipped with heating and mixing devices. The second tank is used

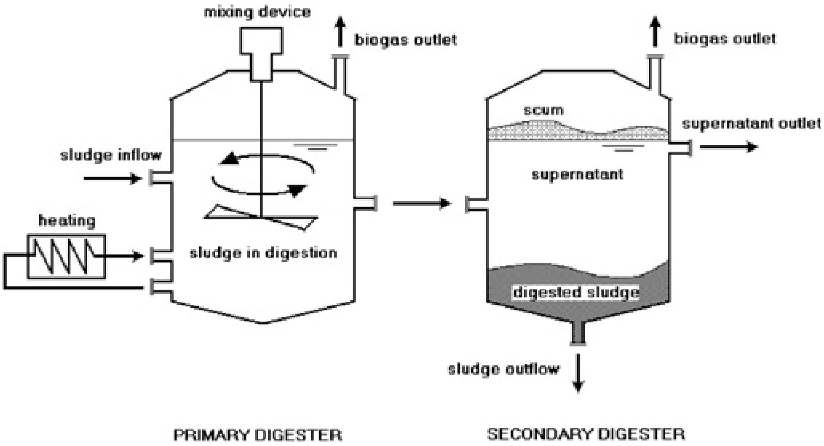


Figure 26.4. Schematic representation of a two-stage high-rate anaerobic sludge digester

for the storage and thickening of the digested sludge, leading to the formation of a clarified supernatant.

There are situations in which the two tanks are designed in an identical way, so that either can be used as the primary digester. In other situations, the secondary digester can be an open tank, a tank without heating, or even a sludge pond (Metcalf and Eddy, 1991).

26.2.3 Septic tank

The septic tank is a unit that carries out the multiple functions of sedimentation and removal of floatable materials, besides acting as a low-rate digester without mixing and heating capabilities. Septic tanks were conceived around 1860, based on the pioneering work of Mouras, in France. They are still extensively used all over the world and constitute one of the main alternatives for the primary treatment of sewage from residences and small areas that are not served by sewerage networks. The operation of septic tanks can be described as follows:

- The settleable solids present in the influent sewage go to the bottom of the tank and form a sludge layer.
- The oils, grease and other lighter materials present in the influent sewage float on the surface of the tank, forming a scum layer.
- The sewage, free from the settled and floated material, flows between the sludge and scum layers and leaves the septic tank at the opposite end, from where it is directed to a post-treatment unit or to final disposal.
- The organic matter kept at the bottom of the tank undergoes facultative and anaerobic decomposition, and is converted into gaseous compounds such as CO_2, CH_4 and H_2S. Although H_2S is produced in septic tanks, odour problems are not usually observed as it combines with metals accumulated in the sludge and forms insoluble metallic sulfides.
- The anaerobic decomposition provides a continuous reduction of the sludge volume deposited at the bottom of the tank. There is always an accumulation during the months of operation of the septic tank and consequently the sludge and scum accumulation reduces the net volume of the tank, which demands periodic removal of these materials.

To optimise the retention of settleable and floatable solids inside the tank, the tank is usually equipped with internal baffles close to the inlet and outlet points. Multiple compartments are also used with the purpose of reducing the amount of solids in the effluent, although single-chamber tanks are more commonly used, as illustrated in Figure 26.5.

Improvement of the septic tank can be achieved by imposing an upward flow and gas/solid/liquid separation at the top, as in the so-called UASB septic tank (van Lier *et al.*, 2002). This system configuration differs from the conventional septic tank by the upflow mode, which allows a better mixing between the influent and the biomass present at the bottom of the tank, resulting in improved biological conversion of dissolved components. In addition, the upward flow and the gas/solid/liquid

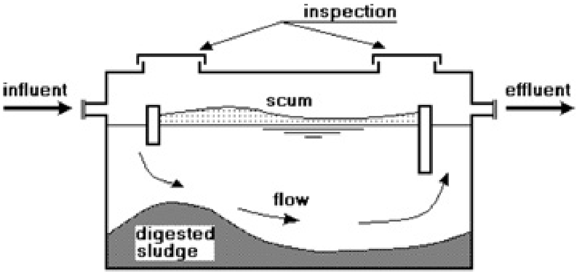


Figure 26.5. Schematic representation of a single-chamber septic tank

separator enhance the physical removal of suspended solids. The UASB septic tank differs from the conventional UASB reactor (see Section 26.3.3) mainly in relation to sludge accumulation. In the case of UASB septic tank, sludge needs to be removed only once in 1 or 2 years, depending on the design of the reactor.

26.2.4 Anaerobic pond

Anaerobic ponds constitute a very appropriate alternative for sewage treatment in warm-climate regions, and they are usually combined with facultative ponds. They are also frequently used for the treatment of wastewaters with a high concentration of organic matter, such as those from slaughterhouses, dairies, breweries, etc. Figure 26.6 illustrates a typical anaerobic pond.

Owing to the large dimensions and the long hydraulic detention times, anaerobic ponds can be classified as low volumetric organic load reactors. In their typical configuration, the operation of the anaerobic ponds is very similar to that of septic tanks and uses the same basic removal mechanisms described in the previous section. However, the dimensions of the anaerobic ponds are superior to those of the septic tanks, which gives them some different characteristics:

- Because of the great volumes and high depths, there is no need for the systematic removal of the sludge deposited at the bottom of the anaerobic ponds, and cleaning is expected to be required at intervals of a few years.
- Because they are open reactors, and also because of the large areas occupied, there is always the possibility of release of bad odours and proliferation of insects, which requires great care to be taken when choosing their location.

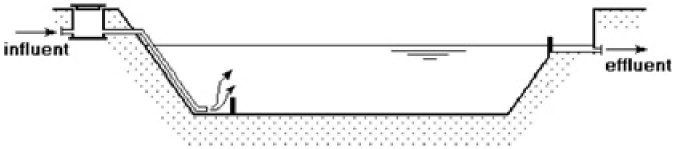


Figure 26.6. Schematic representation of an anaerobic pond

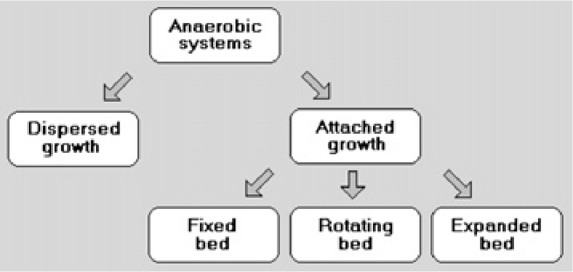


Figure 26.7. Classification of the anaerobic systems

The main design criteria are based on a volumetric organic load (kgBOD/m^3·d). For domestic sewage, this usually leads to detention times in the order of 3 to 6 days.

Even though the minimum cell residence time of the acetoclastic methanogenic archaea is around 3.3 days, for a temperature of 30 °C, there has been a recent tendency of reducing the detention times in the anaerobic ponds to around 1 to 2 days. This can be achieved if the retention time of the biomass can be maintained above 3 days, to guarantee the maintenance of a stable bacterial population and an intimate biomass–sewage contact. These conditions can be accomplished through a better distribution of the influent through the bottom of the pond, at several points, aimed at simulating the feeding of UASB reactors (see Section 26.3.3). In this manner, biomass development mechanisms with good settling and activity characteristics are favoured, increasing the solids retention in the system.

Additional information on anaerobic ponds, including design criteria and example, is presented in Chapter 14.

26.3 HIGH-RATE SYSTEMS

26.3.1 Preliminaries

As discussed in Chapter 25, anaerobic reactors operated with short hydraulic detention times and long solids retention times need to incorporate biomass retention mechanisms, thereby making up the so-called high-rate systems. Several types of high-rate anaerobic reactors are used for the treatment of sewage and these can be classified into two large groups, according to the type of biomass growth in the system, as illustrated in Figure 26.7.

The concept of dispersed bacterial growth is associated with the presence of free bacterial flocs or granules. On the other hand, the concept of attached bacterial growth requires the development of bacteria joined to an inert support material, leading to the formation of a biological film (biofilm).

26.3.2 Systems with attached bacterial growth

The systems with attached bacterial growth can be divided into *fixed bed*, *rotating bed* and *expanded bed* reactors, as described below (adapted from Stronach *et al.*, 1986).

(a) Fixed bed anaerobic reactors

The more commonly known example of reactors with an attached bacterial growth, in a fixed bed, are the **anaerobic filters**. These are characterised by the presence of a stationary packing material, in which the biological solids can attach to or be kept within the interstices. The mass of microorganisms attached to the support material or kept in their interstices degrades the substrate contained in the sewage flow and, although the biomass is released sporadically, the average residence time of solids in the reactor is usually above 20 days.

The first investigations concerning anaerobic filters date from the end of the 1960s and ever since they have had a growing application in the treatment of different types of industrial and domestic effluents. These filters are usually operated with a vertical flow, upward or downward, with the upflow being more commonly used. In the upflow configuration, the liquid is introduced at the bottom, flows through a filter layer (support medium) and is discharged through the upper part (Figure 26.8). In the downflow configuration, sewage is distributed in the upper part of the filter, above the support medium, and is collected in the lower part of the reactor. Downflow reactors can be used with submerged or non-submerged support medium. Effluent recirculation is more commonly practised in this second configuration (Figure 26.9).

There has been an improvement in the optimisation and efficiency of these systems with the increase of microbiological and biochemical knowledge, which

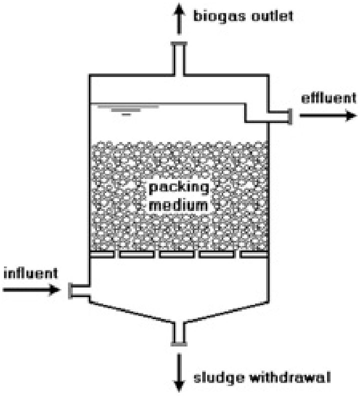


Figure 26.8. Schematic representation of an upflow anaerobic filter

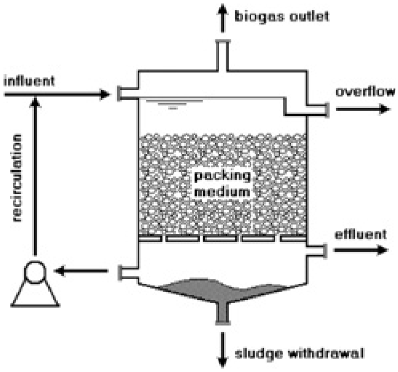


Figure 26.9. Schematic representation of a downflow anaerobic filter

has enhanced their applicability. It can be verified that the average residence time of the microorganisms in the reactors is very high. This is because they are attached to the support medium, which favours a good treatment process performance.

The most important characteristics of a biological treatment are the solids retention time and the concentration of microorganisms present in the medium. The long solids residence times in the reactors, associated with the short hydraulic detention times, provide the anaerobic filter with a great potential for application to the treatment of low-concentration wastewater. A significant portion of the biomass is found as suspended flocs, which are held in the empty spaces of the support medium (interstitial retention), a fact that caused some researchers to state that the shape of the support material is more important than the type of material employed.

The main disadvantage of anaerobic filters is the accumulation of biomass at the bottom of upflow reactors, where it can lead to blockage or the formation of hydraulic short circuits. In this respect, the downflow filters are more suitable for the treatment of wastes that contain higher concentrations of suspended solids. Further details about the design and operation of anaerobic filters are presented in Chapter 27.

(b) Rotating bed anaerobic reactor

The rotating bed reactor, also called aerobic biodisc, was initially documented in 1928, but it was not until the appearance of plastic materials as effective, light and economical support mediums that the process had a wide application to sewage treatment. In this system, the microorganisms attach to the inert support medium and form a biological film. The support medium, with a sequential disc configuration, is partly or totally submerged and rotates slowly around a horizontal axis in a tank through which the sewage flows.

The anaerobic biodisc was developed by Friedman and Tait (1980). The system configuration is similar to that of the aerobic biodisc (Figure 26.10), except that the tank is covered to avoid contact with air. The submergence of the discs is

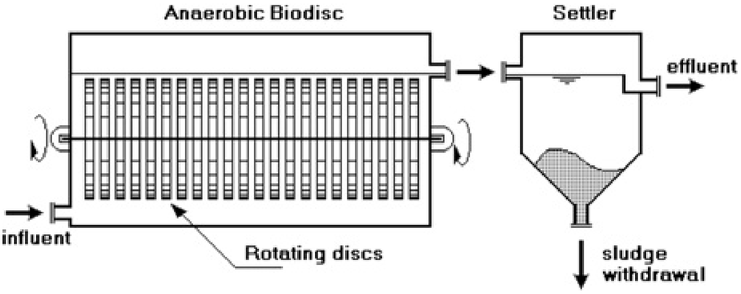


Figure 26.10. Schematic representation of an anaerobic biodisc

also usually larger than that in the aerobic systems, as the transfer of oxygen is not required. The θ_c/t relation (solids retention time/hydraulic detention time) is very high and blocking should not occur in the system, since the rotation speed of the discs is such that the shearing forces promote the removal of the excess biomass kept between the discs. However, care should be taken in the transfer of results obtained in the laboratory to full-scale plants (scale-up), as the rotation speed substantially increases with the increase of the disc diameter. In high rotation speed conditions, the shearing forces can prevent biomass attachment.

(c) Expanded bed anaerobic reactors

The development of the expanded and fluidised bed anaerobic reactors practically eliminated the problems of the limitation of substrate diffusion, usually inherent to the stationary bed processes. In the expanded and fluidised bed processes the biomass grows into reduced thickness films, attached to small sized particles, in contrast to the stationary bed processes, in which the biofilm has considerably larger thickness and is attached to a support medium also of larger dimensions. The expansion and fluidisation of the medium reduces or eliminates blockage problems, besides increasing the biomass retention and its contact with the substrate, thereby allowing significant reductions in the hydraulic detention times in the reactors. Although the distinction between expansion and fluidisation is frequently not clearly defined, two main systems can be characterised.

Expanded bed anaerobic reactor. The process of attached growth and expanded bed was developed by Jewell (1981), as an extension of the existent anaerobic processes. The expanded bed reactors consist of a cylindrical structure, packed with inert support particles to about 10% of its volume. Several types of materials have been used as support mediums, including sand, gravel, coal, PVC, resins, etc. These support particles, with diameters in the order of 0.3 to 3.0 mm, are slightly larger than those used in fluidised bed reactors. The biofilm grows attached to the particles, which are expanded by the upward velocity of the liquid, increased by the high rate of recirculation applied. The expansion of the bed is maintained

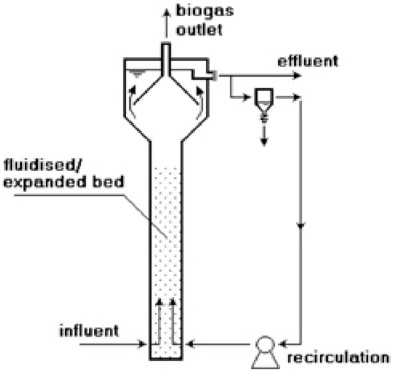


Figure 26.11. Schematic representation of an expanded/fluidised bed reactor

at a level required for each support particle to preserve its relative position to each of the other particles inside the bed. The expansion of the bed is usually maintained between 10 and 20%. The attached growth and expanded bed reactor was considered the first anaerobic process capable of treating diluted sewage at room temperature (Jewell, 1981). In fact, the system has proved to be very efficient in treating very low concentration sewage (in the range of 150 to 600 mgCOD/L), with minimum hydraulic detention times (in the order of 30 to 60 minutes). In these conditions, COD removal efficiencies of about 60 to 70% can be obtained. The formation of a high-activity biomass, with a concentration in the order of 30 gVSS/L, and the retention and filtration of fine inert particles are the reasons for the high-quality effluent in terms of COD and suspended solids.

Fluidised bed anaerobic reactor. The operating principles of the fluidised bed reactor (Figure 26.11) are basically the same as those of the expanded bed reactor, except for the size of the particles of the support medium and the expansion rates. In this case, the upward velocity of the liquid should be sufficiently high to fluidise the bed until it reaches the point at which the gravitational force is equalled by the upward drag force. A high recirculation rate is required and, as a result, each independent particle does not maintain a fixed position inside the bed. The expansion of very fine particles (0.5 to 0.7 mm) guarantees a very large surface area for the growth of a uniform biofilm around each particle. The expansion degree usually varies between 30 and 100%. Volumetric loads as high as 20 to 30 kgCOD/m^3·d have been reported using soluble wastes of medium and high concentrations, with COD removal efficiencies between 70 and 90%.

26.3.3 Systems with dispersed bacterial growth

The efficiency of the systems with dispersed bacterial growth depends largely on the capacity of the biomass to form flocs and settle. Included among the processes with dispersed bacterial growth are the two-stage reactors, baffled reactors and the

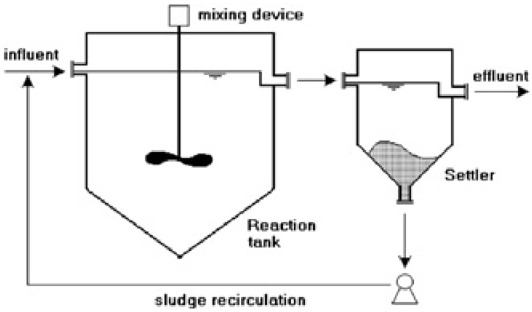


Figure 26.12. Schematic representation of a two-stage reactor

upflow sludge blanket reactors and their variants (expanded granular sludge bed and anaerobic reactor with internal recirculation).

(a) Two-stage anaerobic reactor

The two-stage anaerobic reactor (anaerobic contact process) (Figure 26.12) was developed in the 1950s for the treatment of concentrated industrial wastewater. The system involves the use of a complete-mix tank (anaerobic reactor) followed by a device for the separation and the return of solids. Conceptually, the two-stage reactor is similar to the aerobic activated sludge system. The essence of the two-stage process is that the biomass that is flocculated in the reactor, along with the undigested influent solids that are taken out of the system, is retained through a solids separation device and returned to the first stage reactor where it is mixed with the influent wastewater. The practical difficulty of the two-stage process is the separation and concentration of the effluent solids, as the presence of gas-producing particles leads the biomass flocs to float instead of settling. Several methods have been used or recommended to eliminate these problems, through sedimentation, chemical flocculation, vacuum degasification, flotation and centrifugation, thermal shock, filter membrane, etc.

(b) Baffled anaerobic reactor

The baffled reactor (Figure 26.13) resembles a septic tank with multiple chambers in series and with a more effective feeding device to the chambers. To obtain this configuration, the reactor is equipped with vertical baffles that force the liquid to make a sequential downflow and upflow movement, to guarantee a larger contact of the wastewater with the biomass present at the bottom of the unit. According to Campos (1994), this reactor presents several of the main advantages of the UASB reactors and could be built without the gas separator, therefore with smaller depths, which facilitates its burying, thus representing a reduction in construction costs. However, the project characteristics are not always adequate to guarantee good operational conditions in larger size units. For instance, an excessive loss of solids, in the case of great variations and excessive peaks of the influent flow, may

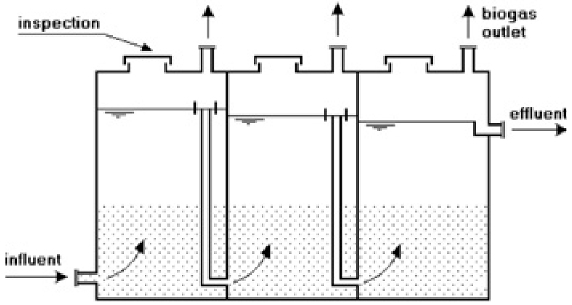


Figure 26.13. Schematic representation of a baffled reactor

occur in this type of reactor, as the system does not have auxiliary mechanisms for biomass retention.

(c) Upflow anaerobic sludge blanket reactor

The upflow anaerobic sludge blanket (UASB) reactor was developed by Lettinga and co-workers, being initially largely applied in Holland. The process essentially consists of an upflow of wastewater through a dense sludge bed with high microbial activity. The solids profile in the reactor varies from very dense and granular particles with good settleability close to the bottom (*sludge bed*) to a more dispersed and light sludge close to the top of the reactor (*sludge blanket*).

Conversion of organic matter takes place in all reaction areas (bed and sludge blanket), and the mixing of the system is promoted by the upward flow of wastewater and gas bubbles. The wastewater enters at the bottom and the effluent leaves the reactor through an internal settling tank in the upper part of the reactor. A gas and solids separation device located below the settling tank guarantees optimal conditions for sedimentation of the particles that stray from the sludge blanket, allowing them to return to the digestion compartment instead of leaving the system. Although part of the lightest particles is lost together with the effluent, the average solids retention time in the reactor is maintained sufficiently high to sustain the growth of a dense mass of methane-forming microorganisms, in spite of the reduced hydraulic detention time.

One of the fundamental principles of the process is its ability to develop a high-activity biomass. This biomass can be in the form of flocs or granules (1 to 5 mm). The cultivation of a good-quality anaerobic sludge is achieved through a careful start-up of the process, during which the artificial selection of the biomass is imposed, allowing the lightest poor-quality sludge to be washed out of the system while retaining the good-quality sludge. The heaviest sludge usually grows close to the bottom of the reactor, presenting a total solids concentration in the order of 40 to 100 gTS/L. Normally mechanical mixing devices are not used, as they seem to have an adverse effect on the aggregation of the sludge and, consequently, on the formation of granules.

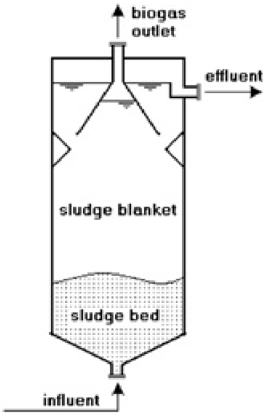


Figure 26.14. Schematic representation of an UASB reactor

The second fundamental principle of the process is the presence of a gas and solids separation device, which is located in the upper part of the reactor. The main purpose of this device is the separation of the gases contained in the liquid mixture, so that a zone favouring sedimentation is created in the upper part of the reactor.

The design of UASB reactors (Figure 26.14) is very simple and does not require the installation of any sophisticated device or packing medium for biomass attachment and retention. The process was initially developed for the treatment of concentrated wastewater, with very good results. However, similarly to the expanded bed process, in warm-climate regions, UASB reactors have also been applied for the treatment of low-concentration wastewater (domestic sewage) with very good results. As a consequence, UASB reactors are currently one of the preferred alternatives for sewage treatment in these regions. More details about the design and operation of UASB reactors are given in Chapter 27.

(d) Expanded granular sludge bed anaerobic reactor

The expanded granular sludge bed (EGSB) anaerobic reactor (Figure 26.15) greatly resembles the UASB reactor, except in respect to the sludge type and the expansion degree of the sludge bed. Mainly granular-type sludge is retained in the EGSB reactor and is maintained expanded because of the high hydraulic rates applied to the system. This condition intensifies the hydraulic mixing in the reactor and makes a better biomass–substrate contact. The high surface velocities of the liquid in the reactor (in the order of 5 to 10 m/hour) are achieved through the application of a high effluent recirculation rate, combined with the use of reactors with a high height/diameter ratio, around 20 or more (Kato, 1994; Lettinga, 1995). In contrast, in the UASB reactors, the sludge bed remains somewhat static, since the surface velocities of the liquid are usually lower, in the order of 0.5 to 1.5 m/hour.

Regarding the applicability of EGSB reactors, these are mainly intended for the treatment of soluble effluents, as the high surface velocities of the liquid inside

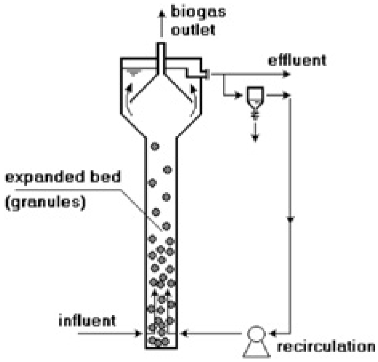


Figure 26.15. Schematic representation of an expanded granular bed reactor

the reactor do not enable the efficient removal of particulate organic materials. In addition, the excessive presence of suspended solids in the influent can be detrimental to the maintenance of the good characteristics of the granular sludge in the reactor.

As a practical result of the high upward velocities applied to the expanded granular sludge bed reactors, they can be much higher, in the order of 20 m, which results in a significant reduction in the area required. This is particularly interesting in the case of treatment of soluble effluents from industries with little space available. Figure 26.16 illustrates the volumetric organic loads that can be applied to EGSB and UASB reactors considering the treatment of low-concentration soluble wastewater assuming: (i) a granular sludge concentration of 25 gVSS/L; and (ii) 100% acidified effluent (volatile fatty acids).

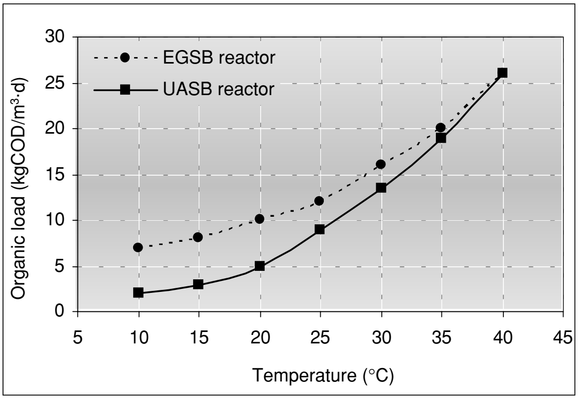


Figure 26.16. Volumetric organic loads in UASB and EGSB reactors (adapted from Lettinga, 1995)

(e) Anaerobic reactor with internal recirculation

The anaerobic reactor with internal recirculation can be considered a variation of the UASB reactor, and has been developed with the objective of guaranteeing a larger efficiency when submitted to high volumetric organic loads (up to 30 to 40 kgCOD/m^3·d). To allow the application of high loads, it is necessary to have a more efficient gas, solids and liquid separation, as the high turbulence caused by the production of gases hinders the biomass retention in the system.

In the reactor with internal recirculation, the gas, solids and liquid separation is done in two stages:

- In the first stage the separation of the largest portion of the biogas produced in the system occurs, thereby decreasing the turbulence in the upper part of the reactor.
- In the second stage the separation of the solids occurs, which guarantees high biomass retention in the system and a more clarified effluent.

Basically, the reactor with internal recirculation consists of two UASB reactor compartments, one on top of the other, with the first compartment being subjected to high organic loads. This specific task of gas separation in two stages is done in a larger height reactor (16 to 20 m), making the gases collected in the first stage drag the internal mixture (gas, solids and liquid) to the upper part of the reactor (gas lifting effect). After the separation of the gases in the upper part of the reactor, solids and liquids recirculate to the first compartment, which provides high mixing and the contact of the recirculated biomass with the influent wastewater at the base of the reactor (see Figure 26.17).

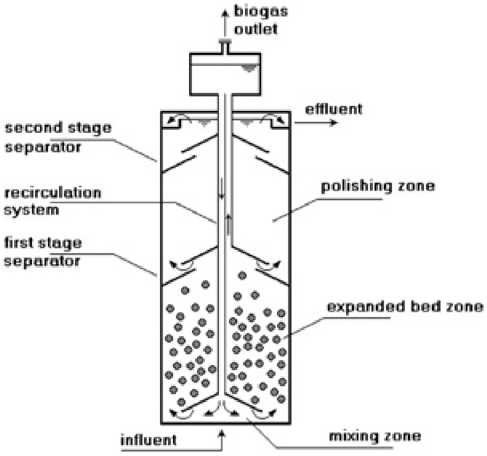


Figure 26.17. Schematic representation of a reactor with internal recirculation

According to Yspeert et al. (1995), the reactor with internal recirculation incorporates four basic items:

- *Mixing zone*: located at the bottom of the reactor, making possible an effective mixture of the influent wastewater with the biomass and the effluent from the recirculation device. This results in dilution and conditioning of the raw influent waste.
- *Expanded bed zone*: located immediately above the base of the reactor and constitutes the first stage of the reactor. This area contains the high-concentration granular sludge maintained expanded owing to the high upflow velocities caused by the influent, by the recirculation flow and by the biogas produced. The effective contact between the influent waste and the biomass results in a high sludge activity, making possible the application of high organic loads, and in high conversion rates. The high intensity of the biomass mixing in the zone favours the application of this reactor type for the treatment of highly concentrated wastewaters.
- *Polishing zone*: constitutes the second stage of the reactor and is located immediately above the separator of the expanded bed zone. In this area, effective post-treatment and additional biomass retention occur owing to three principal aspects: (i) low applied loads; (ii) high hydraulic detention times; and (iii) proximity to a plug-flow regime. As a result of the almost complete biodegradable COD removal in the expanded bed zone and the collection of gases by the first separator, the turbulence caused by the upward velocity of the liquid in the polishing zone is low.
- *Recirculation system*: comprises a device that makes the internal circulation possible through the gas-lift principle. This condition is created by the difference in the biogas capture between the upflow (gas, solids and liquid flow) and downflow (solids and liquid flow) branches of the recirculation system, without the need for any type of pumping. In studies performed in a pilot reactor of 17 m^3, treating wastes with a concentration of 3,500 mgCOD/L, a recirculation flow approximately 2.5 times the gas flow was obtained.

26.4 COMBINED TREATMENT SYSTEMS

In this chapter, the main anaerobic systems currently used for the treatment of solid and liquid wastes were described and classified, for convenience, into *conventional systems* and *high-rate systems*. There is a consensus that, in most of the applications, the anaerobic systems should be considered a first stage of the treatment, as they are not capable of producing final effluents with very good quality.

Obviously, in some situations, depending on the characteristics of the influent wastewater and the final discharge quality requirements, anaerobic systems can constitute complete treatment, or the first phase (in time) in the implementation of the treatment system along the planning horizon. However, in most of the situations, a combined treatment system has been used to obtain the substantial advantages of the incorporation of an anaerobic system as the first stage, followed by a post-treatment system. In this respect, several post-treatment alternatives have been researched, reported and implemented in the last few years, including both aerobic and anaerobic systems. Virtually all processes capable of treating raw sewage are also capable of acting as post-treatment for the effluent from anaerobic reactors. Post-treatment of anaerobic effluents is covered in Chapter 29.

27

Design of anaerobic reactors

27.1 ANAEROBIC FILTERS

27.1.1 Preliminaries

The first works on anaerobic filters date from the late 1960s and ever since they have had a growing application, representing today an advanced technology for the effective treatment of domestic sewage and a diversity of industrial effluents. The upflow anaerobic filter is basically a contact unit, in which sewage passes through a mass of biological solids contained inside the reactor. The biomass retained in the reactor can be in three different forms:

- thin biofilm layer attached to the surfaces of the packing medium
- dispersed biomass retained in the interstices of the packing medium
- flocs or granules retained in the bottom compartment, below the packed bed

The soluble organic compounds contained in the influent sewage come in contact with the biomass, being diffused through the surfaces of the biofilm or the granular sludge. They are then converted into intermediate and final products, specifically methane and carbon dioxide.

The usual configurations of anaerobic filters are either upflow or downflow. In upflow filters, the packing bed is necessarily submerged. The downflow filters can work either submerged or non-submerged. They are usually covered, but they can be implemented uncovered, when there is no concern with the possible release of bad odours.

Biological Wastewater Treatment in Warm Climate Regions by Marcos von Sperling and Carlos Augusto de Lemos Chernicharo. ISBN: 1 84339 002 7. Published by IWA Publishing, London, UK.

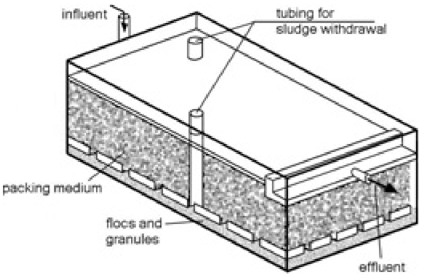


Figure 27.1. Schematic drawing of an upflow anaerobic filter (adapted from Gonçalves et al., 2001)

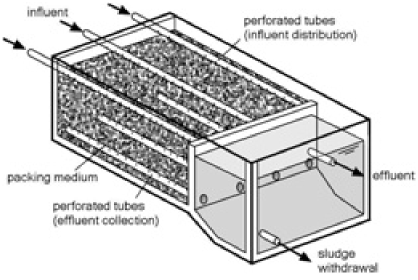


Figure 27.2. Schematic drawing of a submerged downflow anaerobic filter (adapted from Gonçalves et al., 2001)

Figures 27.1 and 27.2 present schematic drawings of submerged downflow and upflow anaerobic filters, where the main devices that guarantee the proper functioning of the treatment unit can be observed (Gonçalves *et al.*, 2001).

Although anaerobic filters can be used as the main wastewater treatment unit, they are more appropriate for post-treatment (polishing), adding operational safety and stability to the treatment system as a whole.

The effluent from anaerobic filters is usually well clarified and has a relatively low concentration of organic matter, although it is rich in mineral salts. It is very good for land application, not only for infiltration, but also for irrigation with crop production purposes, provided that the concern with pathogenic microorganisms, usually present in large amounts in the effluents from filters that treat domestic sewage, is not disregarded. In these cases, disinfection may become necessary, and the usual existing processes can be applied.

The main limitations of the anaerobic filters result from the risk of bed obstruction (clogging of the interstices) and from the relatively large volume, due to the space occupied by the inert packing material.

Anaerobic filters have been used in different system configurations in Brazil, for the post-treatment of effluents from medium and large anaerobic reactors, as illustrated in Figures 27.3 and 27.4 (Gonçalves *et al.*, 2001).

27.1.2 Physical aspects

(a) Reactor configuration

Anaerobic filters can have several shapes, configurations and dimensions, provided that the flow is well distributed over the bed. In full scale, anaerobic filters usually present either a cylindrical or a rectangular shape. The diameters (or width) of the tanks vary from 6 to 26 m, and their height from 3 to approximately 13 m. The volumes of the reactors vary from 100 to 10,000 m^3. The packing media have been designed to occupy from the total depth of the reactor to approximately 50 to 70% of the height of the tanks. There are different types of plastic packing

Figure 27.3. Anaerobic filter after upflow anaerobic sludge blanket (UASB) reactor (source: Colombo WWTP, SANEPAR/ Brazil)

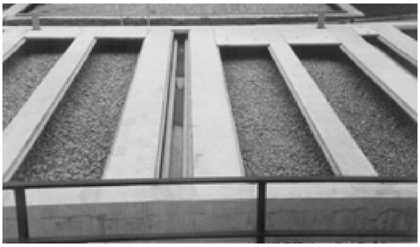


Figure 27.4. Anaerobic filter after UASB reactor (source: Ipatinga WWTP, COPASA, Brazil)

mediums available in the market, ranging from corrugated rings to corrugated plate blocks. The specific surface area of these plastic materials usually ranges from 100 to 200 m^2/m^3. Although some types of packing media are more efficient than others in the retention of biomass, the final choice will depend on the local specific conditions, on economic considerations and on operational factors.

The most recent installations of upflow anaerobic filters have been of the hybrid type, in which there is a zone without packing material, located at the lower part of the reactor, which allows the accumulation of granular sludge. The performance of the hybrid anaerobic filters depends on the contact of the wastewater with the biomass dispersed on the sludge bed and with the biofilm attached to the packing medium. The determination of the amount of packing material to be used in hybrid reactors is still subjective. There is a minimum amount that should be enough to promote some complementary removal of organic matter, and also to help in the retention of biological solids. As recommended by Young (1991), the packed bed should be placed in the upper two-thirds of the height of the reactor, and this medium should not be lower than 2 m. Lower heights should only be adopted from pilot tests or in full-scale systems treating the same type of effluent.

It should be emphasised that the recommendations made by Young (1991) refer mainly to the use of anaerobic filters for treatment of industrial effluents, a situation in which the COD removal occurs throughout the height of the packed bed. In the treatment of more diluted effluents, such as domestic sewage, the removal of organic matter occurs mainly in the lower part of the anaerobic filter (in the bottom compartment and in the beginning of the packed bed), which leads to the use of reduced heights of packing medium.

(b) Packing medium

The purpose of the packing medium is to retain solids inside the reactor, either by the biofilm formed on the surface of the packing medium or by the retention

Table 27.1. Requirements for packing media of anaerobic filters

Requirement	Objective
▪ Be structurally resistant	▪ Support their own weight, added to the weight of the biological solids attached to the surface
▪ Be biologically and chemically inert	▪ Allow no reaction between the bed and the microorganisms
▪ Be sufficiently light	▪ Avoid the need for expensive, heavy structures, and allow the construction of relatively higher filters, which implies a reduced area necessary for the installation of the system
▪ Have a large specific area	▪ Allow the attachment of a larger quantity of biological solids
▪ Have a high porosity	▪ Allow a larger free area available for the accumulation of bacteria and reduce the possibility of clogging
▪ Enable the accelerated colonisation of microorganisms	▪ Reduce the start-up time of the reactor
▪ Present a rough surface and a non-flat format	▪ Ensure good attachment and high porosity
▪ Have a reduced price	▪ Make the process feasible, not only technically, but also economically

Source: Adapted from Pinto and Chernicharo (1996) and Souza (1982), quoted by Carvalho (1994)

of solids in the interstices of the medium or below it. The main purposes of the support layer are as follows:

- to act as a device to separate solids from gases
- to help promote a uniform flow in the reactor
- to improve the contact between the components of the influent wastewater and the biological solids contained in the reactor
- to allow the accumulation of a large amount of biomass, with a consequently increased solids retention time
- to act as a physical barrier to prevent solids from being washed out from the treatment system

Table 27.1 presents the main desirable requirements for packing medium of anaerobic filters.

Several types of materials have been used as packing media in biological reactors, including quartz, ceramic blocks, oysters and mussel shells, limestone, plastic rings, hollow cylinders, PVC modular blocks, granite, polyethylene balls, bamboo, etc.

Recent studies demonstrated the applicability and feasibility of another packing medium alternative for anaerobic filters: blast furnace slag. This material has been used for over 5 years, and no indication of deterioration or bed clogging has been noticed. The samples removed for analyses demonstrated the integrity of the stones and the high attachment capacity of the anaerobic biofilm (Pinto, 1995; Pinto and Chernicharo, 1996).

The clogging of the packing medium has been one of the main concerns of designers and users of anaerobic filters. These problems are more associated with upflow anaerobic filters using stone and crushed stone as packing material. The most modern filters, packed with plastic material, have had no clogging problems, even when the specific surface areas of the packing medium are low, amounting to 100 m^2/m^3. To minimise the clogging effects of the packing medium, cleaning devices should be considered over the height of the filter, to remove the excess solids retained in the filtering medium. The operational aspects are also important to avoid the clogging of the filter, as discussed in Chapter 28.

27.1.3 Hydraulic aspects

(a) Recirculation of effluent

The function and benefits of effluent recirculation in anaerobic filters are not well defined yet. By means of experiments made in laboratories, it has been noticed that the application of recirculation rates of up to 10 times the influent flow provides an improved efficiency to the system. A significantly reduced efficiency was noticed above the recirculation ratio of 10:1.

Recirculation of effluents from either upflow or downflow anaerobic filters is not usually necessary when treating domestic effluents from septic tanks, considering that the concentrations of influent organic matter to the anaerobic filter are not very high (Andrade Neto, 1997).

The recirculation of effluents should not be the first method to lessen the transient conditions of influent loads. High recirculation rates can cause the increase of the upflow velocities, with the consequent loss of biomass.

(b) Upflow velocity

Besides the hydraulic detention time and the effluent recirculation, other hydraulic factors intervening in the process are the upflow velocity and the flow variations. The upflow velocity should be maintained below the limit above which solids are significantly lost in the effluent. In full-scale reactors, the upflow velocity, including the recirculation flow, is usually around 2 m/hour. However, the maximum upflow velocity depends on the density of the suspended solids and on the magnitude of the granulation. The upflow velocity should be maintained low during the start-up of the process, to reduce solids wash out in the effluent. During start-up, effluent recirculation can favour pH control in the reactor, so that the upflow velocities (including the recirculation) do not exceed 0.4 m/hour. The recirculation rates can be gradually increased as the reactor advances to maturity, but upflow velocities higher than 1.0 m/hour can cause an excessive loss of solids.

27.1.4 Performance relationships

Although pilot and laboratory studies contribute to the development of relationships between the several design and operational factors, a general relationship of unrestricted acceptance has not yet been developed to be used in the design of full-scale anaerobic filters.

Young (1991) gathered operational data from several anaerobic filters and established a statistical correlation among them, aiming at the determination of the parameters that influenced the performance of the system. The parameters analysed in the multiple linear regression models included hydraulic detention time, wastewater concentration, surface area of the packing medium, slope of the corrugated plates of the packing medium and volumetric organic load. The statistical studies indicated that the hydraulic detention time was the parameter that had a higher influence on COD removal efficiency in the system, for reactors packed with both synthetic medium and stones. Regarding the corrugated modules, the increased surface area seemed not to influence significantly the efficiency of the system, while the size of the empty spaces and the geometry of the corrugated material did influence the efficiency of the reactors. In addition, the introduction of the slope of the corrugated plates in the linear regression model had a positive impact on the correlation of the analysed data. The general relationship capable of describing the performance of anaerobic filters treating different types of effluents proposed by Young (1991) was

$$E = 100 \times (1 - S_k \times t^{-m}) \tag{27.1}$$

where:

E = efficiency of the system (%)
t = hydraulic detention time (hour)
S_k = coefficient of the system
m = coefficient of the packing medium

It is worth mentioning that this relation is used to estimate the performance of full-scale and laboratory reactors with relative precision, when they use cross-flow synthetic packing medium with a surface area of approximately 100 m^2/m^3. For this situation, the coefficients S_k and m assume values of 1.0 and 0.55, respectively. For stone bed reactors, the value of the coefficient m is approximately 0.40.

Treatment efficiency is also related to temperature by means of the following expression:

$$E_T = 1 - (1 - E_{30})\,\theta^{(T-30)} \tag{27.2}$$

where:

E_T = efficiency of the process at temperature T (°C)
E_{30} = efficiency of the process at the temperature of 30 °C
T = operational temperature (°C)
θ = temperature coefficient (1.02 to 1.04)

27.1.5 Design criteria

The use of anaerobic filters for the treatment of domestic wastewater has been intended mainly for the polishing of effluents from septic tanks and UASB reactors. In this serial configuration, the main design considerations are described below.

(a) Hydraulic detention time

The hydraulic detention time refers to the average time of residence of the liquid inside the filter, calculated by the following expression:

$$\boxed{t = \frac{V}{Q}} \tag{27.3}$$

where:

t = hydraulic detention time (hour)
V = volume of the anaerobic filter (m^3)
Q = average influent flowrate (m^3/d)

In the case of anaerobic filters applied to the post-treatment of effluents from anaerobic reactors, the design criteria and parameters are still very scarce. The result of studies developed by the Brazilian National Research Programme on Basic Sanitation, PROSAB (Gonçalves *et al.*, 2001), using anaerobic filters filled with a stone bed for the polishing of effluents from septic tanks and UASB reactors, showed that they are capable of producing effluents that meet less stringent discharge standards (BOD $\leq$ 60 mg/L, TSS $\leq$ 40 mg/L), when operated under hydraulic detention times ranging from **4 to 10 hours**.

(b) Temperature

Anaerobic filters can be satisfactorily operated at temperatures ranging from 25 to 38 °C. Usually, the degradation of complex wastewater, whose first stage of the fermentation process is hydrolysis, requires temperatures higher than 25 °C. Otherwise, hydrolysis may become the limiting stage of the process.

Observations carried out in laboratory and full-scale reactors indicate that short-term temperature changes are capable of altering COD removal efficiency more than if the reactors were operated at two different, but constant temperatures.

In spite of the recommendation that anaerobic filters should be operated within the temperature range from 25 to 38 °C, satisfactory results have been observed for filters operating within the temperature range from 20 to 25 °C (and even lower), especially when applied to the post-treatment of effluents from septic tanks and UASB reactors (Gonçalves *et al.*, 2001) .

(c) Packing medium height

Based on the Brazilian experience and on studies developed by the Brazilian National Research Programme on Basic Sanitation, PROSAB (Gonçalves *et al.*, 2001) using anaerobic filters filled with a stone bed for the polishing of effluents from septic tanks and UASB reactors, it is recommended for most applications that the packed bed height should be between **0.8 and 3.0 m**. The upper height limit of the packed bed is more appropriate for reactors with lower risk of bed obstruction, which depends mostly on the flow direction, on the type of packing material and on the influent concentrations. A more usual value should amount to approximately **1.5 m**.

(d) Hydraulic loading rate

The hydraulic loading rate refers to the volume of wastewater applied daily per unit area of the filter packing medium, as calculated by Equation 27.4,

$$\boxed{\mathrm{HLR} = \frac{Q}{A}} \tag{27.4}$$

where:

HLR = hydraulic loading rate ($m^3/m^2 \cdot d$)
Q = average influent flowrate (m^3/d)
A = surface area of the packing medium (m^2)

The result of studies developed by the Brazilian National Research Programme on Basic Sanitation, PROSAB (Gonçalves *et al.*, 2001), using anaerobic filters filled with a stone bed for the polishing of effluents from septic tanks and UASB reactors, showed that the filters are capable of producing effluents of good quality when operated under surface hydraulic loading rates ranging from **6 to 15 $m^3/m^2 \cdot d$**.

(e) Organic loading rate

The volumetric organic loading rate refers to the load of organic matter applied daily per unit volume of the filter or packing medium, as calculated by Equation 27.5,

$$\boxed{L_v = \frac{Q \times S_0}{V}} \tag{27.5}$$

where:

L_v = volumetric organic loading rate (kgBOD/$m^3 \cdot d$ or kgCOD/$m^3 \cdot d$)
Q = average influent flowrate (m^3/d)
S_0 = influent BOD or COD concentration (kgBOD/m^3 or kgCOD/m^3)
V = total volume of the filter or volume occupied by the packing medium (m^3)

While anaerobic filters have been designed to support organic loads of up to 16 kgCOD/$m^3 \cdot d$ (considering the total volume), the operational loads do not usually exceed 12 kgCOD/$m^3 \cdot d$, except when the wastewater presents concentrations higher than 12,000 mgCOD/L. This implies the existence of a concentration above which filters are designed based on the organic loading criterion, and below which the design is based on the hydraulic loading criterion. For the treatment of *domestic*

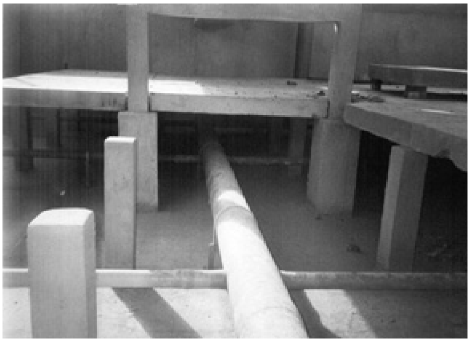


Figure 27.5. Sewage distribution device at the bottom of an anaerobic filter (Ipatinga WWTP, COPASA, Brazil)

Figure 27.6. Effluent collection launder on the top of an anaerobic filter (Ipatinga WWTP, COPASA, Brazil)

sewage, the design of anaerobic filters is ruled by the hydraulic detention time parameter.

Studies made by PROSAB indicated that the anaerobic filters are capable of producing good-quality effluents when operated under organic loading rates from **0.15 to 0.50 kgBOD/m^3·d** (total filter volume) and from **0.25 to 0.75 kgBOD/m^3·d** (packed bed volume).

(f) Effluent distribution and collection systems

A very important aspect of the design of anaerobic filters concerns the detailing of the wastewater inlet and outlet devices, since the efficiency of the treatment system depends substantially on the good distribution of the flow on the packing bed, and this distribution is subject to the correct calculation of the inlet and outlet devices.

In the case of upflow anaerobic filters, one flow distribution tube has been used for **every 2.0 to 4.0 m^2** of filter bottom area. Figures 27.5 and 27.6 show the wastewater distribution device, through perforated tubes, and the effluent collection launder. The details of the bottom compartment and the perforated slab that will sustain the packing bed are shown in these figures.

(g) Sludge sampling and removal devices

These devices are intended mainly for monitoring the growth and quality of the biomass in the reactor, enabling more control actions over the solids in the system. Thus, the design of anaerobic filters should allow easy means for the sampling and periodical removal of the sludge, by means of appropriate and sufficient devices. At least two sludge samplers should be included, one close to the bottom and the other immediately below the packed bed, to allow the monitoring of the concentration and height of the sludge bed. Additionally, other sludge samplers can be planned over the height of the packed bed (every 0.5 or 1.0 m). These samplers help considerably to plan the discharge of the excess sludge before it can adversely influence through blockage and clogging of the packing medium.

(h) Efficiencies of anaerobic filters

The expected efficiencies for anaerobic filters can be estimated from the performance relationship presented in Equation 27.1. However, as this relation is empirical, having the hydraulic detention time and the characteristics of the packing medium as main dependent variables, its limitations should be recognised. Van Haandel & Lettinga (1994) propose other empirical constants for Equation 27.1, obtained from the fitting of experimental data from different researches on anaerobic filters:

$$E = 100 \times (1 - 0.87 \times t^{-0.50}) \tag{27.6}$$

where:

E = efficiency of the anaerobic filter (%)
t = hydraulic detention time (hour)
0.87 = empirical constant (coefficient of the system)
0.50 = empirical constant (coefficient of the packing medium)

However, van Haandel and Lettinga (1984) emphasise the limitation of Equation 27.6 in two aspects:

- absence of reports about the use of real-scale anaerobic filters treating domestic sewage
- limited number of data used for determination of the empirical constants of Equation 27.6, which showed great deviations amongst themselves.

Pilot-scale research using anaerobic filters as the first treatment unit, preceded only by preliminary treatment devices (fine screening and grit removal), indicated average BOD and COD removal efficiencies ranging between 68 and 79%. These results were obtained for filters treating domestic wastewater, operating with constant flow and hydraulic detention times varying from 6 to 8 hours (Pinto, 1995).

In situations in which the anaerobic filters are used as post-treatment units for effluents from septic tanks and UASB reactors, the BOD removal efficiencies expected for the system as a whole vary from 75 to 85%.

From the efficiency expected for the system, the COD or BOD concentration in the final effluent can be estimated as follows:

$$C_{effl} = S_0 - \frac{E \times S_0}{100} \tag{27.7}$$

where:

C_{effl} = effluent total BOD or COD concentration (mg/L)
S_0 = influent total BOD or COD concentration (mg/L)
E = BOD or COD removal efficiency (%)

Table 27.2. Design criteria for anaerobic filters applied to the post-treatment of effluents from anaerobic reactors

Design criteria/parameter	Range of values, as a function of the flowrate		
	for $Q_{average}$	for $Q_{daily\text{-}maximum}$	for $Q_{hourly\text{-}maximum}$
Packing medium	Stone	Stone	Stone
Packing bed height (m)	0.8 to 3.0	0.8 to 3.0	0.8 to 3.0
Hydraulic detention time* (hour)	5 to 10	4 to 8	3 to 6
Surface loading rate ($m^3/m^2 \cdot d$)	6 to 10	8 to 12	10 to 15
Organic loading rate ($kgBOD/m^3 \cdot d$)	0.15 to 0.50	0.15 to 0.50	0.15 to 0.50
Organic loading in the packed bed ($kgBOD/m^3 \cdot d$)	0.25 to 0.75	0.25 to 0.75	0.25 to 0.75

* The adoption of the lower limits of HDT for the design of anaerobic filters requires special care regarding the type of packing medium, the presence of TSS in the influent and the height of the packing bed. Besides that, the operational routine will demand a higher sludge discharge frequency, to avoid clogging problems.

Source: Gonçalves *et al.* (2001)

(i) Summary of design criteria

A summary of the main criteria and parameters for the design of anaerobic filters, applied to the post-treatment of effluents from anaerobic reactors, as covered in the previous items, is presented in Table 27.2.

Example 27.1

Design an anaerobic filter for the post-treatment of effluents generated in a UASB reactor, with the following design elements being known:

Data:

- Population: P = 20,000 inhabitants
- Average influent flowrate: $Q_{av} = 3{,}000\ m^3/d$
- Maximum daily influent flowrate: $Q_{max\text{-}d} = 3{,}600\ m^3/d$
- Maximum hourly influent flowrate: $Q_{max\text{-}h} = 5{,}400\ m^3/d$
- Influent organic load to the UASB reactor: $L_{0\text{-}UASB} = 1{,}000$ kgBOD/d
- Average influent BOD concentration to the UASB reactor: $S_{0\text{-}UASB} = 333$ mg/L
- BOD removal efficiency expected for the UASB reactor: 70%
- Influent organic load to the anaerobic filter: $L_{0\text{-}AF} = 300$ kgBOD/d
- Average influent BOD concentration to the anaerobic filter: $S_{0\text{-}AF} = 100$ mg/L

Solution:

(a) Adoption of a hydraulic detention time (t)

According to Table 27.2, the anaerobic filters should be designed with HDT between 3 and 10 hours. Value adopted: t = 8 hours (for average flowrate)

Example 27.1 (Continued)

(b) Calculation of the volume of the filter, according to Equation 27.3 (V)

$$V = (Q \times t) = [(3{,}000\ m^3/d)/(24hours/d)] \times 8\ hours = 1{,}000\ m^3$$

(c) Adopt depth for the packed bed and for the filter:

According to Table 27.2, the anaerobic filters should be designed with packed bed heights between 0.80 and 3.00 m. Value adopted for the packed bed: $h_1 = 1.50\ m$

The height of the bottom compartment (h_2) and free depth to the effluent collection launder (h_3) should also be defined. Values adopted: $h_2 = 0.60\ m$ and $h_3 = 0.30\ m$.

The total resulting depth for the filter will be:

$$H = h_1 + h_2 + h_3 = 1.50 + 0.60 + 0.30 = 2.40\ m$$

(d) Calculation of the area of the anaerobic filter (A)

$$A = V/H = (1{,}000\ m^3)/(2.40\ m) = 416.7\ m^2$$

(e) Calculation of the volume of the packed bed (V_{pb})

$$V_{pb} = A \times h_1 = 416.7\ m^2 \times 1.50\ m = 625.1\ m^3$$

(f) Verification of the hydraulic loading rate (HLR), according to Equation 27.4

For average flowrate: $HLR_1 = Q_{av}/A = (3{,}000\ m^3/d)/(416.7\ m^2) = 7.2\ m^3/m^2{\cdot}d$

For maximum daily flowrate: $HLR_2 = Q_{max\text{-}d}/A = (3{,}600\ m^3/d)/(416.7\ m^2) = 8.6\ m^3/m^2{\cdot}d$

For maximum hourly flowrate: $HLR_3 = Q_{max\text{-}h}/A = (5{,}400\ m^3/d)/(416.7\ m^2) = 13.0\ m^3/m^2{\cdot}d$

According to Table 27.2, it is verified that the surface hydraulic loading rate values are within the recommended ranges for the three flow conditions applied.

(g) Verification of the average organic loading rate applied to the anaerobic filter and to the packed bed (L_v), according to Equation 27.5

$$L_{v1} = (Q \times S_0)/V = [(3{,}000\ m^3/d) \times (0.100\ kgBOD/m^3)]/(1{,}000m^3)$$
$$= 0.30\ kgBOD/m^3{\cdot}d$$

***Example 27.1* (*Continued*)**

$$L_{v2} = (Q \times S_0)/V_{pb} = [(3{,}000\ \text{m}^3/\text{d}) \times (0.100\ \text{kgBOD/m}^3)]/(625.1\ \text{m}^3)$$
$$= 0.48^{*}\ \text{kgBOD/m}^3{\cdot}\text{d}$$

(*) In practice, it is noticed that a large part of the influent organic load is removed in the lower part (bottom compartment) of the anaerobic filter, which makes the volumetric organic loads applied to the packed bed much lower.

(h) Determination of the filter dimensions

Adopt 2 square section filters, each with an area of 208.8 m^2 (14.45 m × 14.45 m)

(i) Estimation of the efficiency of the anaerobic filter (E), according to Equation 27.6:

$$E = 100 \times (1 - 0.87 \times t^{-0.50}) = 100 \times (1 - 0.87 \times 8^{-0.50}) = 69\%$$

(j) Estimation of the BOD concentration in the final effluent (equation 27.7):

$$BOD_{effl} = S_0 - (E{\cdot}S_0)/100 = 100 - (69\% \times 100)/100 = 31\ \text{mg/L}$$

27.2 UPFLOW ANAEROBIC SLUDGE BLANKET REACTORS

27.2.1 Preliminaries

The use of UASB reactors for the treatment of domestic sewage is already a reality in tropical countries, especially in Brazil, Colombia and India. The successful experience in these countries is a strong indication of the potential of this type of reactor for the treatment of domestic sewage.

The anaerobic process through UASB reactors presents several advantages in relation to conventional aerobic processes, especially when applied in warm-climate locations, such as most of the developing countries. In these situations, a system can have the following main characteristics:

- compact system, with low land requirements
- low construction and operating costs
- low sludge production
- low energy consumption (just for the influent pumping station, when necessary)
- satisfactory COD and BOD removal efficiencies, amounting to 65 to 75%
- high concentration and good dewatering characteristics of the excess sludge

Although the UASB reactors present many advantages, there are still some disadvantages or limitations:

- possibility of release of bad odours
- low capacity of the system in tolerating toxic loads
- long time interval necessary for the start-up of the system
- need for a post-treatment stage

In situations in which the wastewater is predominantly domestic, the presence of sulfur compounds and toxic materials usually occurs at very low levels, being well handled by the treatment system. When well designed, constructed and operated, the system should not present bad smell and failure problems due to the presence of toxic elements and/or inhibitors.

The start-up of the system can be slow (4 to 6 months), but only in situations in which seed sludge is not used. In the past few years, with the use of well-based start-up methodologies and the establishment of appropriate operational routines, significant progresses were achieved towards reducing the start-up period of the systems and minimising the operational problems in this phase. In situations already reported (Chernicharo and Borges, 1996), in which small amounts of seed sludge were used (less than 4% of the reactor volume), the start-up period was reduced to 2 or 3 weeks. In any case, the quality of the biomass to be developed in the system will depend on an appropriate operational routine and, consequently, on the stability and efficiency of the treatment process.

However, apart from the great advantages of the UASB reactors, the quality of the effluent produced usually does not comply with most discharge standards established by environmental agencies. Until recent years there were not many experiences that consolidate an overall view of the combined stages of *anaerobic treatment* and *post-treatment.* However, important advances have been achieved recently, as mentioned by Chernicharo *et al.* (2001b).

The design of UASB reactors is very simple and does not require the installation of any sophisticated equipment or packing medium for biomass retention. In spite of the accumulated knowledge on UASB reactors, there are still no clear, systematised guidelines accessible by designers for the design of these reactors. It is important that the several design criteria and parameters for UASB reactors are expressed in a clear and sequential manner, allowing the dimensioning of the reaction, sedimentation and gas capture chambers.

27.2.2 Process principles

The reactor is initially inoculated with sufficient quantities of anaerobic sludge, and its low-rate feeding is started soon afterwards, in the upflow mode. This initial period is referred to as *start-up* of the system, being the most important phase of the operation of the reactor. The feeding rate of the reactor should be increased progressively, according to the success of the system response. After some months of operation, a highly concentrated *sludge bed* (4 to 10%, that is, 40 to 100 gTS/L)

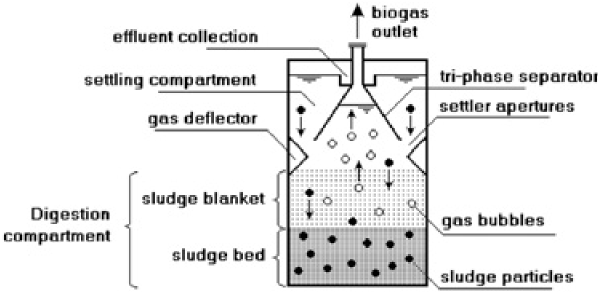


Figure 27.7. Schematic drawing of a UASB reactor

is developed close to the bottom of the reactor. The sludge is very dense and has excellent settling characteristics. The development of sludge granules (diameters from 1 to 5 mm) may occur, depending on the nature of the seeding sludge, on the characteristics of the wastewater and on the operational conditions of the reactor.

An area of more dispersed bacterial growth, named *sludge blanket*, is developed above the sludge bed, with solids presenting lower concentrations and settling velocities. The concentration of sludge in this area usually ranges between 1 and 3%. The system is self-mixed by the upflow movement of biogas bubbles and by the liquid flow through the reactor. During the start-up of the system, when the biogas production is usually low, some form of additional mixing, such as by the recirculation of gas or effluent, may become necessary. Substrate is removed throughout the bed and sludge blanket, although removal is more pronounced at the sludge bed.

The sludge is carried by the upflow movement of the gas bubbles, and the installation of a *three-phase separator* (gases, solids and liquids) in the upper part of the reactor is necessary, to allow sludge retention and return. There is a sedimentation chamber around and above the three-phase separator, where the heaviest sludge is removed from the liquid mass and returned to the digestion compartment, while the lightest particles leave the system together with the final effluent (see Figure 27.7).

The installation of the gas, solids and liquid separator guarantees the return of the sludge and the high retention capacity of large amounts of high-activity biomass, with no need for any type of packing medium. As a result, UASB reactors present high solids residence times (sludge age), much higher than the hydraulic detention times, which is a characteristic of the high-rate anaerobic systems. Sludge ages in UASB reactors usually exceed 30 days, leading to stabilisation of the excess sludge removed from the system.

The UASB reactor is capable of supporting high organic loading rates and the great difference, when compared with other reactors of the same generation, is its constructive simplicity and low operational costs. The most important principles that govern the operation of UASB reactors are:

- the upward flow should assure a maximum contact between the biomass and the substrate
- short circuits should be avoided, to allow retention times sufficient for the degradation of the organic matter
- the system should have a well designed device capable of separating suitably the biogas, the liquid and the solids, releasing the first two and allowing the retention of the last
- the sludge should be well adapted, with high specific methanogenic activity and excellent settling characteristics. If possible, the sludge should be granulated, once this type of sludge presents much better characteristics than those of the flocculent sludge

27.2.3 Typical configurations

UASB reactors were initially designed for the treatment of industrial effluents as cylindrical or prismatic-rectangular structures, where the areas of the digestion and sedimentation compartments were equal, therefore forming vertical wall reactors. The adaptation of these reactors to the treatment of low-concentration wastewater (such as domestic sewage) has led to different configurations, in view of the following main aspects:

- In the design of UASB-type reactors treating low-concentration sewage, the design is ruled by the hydraulic loading criteria, and not by the organic loading criteria, as discussed in the following item. In this situation, the upward velocity in the digestion and sedimentation compartments becomes essentially important: excessive velocities result in the loss of biomass from the system, thus reducing the stability of the process. Consequently, the height of the reactor should be reduced and its cross section should be increased, to keep the upward velocities within suitable ranges (see Table 27.14).
- For reactors treating industrial effluents, the influent is usually distributed from the bottom of the reactor, unlike reactors treating domestic sewage, where the influent distribution device is located in the upper part of the reactor (see Figures 27.8 to 27.10). Consequently, the surface area of the sedimentation compartment may be reduced in view of the area occupied by the influent distribution device. Thus, depending on the hydraulic loads applied to the system, it may be necessary to use larger cross sections close to the sedimentation compartment, to reduce the upward velocities and enable the sedimentation of the sludge in this compartment. In this case, the reactor adopts a variable section, smaller close to the digestion compartment and larger close to the sedimentation compartment (see Figure 27.9).
- The implementation of an equalisation tank is usually planned upstream the UASB reactor in the treatment of industrial effluents, allowing its operation to be carried out within more uniform flow and organic loading ranges.

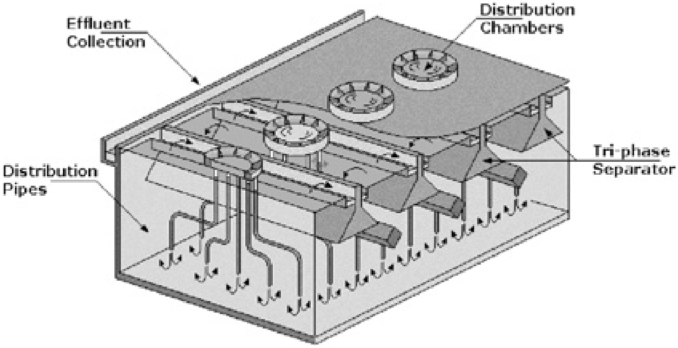


Figure 27.8. Schematic representation of a rectangular UASB reactor

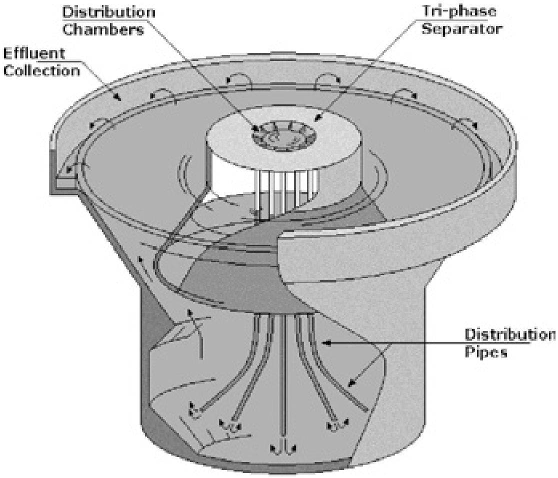


Figure 27.9. Schematic representation of a circular UASB reactor

> On the other hand, the influent to a domestic sewage treatment plant undergoes no equalisation (unless there is a pumping station), exposing the UASB reactor to flow and load variations that may be extremely high. Once again, the increased cross section of the reactor close to the sedimentation compartment may be a necessary strategy to guarantee low upward velocities during peak flows.

The shape of the reactors in plan can be either circular or rectangular. Circular reactors are more economical from the structural point of view, being used more for small populations, usually with a single unit. Rectangular reactors are more suitable for larger populations, when modulation becomes necessary, once

Figure 27.10. View of a full-scale UASB reactor
Source: Ipatinga WWTP, COPASA, Brazil

a wall can serve two contiguous modules. Figures 27.8 and 27.9 illustrate two typical configurations of UASB reactors, a rectangular one and a circular one. Figure 27.10 shows a full-scale rectangular UASB reactor.

27.2.4 Design criteria

One of the most important aspects of the anaerobic process applying UASB reactors is its ability to develop and maintain high-activity sludge of excellent settling characteristics. For this purpose, several measures should be taken in relation to the design and operation of the system.

The main design criteria for reactors treating organic wastes of either domestic or industrial nature are presented below. Specific criteria should be adopted for certain types of industrial effluents in view of the concentration of the influent wastewater, the presence of toxic substances, the amount of inert and biodegradable solids and other aspects.

(a) Volumetric hydraulic load and hydraulic detention time

The volumetric hydraulic load is the amount (volume) of wastewater applied daily to the reactor, per unit of volume. The hydraulic detention time is the reciprocal of the volumetric hydraulic load,

$$\mathrm{VHL} = \frac{\mathrm{Q}}{\mathrm{V}} \tag{27.8}$$

where:

VHL = volumetric hydraulic load ($\mathrm{m^3/m^3{\cdot}d}$)

Q = flowrate ($\mathrm{m^3/d}$)

V = total volume of the reactor ($\mathrm{m^3}$)

$$t = \frac{1}{VHL} \tag{27.9}$$

where:

t = hydraulic detention time (d)

or

$$\boxed{t = \frac{V}{Q}} \tag{27.10}$$

Experimental studies demonstrated that the volumetric hydraulic load should not exceed the value of 5.0 $m^3/m^3 \cdot d$, which is equivalent to a minimum hydraulic detention time of 4.8 hours.

The design of reactors with higher hydraulic loading values (or lower hydraulic detention times) can be detrimental to the operation of the system in relation to the following main aspects:

- excessive loss of biomass, that is washed out with the effluent, due to the resulting high upflow velocities in the digestion and settling compartments
- reduced solids retention time (sludge age), and a consequently decreased degree of stabilisation of the solids
- possibility of failure in the system, once the biomass residence time in the system becomes shorter than its growth rate

As shown previously, the hydraulic detention time parameter (t) is of fundamental importance, since it is directly related to the velocity of the anaerobic digestion process, and that, in turn, depends on the size of the reactor. For average temperatures close to 20 °C, the hydraulic detention time can vary from 6 to 16 hours, depending on the type of wastewater. Pilot-scale studies with reactors operated at an average temperature of 25 °C and fed with domestic sewage with relatively high alkalinity showed that a 4-hour hydraulic detention time did not affect the performance of these reactors or their operational stability (van Haandel and Catunda, 1998).

Hydraulic detention times ranging from **8 to 10 hours**, considering the daily average flowrate, have been adopted for the treatment of *domestic sewage* at a temperature of approximately 20 °C. The detention time for the maximum flowrate should not be shorter than 4 hours, and the maximum flow peaks should not be longer than 4 to 6 hours. Table 27.3 presents some guidelines for the establishment of hydraulic detention times in designs of UASB reactors treating domestic sewage.

Thus, knowing the influent flowrate and assuming a certain design hydraulic detention time, the volume of the reactor can be calculated by Equation 27.10, rearranged as follows:

$$\boxed{V = Q.t} \tag{27.11}$$

Table 27.3. Recommended hydraulic detention times for UASB reactors treating domestic sewage

Sewage temperature (°C)	Hydraulic detention time (hour)	
	Daily average	Minimum (during 4 to 6 hour)
16 to 19	>10 to 14	>7 to 9
20 to 26	>6 to 9	>4 to 6
>26	>6	>4

Source: Adapted from Lettinga and Hulshoff Pol (1991)

(b) Organic loading rate

The volumetric organic load is defined as the amount (mass) of organic matter applied daily to the reactor, per volume unit:

$$L_v = \frac{Q \times S_0}{V} \tag{27.12}$$

where:

L_v = volumetric organic loading rate (kgCOD/m^3·d)
Q = flowrate (m^3/d)
S_0 = influent substrate concentration (kgCOD/m^3)
V = total volume of the reactor (m^3)

Hence, knowing the flowrate and the concentration of the influent wastewater, and assuming a certain design volumetric organic load (L_v), the volume of the reactor can be calculated by Equation 27.12, rearranged as follows:

$$\boxed{V = \frac{Q \times S_0}{L_v}} \tag{27.13}$$

In the case of *industrial effluents* with a high concentration of organic matter, literature reports extremely high organic loads successfully applied to pilot facilities (45 kgCOD/m^3·d), although the organic loads adopted in the design of full-scale plants have been, as a rule, **lower than 15 kgCOD/m^3·d**. For such effluents, the volumetric organic load to be applied is what defines the reactor volume. Concerning *domestic sewage* with a relatively low concentration of organic matter (usually lower than 1,000 mgCOD/L), the volumetric organic load to be applied is much lower, ranging from **2.5 to 3.5 kg COD/m^3·d**; higher values result in excessive hydraulic loads and, consequently, in excessive upflow velocities. In this case, as stated previously, the reactor should be designed considering the volumetric hydraulic load. For example, Figure 27.11 illustrates the relation between wastewater concentration and the criteria used to determine the volume of the reactor, considering the following established data: t = 8 hours, L_v = 15 kgCOD/m^3·d and Q = 250 m^3/hour.

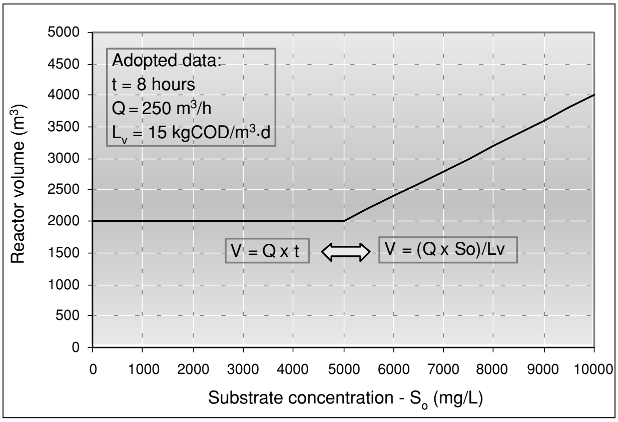


Figure 27.11. Relation between wastewater concentration and reactor volume (adapted from Lettinga and Hulshoff Pol, 1995)

(c) Biological loading rate (sludge loading rate)

The biological or sludge loading rate refers to the amount (mass) of organic matter applied daily to the reactor, per unit of biomass present:

$$\boxed{L_s = \frac{Q \times S_0}{M}} \qquad (27.14)$$

where:

L_s = biological or sludge loading rate (kgCOD/kgVS·d)
Q = average influent flowrate (m^3/d)
S_0 = influent substrate concentration (kgCOD/m^3)
M = mass of microorganisms present in the reactor (kgVS/m^3)

The procedures to determine the amount of biomass in the reactor were covered in Chapter 25.

Literature recommends that the initial biological loading rate during the start-up of an anaerobic reactor should range from 0.05 to 0.15 kgCOD/kgVS·d, depending on the type of effluent being treated. These loads should be gradually increased, according to the efficiency of the system.

The maximum biological loading rate depends on the methanogenic activity of the sludge. For domestic sewage, the methanogenic activity usually ranges from 0.3 to 0.4 kgCOD/kgVS·d, which is, therefore, the limit for the biological load.

Recent experiments with UASB reactors treating domestic sewage indicated that the application of biological loading rates ranging from 0.30 to 0.50 kgCOD/kgVS·d during the start-up of the system did not harm the stability of the process in terms of pH and volatile fatty acids.

(d) Upflow velocity and reactor height

The upflow velocity of the liquid is calculated from the relation between the influent flowrate and the cross section of the reactor, as follows:

$$\boxed{v = \frac{Q}{A}} \tag{27.15}$$

where:

v = upflow velocity (m/hour)
Q = flow (m^3/hour)
A = area of the cross section of the reactor, in this case the surface area (m^2)

or alternatively, from the ratio between the height and the HDT:

$$v = \frac{Q \times H}{V} = \frac{H}{t} \tag{27.16}$$

where:

H = height of the reactor (m)

The maximum upflow velocity in the reactor depends on the type of sludge present and on the loads applied. For reactors operating with flocculent sludge and organic loading rates ranging from 5.0 to 6.0 kgCOD/m^3·d, the average upflow velocities should amount to **0.5 to 0.7 m/hour**, with temporary peaks up to 1.5 to 2.0 m/hour being tolerated for 2 to 4 hours. For reactors operating with granular sludge, the upflow velocities can be significantly higher, amounting to 10 m/hour. For the treatment of domestic sewage, the upflow velocities presented in Table 27.4 are recommended.

A close relation between the upflow velocity the height of the reactor and the hydraulic detention time can be verified in Equation 27.16, as shown in Figure 27.12. For the upflow velocities (v) and the hydraulic detention times (t) recommended for the design of UASB reactors treating domestic sewage (v usually lower than 1.0 m/hour for Q_{av} and t between 6 and 10 hours for temperatures ranging between 20 and 26 °C), reactor depths should be between **3 and 6 m**.

Table 27.4. Upflow velocities recommended for the design of UASB reactors treating domestic sewage

Influent flowrate	Upflow velocity (m/hour)
Average flow	0.5 to 0.7
Maximum flow	<0.9 to 1.1
Temporary peak flows (*)	<1.5

(*) flowrate peaks lasting 2 to 4 hours
Source: Adapted from Lettinga and Hulshoff Pol (1995)

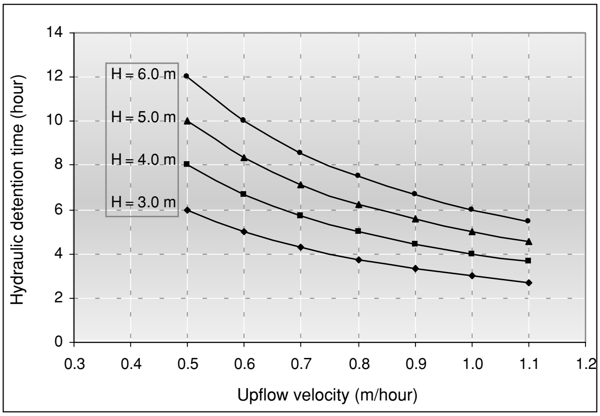


Figure 27.12. Relation between upflow velocity and HDT for different reactor heights

(e) UASB reactor efficiencies

Mathematical models applied to the design and operation of anaerobic systems have still been little used in practice, particularly for systems treating complex substrates such as domestic sewage, although valuable achievements in this field are expected in the following years with the release of the Anaerobic Digestion Model No. 1 (Batstone *et al.*, 2002), developed by the IWA task group for mathematical modelling of anaerobic digestion processes. Meanwhile, the efficiency of UASB reactors is estimated mainly by means of empirical relations, obtained from experimental results of systems in operation.

Figures 27.13 and 27.14 show the operational results of 16 full-scale UASB reactors, all of them operating within the temperature range between 20 and 27 °C, influent COD between 300 and 1,400 mg/L and influent BOD between 150 and 850 mg/L. It can be noted that the COD and BOD removal efficiencies are

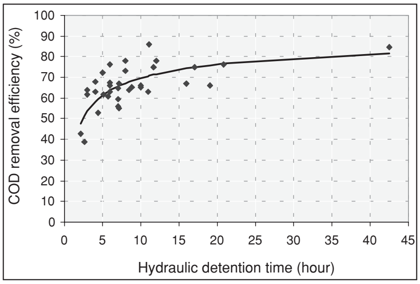


Figure 27.13. COD removal efficiencies in UASB reactors treating domestic sewage

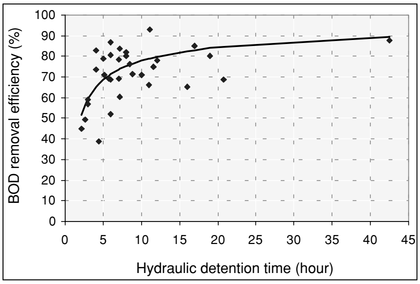


Figure 27.14. BOD removal efficiencies in UASB reactors treating domestic sewage

substantially affected by the hydraulic detention time of the system, ranging from 40 to 70% for COD removal and from 45 to 90% for BOD removal.

From the fitting of the operational results of these 16 reactors, efficiency curves were obtained and represented by Equations 27.17 and 27.18. These equations make it possible to estimate the COD and BOD removal efficiencies of UASB reactors treating domestic sewage under tropical conditions (wastewater temperature within 20 and 27 °C) as a function of the hydraulic detention time. However, their limitation should be emphasised due to the small number of data used for the determination of the empirical constants, which showed great deviations amongst themselves.

$$E_{COD} = 100 \times (1 - 0.68 \times t^{-0.35}) \tag{27.17}$$

where:

E_{COD} = efficiency of the UASB reactor in terms of COD removal (%)
t = hydraulic detention time (hour)
0.68 = empirical constant
0.35 = empirical constant

$$E_{BOD} = 100 \times (1 - 0.70 \times t^{-0.50}) \tag{27.18}$$

where:

E_{BOD} = efficiency of the UASB reactor in terms of BOD removal (%)
t = hydraulic detention time (hour)
0.70 = empirical constant
0.50 = empirical constant

Estimation of the COD and BOD concentrations in the final effluent

From the efficiency expected for the system, the COD and BOD concentration in the final effluent can be estimated as follows:

$$C_{effl} = S_0 - \frac{E \times S_0}{100} \tag{27.19}$$

where:

C_{effl} = effluent total COD or BOD concentration (mg/L)
S_0 = influent total COD or BOD concentration (mg/L)
E = COD or BOD removal efficiency (%)

Estimation of the SS concentration in the final effluent

The concentration of suspended solids in the final effluent from UASB reactors depends on a series of factors, including:

- the concentration and the settling characteristics of the sludge present in the reactor
- the sludge wastage frequency and the height of the sludge bed and blanket in the reactor

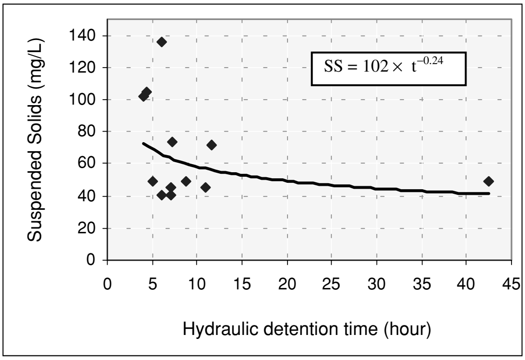


Figure 27.15. SS concentrations in the effluent from UASB reactors treating domestic sewage

- the velocities through the apertures to the sedimentation compartment
- the presence of scum baffles in the sedimentation compartment
- the efficiency of the gas, solids and liquid separator
- the loading rates and the hydraulic detention times in the digestion and sedimentation compartments

In the absence of studies that relate, in a systematised manner, the concentration of solids in the effluent to some of the factors previously mentioned, option was made for the consolidation of the operational results of five reactors taking into account only the hydraulic detention time in the system (see Figure 27.15). The results of solids from the other 11 reactors, which were analysed for COD and BOD removal efficiencies, were not included because they were unusual or not available. It can be observed that the effluent solids concentrations, which varied from 40 to 140 mg/L, were affected by the hydraulic detention time within the system.

From the fitting of the operational results of the five reactors, a curve representing the expected concentration of solids in the effluent was obtained (Equation 27.20). Likewise for COD and BOD, the limitation of this expression is emphasised due to the very reduced number of data used to determine the empirical constants and also to the great deviations existing amongst the data. Besides that, other variables that interfere with the concentration of solids in the effluent are not considered in Equation 27.20.

$$SS = 102 \times t^{-0.24} \qquad (27.20)$$

where:

SS = effluent suspended solids concentration (mg/L)
t = hydraulic detention time (hour)
102 = empirical constant
0.24 = empirical constant

(f) Influent distribution system

To obtain a good performance from UASB reactors, it is essential that the influent substrate is evenly distributed in the lower part of the reactors, to ensure a close contact between the biomass and the substrate. For that purpose and so that the maximum advantage is taken from the biomass present in the reactors, it is essential that preferential pathways (hydraulic short circuits) are avoided through the sludge bed as much as possible. That is particularly important when the process is used in the treatment of low-concentration (such as domestic sewage) and/or low-temperature sewage, once in those situations the biogas production can be very low to allow appropriate mixing within the digestion compartment. Other potential risks for the occurrence of short circuits are:

- short height of the sludge bed
- small number of influent distributors
- occurrence of very concentrated sludge with very high settling velocities

Distribution compartments

An even distribution of the influent is very important in UASB reactors, to ensure a better mixing regime and a reduced occurrence of dead zones on the sludge bed. Thus, the equal division of the influent flow to the several distributing tubes should be done by small compartments (boxes) fed by weirs. Each box feeds a single distribution tube extending to the bottom of the reactor. These compartments, installed in the upper part of the reactor, ensure the uniform distribution of sewage throughout the bottom of the tank, besides enabling the visualisation of occasional increments in the head loss, in each distributor. Once an increased head loss is detected in a distributor, the tube can be easily unblocked by using appropriate rods. Examples of influent distribution structures in UASB reactors are presented in Figures 27.16 and 27.17.

Distribution tubes

Wastewater is routed from the distribution compartments to the bottom of the reactor through distribution tubes. The main requirements for these tubes are as follows:

- the diameter should be large enough to enable a descending sewage velocity **lower than 0.2 m/s**, so that the air bubbles occasionally dragged to inside the tube can go back upwards (opposite the direction of the sewage). The introduction of air bubbles in the reactor should be avoided for the following reasons (van Haandel and Lettinga, 1994): (i) they may cause the aeration of the anaerobic sludge, harming methanogenesis; and (ii) they may cause a potentially explosive mixture with the biogas accumulated close to the three-phase separator. In the case of treatment of low-concentration sewage, this velocity requirement is usually met when the tubes have a 75 mm diameter.

Figure 27.16. Influent distribution structure in a circular reactor (source: Nova Vista WWTP, SAAE Itabira, Minas Gerais, Brazil)

Figure 27.17. Influent distribution structure in a rectangular reactor (source: Ipatinga WWTP, COPASA, Brazil)

- the diameter should be large enough to prevent the solids present in the influent from frequently obstructing the tubes. In this aspect, the excessive presence of solids in the influent can increase the obstruction frequency of the distribution tubes, and the planning of an efficient screening system for the previous removal of solids is essential. Practical experience has shown that distribution tubes with diameters of **75 and 100 mm** meet this requirement.
- the diameter should be small enough to allow a higher flow velocity at its lower end (bottom of the reactor), which favours good mixing and greater contact with the sludge bed. Besides that, a higher velocity helps avoid the deposition of inert solids close to the discharge point of the tube. This requirement is somehow incompatible with the previous ones, once a reduced diameter of the tube hinders the upward movement and the release of air bubbles, besides increasing their possibilities of blocking. A solution that can be adopted is the reduction of the tubing section just close to its lower end, thus keeping an area large enough to avoid blockage. In the case of treatment of domestic sewage, practical experience has shown that *nozzles* with a diameter of approximately **40 to 50 mm** can be used with the purpose of increasing the velocity in the piping exit. For these diameters, the exit velocities are usually higher than 0.40 m/s, which is enough to avoid the deposition of sand close to the ends of the tubes. As an alternative to the nozzles, apertures (windows) can be made on the side

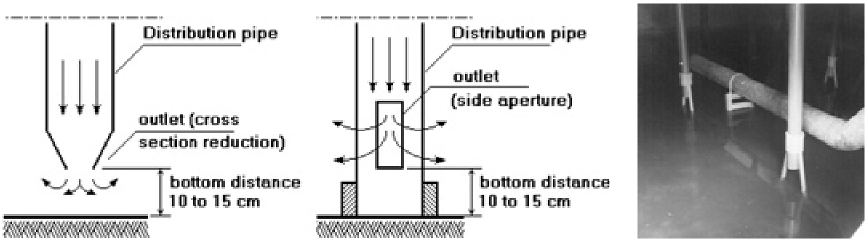


Figure 27.18. Examples of distribution tube ends

ends of the distribution tubes. In this case, two openings with a **25 mm × 40 mm** cross section can be used, creating an area corresponding to a 50 mm nozzle. These devices are illustrated in Figure 27.18.

The lower ends of the distribution tubes should be installed at pre-established points, according to the influence area defined in the design. The maintenance of a fixed position in relation to the bottom of the reactor is important.

Number of distribution tubes

As previously mentioned, the correct distribution of the incoming sewage is one of the most important aspects for the correct operation of the reactor, to ensure an effective contact with the biomass present in the reactor. The number of distribution tubes is determined according to the area of the cross section of the reactor and the influence area adopted for each distributor, as follows:

$$N_d = \frac{A}{A_d} \tag{27.21}$$

where:
N_d = number of distribution tubes
A = area of the cross section of the reactor (m^2)
A_d = influence area of each distributor (m^2)

Preliminary guidelines are presented in Table 27.5 for the influence area of flow distributors in UASB reactors, as a function of the type of sludge and organic loads applied to the system.

In the case of reactors treating domestic sewage, a flocculent sludge is usually developed in the system, with medium to high concentration characteristics. The organic loads applied to the system generally amount from 1.0 to 3.0 kgCOD/m^3·d. In these situations, and according to the guidelines presented in Table 27.5, the influence area of each distributor should be from **1.5 to 3.0 m^2**.

According to a survey done by van Haandel and Lettinga (1994), influence areas of distributors ranging from 1.0 to 4.0 m^2 have been used, as presented in Table 27.6.

Table 27.5. Preliminary guidelines for the influence area of flow distributors in UASB reactors

Sludge type	Organic load applied (kgCOD/m^3·d)	Influence area of each distributor (m^2)
Dense and flocculent (concentration >40 kgTSS/m^3)	<1.0 1.0 to 2.0 >2.0	0.5 to 1.0 1.0 to 2.0 2.0 to 3.0
Relatively dense and flocculent (concentration 20 to 40 kgTSS/m^3)	<1.0 to 2.0 >3.0	1.0 to 2.0 2.0 to 5.0
Granular	<2.0 2.0 to 4.0 >4.0	0.5 to 1.0 0.5 to 2.0 >2.0

Source: Lettinga and Hulshoff Pol (1995)

Table 27.6. Influence areas of flow distributors in UASB reactors treating domestic sewage

System	Influence area of each distributor (m^2)
Itabira (Minas Gerais, Brazil)	2.3 to 3.0
Pedregal (Paraíba, Brazil)	2.0 to 4.0
São Paulo (Cetesb, Brazil)	2.0
Bucaramanga (Colombia)	2.9
Cali (Colombia)	1.0 to 4.0
Kampur (India)	3.7

Source: Adapted from van Haandel and Lettinga (1994)

However, there have been designs that consider an influence area larger than 4 to 5 m^2 for each distribution tube. In these cases, the mixing regime can be affected during the operation of the reactor, harming the contact between biomass and substrate and favouring the creation of dead zones on the sludge bed. Consequently, the efficiency expected for the process may not be reached.

In the particular case of trunk-conical reactors, the influence area of the distribution tubes is not uniform over the height of the digestion compartment, once the cross section of the reactor increases with its height. In these cases, the calculations should consider the cross section close to the deepest part of the reactor (where the sludge bed, more concentrated, is located), that is, close to the first metre of depth of the reactor, to ensure an influence area suitable for the flow distributors.

Considering the low cost of the distribution tubes and the substantial benefits resulting from a correct distribution system, it is recommended that the influence areas of each distributor range from **2.0 to 3.0 m^2** for the treatment of domestic sewage with typical COD concentrations (400 to 600 mg/L).

(g) Three-phase separator

The gas, solids and liquid separator (three-phase separator) is an essential device that needs to be installed in the upper part of the reactor. The main objective of this separator is to maintain the anaerobic sludge inside the reactor, allowing the system to be operated with high solids retention times (high sludge age). This is initially achieved by separating the gas contained in the liquid mixture, enabling, as a consequence, the maintenance of optimal settling conditions in the settling compartment. Once the gas is effectively removed, the sludge can be separated from the liquid in the settling compartment, and then returned to the digestion compartment.

Separation of gases

The design of the gas, solids and liquid separating device (three-phase separator) depends on the characteristics of the wastewater, the type of sludge present in the reactor, the organic load applied, the expected biogas production and the dimensions of the reactor. Aiming at avoiding sludge flotation and the consequent biomass loss from the reactor, the dimensions of the separator should be such that they allow the formation of a liquid–gas interface inside the gas collector sufficient to allow the easy release of the gas entrapped in the sludge. The biogas release rate should be high enough to overcome a possible scum layer, but low enough to quickly release the gas from the sludge, not allowing the sludge to be dragged and consequently accumulated in the gas exit piping. Souza (1986) recommends minimum release rates of 1.0 m^3gas/m^2·hour and maximum rates from 3.0 to 5.0 m^3gas/m^2·hour. The biogas release rate is established by the following equation:

$$K_g = \frac{Q_g}{A_i} \tag{27.22}$$

where:
K_g = biogas release rate (m^3/m^2·hour)
Q_g = expected biogas production (m^3/hour)
A_i = area of the liquid–gas interface (m^2)

Evaluation of the biogas production

The biogas production can be evaluated from the estimated influent COD load to the reactor that is converted into methane gas, according to Chapter 24. In a simplified manner, the portion of COD converted into methane gas can be determined as follows:

$$COD_{CH_4} = Q \times (S_0 - S) - Y_{obs} \times Q \times S_0 \tag{27.23}$$

where:
COD_{CH_4} = COD load converted into methane ($kgCOD_{CH_4}/d$)
Q = average influent flow (m^3/d)
S_0 = influent COD concentration ($kgCOD/m^3$)
S = effluent COD concentration ($kgCOD/m^3$)
Y_{obs} = coefficient of solids production in the system, in terms of COD (0.11 to 0.23 $kgCOD_{sludge}/kgCOD_{appl}$).

The methane *mass* ($kgCOD_{CH_4}/d$) can be converted into *volumetric* production (m^3CH_4/d) by using the following equations:

$$Q_{CH_4} = \frac{COD_{CH_4}}{K(t)} \tag{27.24}$$

where:

Q_{CH_4} = volumetric methane production (m^3/d)
$K(t)$ = correction factor for the operational temperature of the reactor ($kgCOD/m^3$)

$$K(t) = \frac{P \times K_{COD}}{R \times (273 + T)} \tag{27.25}$$

where:

P = atmospheric pressure (1 atm)
K_{COD} = COD corresponding to one mole of CH_4 (64 gCOD/moL)
R = gas constant (0.08206 atm·L/mole·K)
T = operational temperature of the reactor (°C)

Once the theoretical methane production is obtained, the total biogas production can be estimated from the expected methane content. For the treatment of domestic sewage, the methane fraction in the biogas usually ranges from 70 to 80%.

Separation of solids

After the separation of the gases, the liquid and the solid particles that leave the sludge blanket have access to the sedimentation compartment. Ideal conditions for sedimentation of the solid particles occur in this compartment, due to the low upflow velocities and the absence of gas bubbles. The return of the sludge retained in the sedimentation compartment to the digestion compartment does not require any special measure, as long as the following basic guidelines are met:

- installation of deflectors, located immediately below the apertures to the sedimentation compartment, to enable the separation of the biogas, and allow only liquid and solids to enter the sedimentation compartment
- construction of the sedimentation compartment walls with slopes always higher than 45°. Ideally, slopes equal to or higher than 50° should be adopted
- adoption of depths of the sedimentation compartment ranging from 1.5 to 2.0 m
- adoption of surface loading rates and hydraulic detention times in the sedimentation compartment according to Table 27.7

Table 27.7. Surface loading rates and hydraulic detention times in the sedimentation compartment

Influent flow	Surface loading rate (m/hour)	Hydraulic detention time (hour)
Average flow	0.6 to 0.8	1.5 to 2.0
Maximum flow	<1.2	>1.0
Temporary peak flows(*)	<1.6	>0.6

(*) Peak flow lasting between 2 and 4 hours

Apertures to the sedimentation compartment

The apertures that allow the passage of wastewater to the sedimentation compartment should be designed to allow:

- the separation of the gases before the sewage has access to the sedimentation zone, favouring the sedimentation of the solids in the settler compartment. For that purpose, the design of the apertures should allow an appropriate overlap of the gas deflector, to ensure the correct separation of the gas and liquid phases
- the retention of solids in the digestion compartment, by maintaining velocities in the apertures lower than those recommended in Table 27.8
- the return of the solids retained in the sedimentation compartment to the digestion compartment. This return should occur when appropriate slopes of the walls of the sedimentation compartment and gas deflectors are adopted, and also by maintaining compatible velocities through the apertures

Hydraulic detention time in the sedimentation compartment

The hydraulic detention time recommended in the sedimentation compartment ranges from 1 to 2 hours, as presented in Table 27.7. Verifications made in projects already implemented have indicated that the detention times for average flows are not always within the established range. For reactors fed by pumping stations, the detention times tend to be even more reduced, sometimes reaching 0.5 hour when there are two or more pumps in operation.

Table 27.8. Velocities in the apertures to the sedimentation compartment

Influent flow	Velocity (m/hour)
Average flow	<2.0 to 2.3
Maximum flow	<4.0 to 4.2
Temporary peak flows(*)	<5.5 to 6.0

(*) Peak flows lasting between 2 and 4 hours

Figure 27.19. Effluent collection device (plate with V-notch weirs)
Source: Nova Vista WWTP, Itabira, Minas Gerais, Brazil

In situations in which the velocities through the apertures are high and the detention time in the sedimentation compartment is small, a high loss of solids in the effluent and the eventual failure of the treatment system are expected.

(h) Effluent collection

The effluent is collected from the reactor in its upper part, within the sedimentation compartment. The devices usually used for the collection of effluent are plates with V-notch weirs and submerged perforated tubes.

If a launder with V-notch weirs is used (see Figure 27.19), special care should be taken with their levelling, once small slopes in the launder can represent a significant variation in the flow collected at different points. A scum baffle, submerged at approximately 20 cm, should be included along the launder. Additional care regarding the launders refers to the possibility of gas release, particularly H_2S, in view of the turbulence close to the weirs. In this sense, submerged outlets, with no effluent turbulence, are more suitable.

The alternative of using submerged perforated tubes for the collection of effluent has been shown to be very efficient, mainly in three aspects:

- as they are submerged devices, the maintenance of uniform flows in the holes is favoured, and the levelling requirements of the collecting tubes are less important
- the use of submerged tubes decreases or eliminates the risks of turbulence, as well as of release of gases and bad odours
- the submerged collection does not require the use of scum baffles, once the effluent is removed below the scum layer

One of the disadvantages of the collection system by submerged tubes is the possible accumulation of solids in the holes and inside the piping. As cleaning is not always possible, it is recommended that the tubes are laid with a minimum slope of 1%, so that they may be self-cleaned.

(i) Gas system

The uncontrolled release of biogas into the atmosphere is detrimental, not just for the possible occurrence of bad odours in the neighbourhood, but mainly for the risks inherent to the methane gas, which is combustible. Thus, the biogas produced in the reactor should be collected, measured and, later, either used or burnt. The biogas removal system from the liquid–gas interface inside the reactor consists of:

- collecting piping
- sealed compartment with hydraulic seal and biogas purge
- biogas meter
- biogas reservoir

When the biogas is not used, the gas reservoir is replaced by a security valve and a gas burner, preferably located at a safe distance from the reactor, as illustrated in Figures 27.20 and 27.21.

The biogas flow meter is an important device for the monitoring of the amount of gas produced in the system, being essential for the evaluation of the process efficiency. To avoid damage to the meters, caused by the dragging of condensed liquids into the collecting piping, the average biogas flow velocity should not exceed 3.5 m/s.

Further considerations on the collection, treatment and destination of the gases generated in anaerobic reactors can be found in Campos and Pagliuso (1999), Belli Filho *et al.* (2001), Andreoli *et al.* (2003) and Cassini *et al.* (2003).

(j) Sludge sampling and discharge system

The design of the reactor should comprise a group of valves and piping that allows both sampling and discharge of the solids present in the reactor.

Sludge sampling system

The sampling system usually consists of a series of valves installed along the height of the digestion compartment, to enable the monitoring of the growth and quality

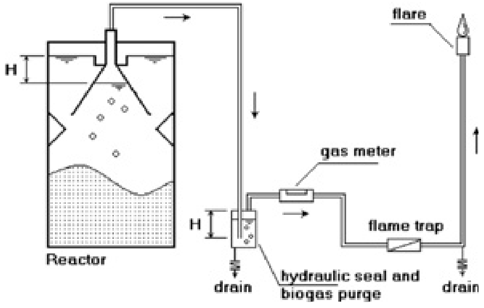


Figure 27.20. Diagram of a gas system in UASB reactors

Figure 27.21. View of a hydric seal and gas burner (source: Ipatinga WWTP, Minas Gerais, Brazil)

of the biomass in the reactor. One of the most important operational routines in the treatment system is the evaluation of the amount and activity of the biomass present in the reactor, by means of two basic mechanisms:

- determination of the solids profile and mass of microorganisms present in the system, as exemplified in Chapter 25 (Example 25.1)
- evaluation of the specific methanogenic activity of the biomass, as exemplified in Chapter 25 (Example 25.2)

The continuous monitoring of the biomass present in the reactor will allow the operation personnel to have more control actions over the solids in the system, such as:

- identification of the height and concentration of the sludge bed in the reactor, allowing the establishment of discharge strategies (discharge amount and frequency)
- determination of the ideal sludge discharge points, according to the results of the specific methanogenic activity tests and the characteristics of the sludge

Thus, to enable the removal and characterisation of the biomass at different levels of the digestion compartment, the installation of valves is recommended, from the base of the reactor, with the following characteristics:

- spacing: 50 cm
- diameter: 1½ to 2 inches
- type: ball valve

Sludge withdrawal system

The sludge discharge system is intended for the periodical removal of the excess sludge produced in the reactor, also allowing the removal of inert material that may accumulate at the bottom of the reactor. At least two sludge withdrawal points should be planned, one close to the bottom of the reactor and another approximately 1.0 to 1.5 m above the bottom (depending on the height of the digestion compartment), to allow a higher operational flexibility. A minimum diameter of 100 mm is recommended for the sludge discharge piping. Figure 27.22 illustrates a sludge sampling and withdrawal system in UASB reactors.

27.2.5 Sludge production and treatment

The solids accumulation rate depends essentially on the type of effluent being treated and is greater when the wastewater has a higher concentration of suspended solids, especially non-biodegradable solids.

In the case of treating soluble effluents, the production of excess sludge is very low and generally few problems are found in the handling, storage and disposal of the sludge. As a result of the low production and the high concentrations of

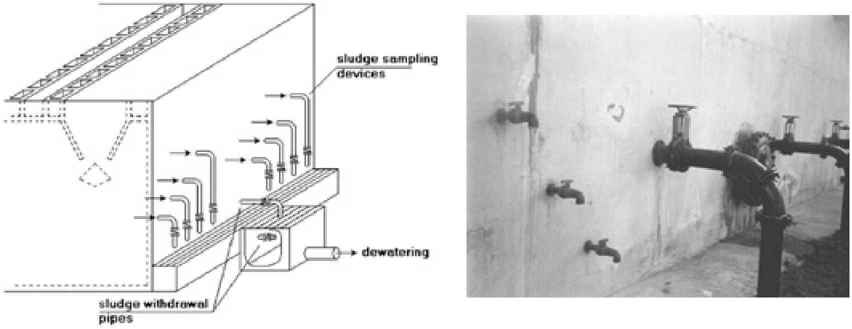


Figure 27.22. Example of sludge sampling and discharge devices in UASB reactors

sludge in the reactor, the discharged volumes are also very small in comparison with aerobic processes.

Some important characteristics of anaerobic excess sludges from UASB reactors are as follows:

- high level of stability due to the high solids retention time in the treatment system, which allows the sludge to be directed to dewatering units without any prior treatment stage
- high concentration, usually in the order of 3 to 5%, allowing the discharge of smaller volumes of sludge
- good dewaterability
- possibility of the use of the dewatered sludge as a soil conditioner in agriculture, as long as care is taken regarding the presence of pathogens

(a) Sludge production

The estimation of the mass production of sludge in UASB reactors can be done through the following equation:

$$\boxed{P_s = Y \times COD_{app}} \tag{27.26}$$

where:

P_s = production of solids in the system (kgTSS/d)
Y = yield or solids production coefficient (kgTSS/kgCOD$_{app}$)
COD_{app} = COD load applied to the system (kgCOD/d)

The values of Y reported for the anaerobic treatment of domestic sewage are in the order of **0.10 to 0.20 kgTSS/kgCOD$_{\mathbf{app}}$**.

The estimation of the volumetric sludge production can be done by the following equation:

$$\boxed{V_s = \frac{P_s}{\gamma \times (C_s/100)}} \tag{27.27}$$

where:

V_s = volumetric sludge production (m^3/d)
γ = sludge density (usually in the order of 1,020 to 1,040 kg/m^3)
C_s = solids concentration in the sludge (%)

(b) Sludge dewatering

Sludge drying beds have been the alternative most commonly used for the dewatering of sludges from UASB reactors treating domestic sewage. This is due to the small volumes of sludge that are discharged from the system, as a result of the low yield and high concentration of the sludge in the reactors.

According to van Haandel and Lettinga (1984), the following procedures are necessary in the design of drying beds:

- estimate the daily mass of sludge that should be discharged from the reactor (Equation 27.26)
- define the productivity of the drying bed, to be expressed in terms of mass of solids that can be applied daily per unit area of the bed ($kgTSS/m^2{\cdot}d$)
- adopt a maximum value of the fraction of the mass of sludge that can be discharged in one batch. Usually this fraction is in the order of 20 to 25% of the mass of sludge present in the reactor
- determine the sludge discharge frequency
- determine the number of beds

Detailed procedures for the design of drying beds are presented in Chapter 50.

(c) Wastewater pre-treatment

According to what was presented in Chapter 26, high-rate anaerobic reactors are designed with much smaller volumes in comparison with those of conventional anaerobic systems. For this reason, the entrance of non-biodegradable solids in the system is highly detrimental to the treatment process. The accumulation of this material in the reactor leads to the formation of dead zones and short circuits, significantly reducing the volume of biomass in the system and the efficiency of the treatment process.

Hence, the treatment of domestic sewage through high-rate anaerobic reactors is only possible if the flowsheet of the treatment plant incorporates preliminary treatment units (screens and grit chambers) aiming at the removal of coarse solids and inorganic settleable solids present in the sewage. In more recent designs a concern with the incorporation of devices that guarantee a greater efficiency in the removal of fine solids (that pass through conventional screens) and fats has been observed, aiming at guaranteeing better operational conditions in the reactor.

For example, the provision of sieves (static or mechanised) with openings in the order of 2 to 6 mm minimise the entrance of solids into the reactor, improving the functioning of the influent distribution device, due to the reduction/elimination of obstructions in the feeding tubes.

Regarding the provision of devices for the removal of fats, this is meant to reduce the scum formation problems in the reactor (as much in the interior of the gas collector as in the settler compartment). Scum, in fact, has frequently led to many operational problems due to the inherent difficulties in its removal.

27.2.6 Summary of the design criteria and parameters

A summary of the main criteria and parameters that orientate the design of UASB reactors for the treatment of domestic sewage, according to the previous items, is presented in Tables 27.9 and 27.10.

27.2.7 Construction aspects

(a) Reactor height

The height to be adopted for the UASB reactors is dependent on the following main factors: (i) type of sludge present in the reactor; (ii) organic loads applied; and/or (iii) volumetric hydraulic loads, that define the upflow velocities imposed to the system. In the case of domestic sewage treatment in reactors that predominantly develop flocculent-type sludge, the upflow velocities imposed to the system lead to reactors with useful heights between 4.0 and 5.0 m, distributed in following way:

- height of settler compartment: 1.5 to 2.0 m
- height of digestion compartment: 2.5 to 3.5 m

Table 27.9. Summary of the main hydraulic criteria for the design of UASB reactors treating domestic sewage

	Range of values, as a function of flow		
Criterion/parameter	for Q_{ave}	for Q_{max}	for $Q_{peak}^{(*)}$
Hydraulic volumetric load ($m^3/m^3 \cdot d$)	<4.0	<6.0	<7.0
Hydraulic detention time (hour)**	6 to 9	4 to 6	>3.5 to 4
Upflow velocity (m/hour)	0.5 to 0.7	<0.9 to 1.1	<1.5
Velocity in the apertures to the settler (m/hour)	<2.0 to 2.3	<4.0 to 4.2	<5.5 to 6.0
Surface loading rate in the settler (m/hour)	0.6 to 0.8	<1.2	<1.6
Hydraulic detention time in the settler (hour)	1.5 to 2.0	>1.0	>0.6

(*) Flow peaks with duration between 2 and 4 hours
(**) Sewage temperature in the range of 20 to 26 °C

Table 27.10. Other design criteria for UASB reactors treating domestic sewage

Criterion/parameter	Range of values
Influent distribution	–
Diameter of the influent distribution tube (mm)	75 to 100
Diameter of the distribution tube exit mouth (mm)	40 to 50
Distance between the top of the distribution tube and the water level in the settler (m)	0.20 to 0.30
Distance between the exit mouth and the bottom of the reactor (m)	0.10 to 0.15
Influence area of each distribution tube (m^2)	2.0 to 3.0
Biogas collector	–
Minimum biogas release rate ($m^3/m^2 \cdot$hour)	1.0
Maximum biogas release rate ($m^3/m^2 \cdot$hour)	3.0 to 5.0
Methane concentration in the biogas (%)	70 to 80
Settler compartment	–
Overlap of the gas deflectors in relation to the opening for the settler compartment (m)	0.10 to 0.15
Minimum slope of the settler walls (°)	45
Optimum slope of the settler walls (°)	50 to 60
Depth of the settler compartment (m)	1.5 to 2.0
Effluent collector	–
Submergence of the scum baffle or the perforated collection tube (m)	0.20 to 0.30
Number of triangular weirs (units/m^2 of the reactor)	1 to 2
Production and sampling of the sludge	–
Solids production yield ($kgTSS/kgCOD_{applied}$)	0.10 to 0.20
Solids production yield, in terms of COD ($kgCOD_{sludge}/kgCOD_{applied}$)	0.11 to 0.23
Expected solids concentration in the excess sludge (%)	2 to 5
Sludge density (kg/m^3)	1020 to 1040
Diameter of the sludge discharge pipes (mm)	100 to 150
Diameter of the sludge sampling pipes (mm)	25 to 50

(b) Construction materials

Considering that the anaerobic degradation of certain compounds can lead to the formation of highly aggressive by-products, the materials used in the construction of anaerobic reactors should be resistant to corrosion.

For construction and cost reasons, concrete and steel have been the materials most commonly used in UASB reactors usually with an internal coating protection in an epoxy base. However, the solids and gas separator located in the upper part of the reactor that is more exposed to corrosion should be fabricated of a more resistant material or more heavily coated. Concrete is the material most frequently used, but experiences have not always been satisfactory due to problems of leaking gases, corrosion and that of constructing a bulky and heavy structure. Non-corrosive and less bulky materials such as PVC, fibreglass and stainless steel are more attractive options.

(c) Corrosion protection

Resistance to corrosion can be intrinsic to the material (e.g. PVC, fibreglass, stainless steel) or can be part of it through special additives or coating/linings (e.g.

Table 27.11. Concrete coatings (comparative characteristics)

Coating	Advantage	Disadvantage
Chlorinated rubber	• Lower cost	• Lower resistance to volatile fatty acids
Bituminous epoxy	• Good resistance to volatile fatty acids • Can be applied with a greater thickness and a lower number of layers • Presents lower permeability	• Much higher cost

Source: Chernicharo *et al.* (1999)

concrete, steel). In the case of steel reactors, the care needs to be greater to avoid corrosion, including the use of special steels and the rigorous control of the coatings employed.

In the case of reinforced concrete reactors, the concern with the protection of the structure should be prior to the construction of the unit, such as in the provision of a concrete with sufficient chemical resistance. In this sense, some factors should be considered with the aim of obtaining lower rates of absorption and permeability:

- use of a concrete with a low water–cement ratio
- rigorous vibration of the concrete
- adequate curing process
- selection of an appropriate cement (Portland Pozzolanic)

In addition, the corrosion effects can be improved or inhibited through the application of acid resistant coatings. A thorough study in relation to the advantages of the different types of coatings was developed by Fortunato *et al.* (1998), that recommended possible coating solutions such as the painting of the reactor with chlorinated rubber or bituminous epoxy. These materials function as chemical barriers for the concrete surfaces exposed to highly aggressive environments. Some comparative characteristics of these types of coatings are presented in Table 27.11.

Example 27.2

Design a UASB reactor, based on the following design elements:

Data:

- Population: P = 20,000 inhabitants
- Average influent flow: $Q_{av} = 3{,}000\ m^3/d$ ($125\ m^3$/hour)
- Maximum hourly influent flow: $Q_{max\text{-}h} = 5{,}400\ m^3/d$ ($225\ m^3$/hour)
- Average influent COD $(S_o) = 600$ mg/L
- Average influent BOD $(S_o) = 350$ mg/L
- Sewage temperature: T = 23 °C (average of the coldest month)
- Solids yield coefficient: $Y = 0.18\ kgTSS/kgCOD_{app}$
- Solids yield coefficient, in terms of COD: $Y_{obs} = 0.21\ kgCOD_{sludge}/kgCOD_{app}$

Example 27.2 **(*Continued*)**

- Expected concentration of the discharge sludge: C = 4%
- Sludge density: γ =1,020 kg/m^3

Solution:

(a) Calculation of the average influent COD load (L_o)

$$L_o = S_o \times Q_{av} = 0.600\ \text{kg/m}^3 \times 3{,}000\ \text{m}^3/\text{d} = 1{,}800\ \text{kgCOD/d}$$

(b) Adopt a value for the hydraulic detention time (t)

t = 8.0 hours (according to Table 27.9)

(c) Determine the total volume of the reactor (V)

$$V = Q_{av} \times t = 125\ \text{m}^3/\text{hour} \times 8\ \text{hours} = 1{,}000\ \text{m}^3$$

(d) Adopt the number of reactor modules (N)

$$N = 2$$

Although there is no limitation to the volume of the reactor, it is recommended that the reactor volume does not exceed 1,500 m^3, due to constructive and operational limitations. In the case of small systems for the treatment of domestic sewage, the adoption of modular reactors presents numerous advantages. In these cases, it has been usual to use modules with volumes no greater than 400 to 500 m^3.

(e) Volume of each module (V_u)

$$V_u = V/N = 1{,}000\ \text{m}^3/2 = 500\ \text{m}^3$$

(f) Adopt a value for the height of the reactor (H)

$$H = 4.5\ \text{m}$$

(g) Determine the area of each module (A)

$$A = V_u/H = 500\ \text{m}^3/4.5\ \text{m} = 111.1\ \text{m}^2$$

Adopt rectangular reactors of 7.45 m × 15.00 m ($A = 111.8\ \text{m}^2$)

(h) Verification of the corrected area, volume and detention time

Corrected total area: $A_t = N \times A = 2 \times 111.8\ \text{m}^2 = 223.6\ \text{m}^2$
Corrected total volume: $V_t = A_t \times H = 223.6\ \text{m}^2 \times 4.5\ \text{m} = 1{,}006\ \text{m}^3$
Corrected hydraulic detention time: $t = V_t/Q_{av} = 1{,}006\ \text{m}^3/(125\ \text{m}^3/\text{hour}) =$ 8.0 hours

(i) Verification of the loads applied

Volumetric hydraulic load (Equation 27.8): $VHL = Q/V = (3{,}000\ \text{m}^3/\text{d})/1{,}006\ \text{m}^3 = 2.98\ \text{m}^3/\text{m}^3\cdot\text{d}$

Example 27.2 (Continued)

Volumetric organic load (Equation 27.12): $L_v = Q_{av} \times S_o/V = (3{,}000\ m^3/d \times 0.600\ kgCOD/m^3)/1006\ m^3 = 1.79\ kgCOD/m^3{\cdot}d$

(j) Verification of the upflow velocities (according to Equation 27.15)

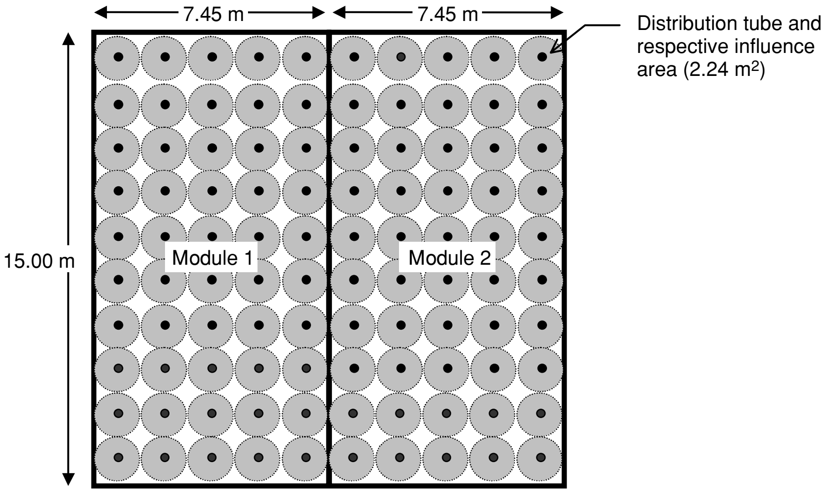
- for Q_{av}: $v = Q_{av}/A = (125\ m^3/hour)/223.6\ m^2 = 0.56\ m/hour$
- for $Q_{max\text{-}h}$: $v = (225\ m^3/hour)/223.6\ m^2 = 1.01\ m/hour$

It can be seen that the upflow velocities found are in agreement with the values shown in Table 27.9.

(k) Influent wastewater distribution system

- Number of distribution tubes
  Adopting an influence area of 2.25 m^2 per distribution tube (according to Table 27.5), then the number of tubes can be calculated in accordance with Equation 27.21:
  $N_d = A/A_d = 223\ m^2/2.25\ m^2 = 99$ distributors. Due to the necessary symmetry of the reactor, adopt 100 distributors, as follows:
- along the length of each module (15.00 m): 10 tubes
- along the width of each module (7.45 m): 5 tubes

Thus, each module will have 50 (10 × 5) distribution tubes, each with an influence area equivalent to: $A_d = 223.6\ m^2/100 = 2.24\ m^2$.



Schematics of the influence area of each distribution tube (bottom of the reactor)

Example 27.2 (Continued)

(l) Estimation of the COD removal efficiency of the system

According to Equation 27.17:

$$E_{COD} = 100 \times (1 - 0.68 \times t^{-0.35}) = 100 \times (1 - 0.68 \times 8.0^{-0.35})$$
$$E_{COD} = 67\%$$

(m) Estimation of the BOD removal efficiency of the system

According to Equation 27.18:

$$E_{BOD} = 100 \times (1 - 0.70 \times t^{-0.50}) = 100 \times (1 - 0.70 \times 8.0^{-0.50})$$
$$E_{BOD} = 75\%$$

(n) Estimation of the COD and BOD concentrations in the final effluent

According to Equation 27.19:

$$C_{effl} = S_o - (E \times S_o)/100$$
$$C\ effl_{COD} = 600 - (67 \times 600)/100 = 198\ \text{mgCOD/L}$$
$$C\ effl_{BOD} = 350 - (75 \times 350)/100 = 88\ \text{mgBOD/L}$$

(o) Evaluation of the methane production

The theoretical production of methane can be estimated from Equations 27.23, 27.24 and 27.25:

$$COD_{CH_4} = Q_{av} \times [(S_0 - C_{effl}) - Y_{obs} \times S_o)]$$
$$COD_{CH_4} = 3{,}000\ \text{m}^3/\text{d} \times [(0.600 - 0.198\ \text{kgCOD/m}^3) - (0.21\ \text{kgCOD}_{sludge}/\text{kgCOD}_{app} \times 0.600\ \text{kgCOD/m}^3)]$$
$$COD_{CH_4} = 828\ \text{kgCOD/d}$$

$$K(t) = (P \times K_{COD})/[R \times (273 + T)]$$
$$K(t) = (1\ \text{atm} \times 64\ \text{gCOD/moL})/[(0.08206\ \text{atm·l/mol·K} \times (273 + 23\ ^\circ\text{C})]$$
$$K(t) = 2.63\ \text{kgCOD/m}^3$$

$$Q_{CH_4} = COD_{CH_4}/K(t)$$
$$Q_{CH_4} = (828\ \text{kgCOD/d})/(2.63\ \text{kgCOD/m}^3)$$
$$Q_{CH_4} = 314\ \text{m}^3/\text{d}$$

(p) Evaluation of the biogas production

The evaluation of the biogas is done from the estimation of the percentage of methane in the biogas. Adopting a methane content of 75%:

$$Q_g = Q_{CH_4}/0.75 = (314\ \text{m}^3/\text{d})/0.75 = 419\ \text{m}^3/\text{d}$$

Example 27.2 (Continued)

(q) Sizing of the gas collectors

Number of gas collectors: 10 (5 in each module)
Length of each collector: $L_g = 7.45$ m (along the width of the reactor)
Total length of the gas collector: $L_t = 10 \times 7.45$ m $= 74.5$ m
Width of the upper part of the gas collector: $W_g = 0.25$ m (adopted)
Total area of the gas collectors (in its upper part): $A_g = L_t \times W_g = 74.5$ m $\times$ 0.25 m $= 18.6$ m^2
Verification of the biogas release rate in the gas collectors (K_g), according t Equation 27.22:

$$K_g = Q_g/A_i = (419/24\ \text{m}^3/\text{hour})/18.6\ \text{m}^2 = 0.94\ \text{m}^3/\text{m}^2{\cdot}\text{hour}$$

Therefore, each biogas collector will have the following dimensions:

- Length: 7.45 m, Width: 0.25 m

(r) Sizing of the apertures to the settler compartment

Adopting 5 three-phase separators in each reactor, as illustrated below, then:
Number of simple apertures: 4 (2 in each module, alongside the walls)
Number of double apertures 8 (4 in each module, between the tri-phas separators)
Equivalent number of simple apertures: $4 + 8 \times 2 = 20$
Length of each aperture: $L_a = 7.45$ m (along the width of the reactor)
Equivalent length of simple openings: $L_t = 20 \times 7.45$ m $= 149.0$ m
Width of each aperture: $W_a = 0.40$ m (adopted)
Total area of the apertures: $A_t = L_t \times W_a = 149.0$ m $\times$ 0.40 m $= 59.6$ m^2
Verification of the velocities through the apertures (v_a):

- for Q_{av}: $v_a = Q_{av}/A_t = (125\ \text{m}^3/\text{hour})/59.6\ \text{m}^2 = 2.1$ m/hour
- for $Q_{max\text{-}h}$: $v_a = (225\ \text{m}^3/\text{hour})/59.6\ \text{m}^2 = 3.79$ m/hour

It can be seen that the velocities found are in agreement with the values in Table 27.9.

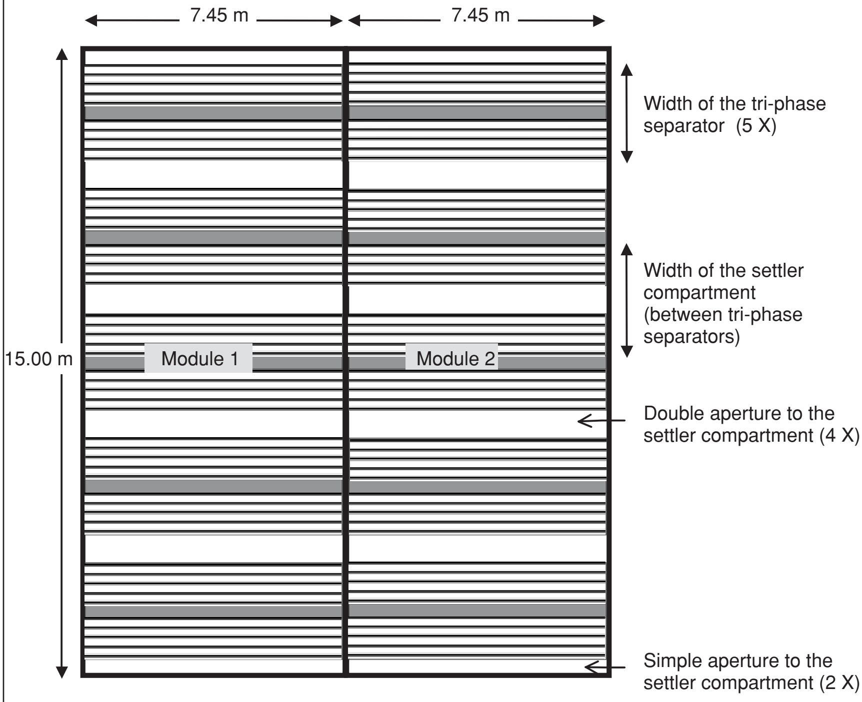
Therefore, each aperture to the settler compartment will have the following dimensions:

- Simple aperture: Length = 7.45 m, Width = 0.40 m
- Double aperture: Length = 7.45 m, Width = 0.80 m

(s) Sizing of the settler compartment

Number of settler compartments: 10 (5 in each reactor)
Length of each settler: $L_s = 7.45$ m (along the width of the reactor)

Example 27.2 **(*Continued*)**



Schematics of the tri-phase separator distribution (top of the reactor)

Total length of the settlers: $L_t = 10 \times 7.45\ \text{m} = 74.5\ \text{m}$
Width of each gas collector: $W_g = 0.30\ \text{m}$ (0.25 m + 0.05 wall thickness)
Width of each settler compartment: $W_s = 15.00\ \text{m} / 5 = 3.00\ \text{m}$
Effective width of each settler: $W_e = 3.00\ \text{m} - 0.30\ \text{m} = 2.70\ \text{m}$
Total area of the settlers: $A_s = L_t \times W_e = 74.5\ \text{m} \times 2.70\ \text{m} = 201.2\ \text{m}^2$
Verification of the surface loading rates of the settlers (v_s)

- for Q_{av}: $v_s = Q_{av}/A_s = (125\ \text{m}^3/\text{hour})/201.2\ \text{m}^2 = 0.62\ \text{m/hour}$
- for $Q_{max\text{-}h}$: $v_s = (225\ \text{m}^3/\text{hour})/201.2\ \text{m}^2 = 1.12\ \text{m/hour}$

It can be seen that the surface loading rates are in agreement with the values in Table 27.9. Therefore, each settler compartment will have the following dimensions in plan:

- Length: 7.45 m, Width 2.70 m

***Example 27.2* (*Continued*)**

To determine the volume of the settler compartment, it is necessary to produce a general arrangement of the three-phase separator, taking into consideration the following aspects:

- height of the upper part of the settler compartment (vertical walls)
- height of the bottom part of the settler compartment (inclined walls)
- detention time for the settler compartment, in agreement with Table 27.9

(t) Evaluation of the sludge production

The expected sludge production in the treatment system can be estimated from Equations 27.26 and 27.27

$$P_s = Y \times COD_{app} = 0.18\ \text{kgTSS/kgCOD}_{app} \times 1{,}800\ \text{kgCOD/d}$$
$$P_s = 324\ \text{kgTSS/d}$$
$$V_s = P_s/(\gamma \times C_s) = (324\ \text{kgTSS/d})/(1{,}020\ \text{kg/m}^3 \times 0.04)$$
$$V_s = 7.9\ \text{m}^3\text{/d}$$

28

Operational control of anaerobic reactors

28.1 IMPORTANCE OF OPERATIONAL CONTROL

28.1.1 Preliminaries

The benefits of any wastewater treatment system, should it be either aerobic or anaerobic, will only be reached in an optimised manner if a logical sequence of actions is followed, as illustrated in Figure 28.1.

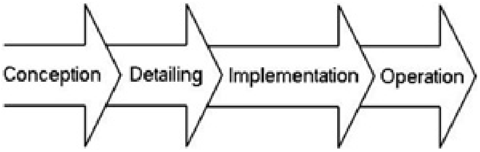


Figure 28.1. Flowchart of actions for a wastewater treatment system

It is assumed from the above flowchart that the main objectives of any wastewater treatment system, that is, protection of the population's health and preservation of the environment, will only be achieved if the treatment plant is well conceived, well detailed, well implemented and also correctly operated. It is in relation to this last action that the operational control of the treatment plants becomes very important. Some aspects that demonstrate the relevance of the operational control

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are described in the following items, as highlighted in the original work developed by Chernicharo *et al.* (1999).

28.1.2 Verification of operational parameters

In countries with little tradition in wastewater treatment, the new treatment plants are usually designed based on parameters not always reliable and, many times, imported from foreign references. In general, these parameters can be verified during the operational phase of the system, taking into consideration the values originally assumed during the design phase. The various parameters of importance that should be verified during the operational phase of the system include:

- influent flowrates
- physical–chemical and microbiological characteristics of the influent wastewater
- efficiency and operational problems of the preliminary treatment units
- production and characteristics of the material retained in the screens and in the grit chamber
- efficiency and operational problems of the anaerobic reactor
- amount and characteristics of the biogas produced in the anaerobic reactor
- amount and characteristics of the sludge produced in the anaerobic reactor, etc.

These operational parameters, amongst others, can be properly evaluated based on the implementation of a monitoring programme, and later compared with the values originally assumed in the design, allowing:

- a revision or adaptation of the operational strategies initially planned for the treatment system
- a better based decision making, taking into consideration possible expansion plans for the system. Should the flows and organic loads be inferior to the design estimates, the project horizon can be increased and the investment with the expansion of the system in the subsequent years reduced. Conversely, flows and organic loads higher than those planned at the design stage will indicate that the project horizon should be reduced and that financial resources should be made available for the expansion of the system

28.1.3 Optimisation of the operational conditions

Another important aspect concerning the operational control of the treatment system is that it can lead to optimised operational conditions, aiming at reducing costs and meeting the discharge standards established by the environmental legislation. In this sense, some operational aspects should be emphasised:

- Determination of the best wastage and dewatering routine for the excess sludge. In the case of treatment plants that dewater the sludge on drying

beds, wastage frequencies and solids loads to the beds leading to shorter drying cycles can be evaluated. Hence, an optimised sludge wastage and dewatering will directly imply a reduced volume of dry sludge to be transported to final disposal. An adequate wastage frequency will reflect directly on a smaller loss of solids in the final effluent, resulting in a better effluent quality in terms of suspended solids and particulate COD and BOD, with a direct impact on the compliance with the environmental legislation.
- Definition of the best practices and routines for operation and cleaning of the screening and grit removal units, aiming at optimising the efficiency of these preliminary treatment units. The removal of coarse materials and grit present in the influent wastewater can be maximised, preventing them from being introduced into the anaerobic reactor. These materials are highly harmful to the operation of the biological reactor, causing not only the obstruction of the sewage distribution piping, but also their accumulation inside the reactor, which causes the decrease of its useful volume and, consequently, a reduction in the efficiency of the system.
- Identification of bad odour points, aiming at providing a greater safety and environmental comfort to the operators and people who live near the treatment plant. In this sense, the effective follow-up of the units potentially subject to the release of foul gases (preliminary treatment, pumping station, anaerobic reactor and drying beds) will allow a greater knowledge of the problematic points, and facilitate the taking of measures and the implementation of adaptations to make odour control possible.

28.1.4 Workers' health and safety

In addition to the aspects previously mentioned, operational control is an important instrument for the identification of practices and routines that can promote the improvement of the workers' health and safety.

Health risks have always been a reason for concern in sewage treatment plants, since both *disability* and *occupational diseases* result in suffering and loss of human resources. Both cause a negative effect on the efficiency of the treatment system, on employees' morale, on public relationships and on costs (WEF, 1996). A good worker's health and safety programme should incorporate three main elements (USEPA, 1988; WEF, 1992):

- *Defined health and safety policy*: it comprises the principles of the whole health and safety programme, providing the workers with the key message of the programme, and making clear that it is supported by the upper management. The support should be visible, that is, the management should support the programme by means of actions and financial resources.
- *Work safety and occupational health committee*: it should be composed of management, supervisors and workers' representatives. Some specific tasks to be performed by the committee are: (i) conduct the health and safety programme; (ii) carry out systematic inspections; (iii) suggest and

provide training; (iv) perform accident investigations; (v) maintain records on the occurrences; and (vi) prepare a health and safety manual.

- *Health and safety training*: the supervisors of the treatment plant should have, above all, their own attitudes and interests regarding health and safety, getting a total knowledge and understanding of the various forms of accident and occupational disease prevention. All new employees should undergo a health and safety programme, as well as all employees should be trained whenever a new equipment or process is added to the treatment plant.

Other details for the establishment of a health and safety programme for wastewater treatment plant operators can be found in WEF (1996).

28.2 OPERATIONAL CONTROL OF THE TREATMENT SYSTEM

28.2.1 Preliminaries

Although the operational simplicity of anaerobic treatment systems is one of its key points, the presence of operation and maintenance personnel is a necessary condition to assure appropriate performance. The three main treatment system control activities are:

- *operation*: refers to the daily or periodic activities necessary to assure a good and stable performance of the treatment system
- *maintenance*: refers to the activities to maintain the structures in the treatment plant in good conditions
- *information*: refers to the communication, preferably in writing, between the different people involved, creating, at the same time, a record of the operation and maintenance of the treatment system

28.2.2 Monitoring of the system

28.2.2.1 Need for system monitoring

The effective operational control of any wastewater treatment system will only be achieved by the implementation of an appropriate monitoring programme, to enable both the verification of the operational parameters and the optimisation of the operational routine.

The monitoring programme should be broad enough to include all the aspects relevant to the operation of the treatment system, without disregarding the local reality and the availability of human resources and material. Therefore, not only the development of physical–chemical and microbiological analyses becomes important, but also the gathering of a series of information on the operation of the system, as covered in the following items.

Usually, the anaerobic treatment systems can be divided into three parts, as presented in the schematic representation of Figure 28.2:

- pre-treatment
- biological treatment, or anaerobic digestion itself
- excess sludge dewatering

The operational activities of the anaerobic treatment systems are related to the different parts of the treatment system, and can be divided into four groups:

- Activities to ensure the appropriate operation of the pre-treatment units, usually consisting of: (i) screen (mechanised or not); (ii) grit chamber (mechanised or not); and (iii) flow measuring device, usually a Parshall flume coupled to the grit chamber
- Activities to evaluate the efficiency of digestion. Usually, anaerobic digestion is applied for the removal of suspended solids and organic matter, besides partially reducing the pathogenic organisms
- Activities to evaluate the operational stability of the digester, that is, to establish if there is any risk of the pH in the anaerobic reactor being reduced to a value lower than the minimum for the optimum methanogenesis ($pH_{min} = 6.5$)
- Activities to determine the amount and quality of the sludge in the reactor and in the excess sludge processing unit. The amount of sludge is important to establish the excess sludge wastage moment. The sludge quality is usually evaluated by specific methanogenic activity (SMA) and sedimentation tests. Regarding the quality of the excess sludge, the stability in which the sludge is wasted from the reactor and the solids fraction (or moisture fraction) in the dewatering unit (drying beds, centrifuges, filters or others) are important

In addition to these four specific groups, there may be others, depending on the intended use of the effluent. For instance, when the effluent is intended to be used (after a complementary treatment) in irrigation, it will be important to monitor the level of the nutrients N and P, although they do not play an important role in the treatment system and their removal is not the purpose of the anaerobic treatment.

In general, the tasks specified in the different groups will be carried out by different people. Thus, the works regarding the pre-treatment system require the frequent presence of personnel to verify whether there are blockages. Usually, the removal of coarse solids and sand collected in the pre-treatment units, as well as of dewatered sludge from the drying beds, will be manual, requiring unskilled labour. On the other hand, sampling of the biological treatment system and the undertaking of analyses to verify treatment efficiency, operational stability and the sludge mass in the reactor require more qualified personnel.

28.2.2.2 Monitoring programme

To facilitate the understanding of the units to be monitored in the system, Figure 28.2 presents a typical flowsheet of a sewage treatment plant consisting of the following units: preliminary treatment, anaerobic reactor and drying bed.

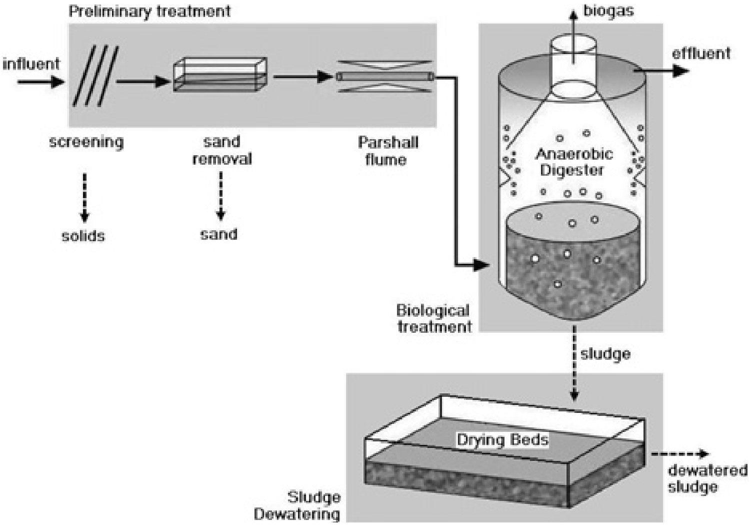


Figure 28.2. Typical flowsheet of an anaerobic wastewater treatment plant with sludge dewatering and preliminary treatment units

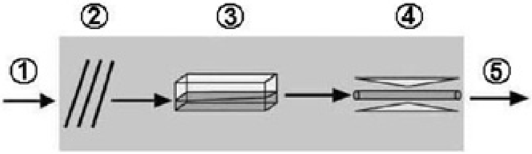
This section focuses only on the operational control of reactors operating close to *steady-state conditions*, that is, the regime in which the system reaches more stable operational conditions, with no significant variations and instabilities over time. Recommendations on the operational control during the start-up period (*transient regime*) of the system are presented in Section 28.3.3.

(a) Monitoring and operation of the preliminary treatment

Good operation of the anaerobic reactor depends fundamentally on the flow and characteristics of the wastewater to be treated and on the correct operation of the preliminary treatment units. An operational routine that allows the screens and grit chambers to be cleaned at a suitable frequency should be established to assure effective removal of the coarse solids and grit present in the wastewater. In the case of domestic sewage, the screen cleaning should be at least daily. Sand should be removed from the chambers once every 1 or 2 weeks, depending on the sand content in the influent wastewater (higher cleaning frequency for, say, 50 L of sand per 1,000 m^3 of influent sewage, and lower cleaning frequency for, say, 25 L of sand per 1,000 m^3 of influent sewage).

Regarding the most important characteristics that affect the anaerobic biodegradability (temperature and pH), these parameters can be easily measured in the influent. The preliminary treatment operation also includes the removal of blockages that may harm the uniform distribution of the influent in the treatment system. In this sense, the concentration of settleable solids is an important parameter.

The following figure and table (Preliminary treatment) identify the main points, parameters and frequency of monitoring at the preliminary treatment stage. The troubleshooting list presented at the end of this chapter identifies some problems that can be found in the daily operation of the preliminary treatment units.



Preliminary treatment

Parameter[1]	Unit	Monitoring points and frequency 1	2	3	4	5
Flow	(L/s)	–	–	–	Daily	–
Screenings volume	(m^3/d)	–	Daily	–	–	–
Sand volume	(m^3/d)	–	–	Daily	–	–
Temperature	°C	–	–	–	–	Daily
pH	–	–	–	–	–	Daily
Settleable solids	(mL/L)	Daily	–	–	–	Daily

(b) Monitoring of the anaerobic reactor

The successful operation of any anaerobic reactor depends on the systemisation and implementation of appropriate operational procedures during the start-up phase and over the operation on a steady-state basis. Three types of monitoring of the anaerobic reactor can be highlighted: (i) monitoring of the efficiency; (ii) monitoring of the stability; and (iii) monitoring of the amount and quality of the sludge.

Monitoring of the efficiency of the reactor

The historical behaviour of the unit and whether its performance is in accordance with the design specifications are established by monitoring the anaerobic reactor. Firstly, the course of the biological process itself is established, in terms of removal efficiency of undesirable constituents, by determining their concentrations in the influent and effluent of the biological reactor. The main constituents to be removed are:

- *suspended solids*: the concentration of suspended solids is determined by gravimetric tests on the total suspended solids (TSS) and on the volatile suspended solids (organic) (VSS). In addition, the traditional settleable solids test (determination of the volume of solids that settle in a 1-L cone during 1 hour) can be valuable if there is no precision scale available.
- *organic matter*: the organic matter removal efficiency is evaluated by the COD test and/or the BOD test. In addition, the biogas (or better, methane) production is a useful parameter in this respect.

- *pathogenic organisms*: regarding the hygienic quality, the establishment of the concentration of two types of organisms is recommended: (i) faecal coliforms (*E. coli*); and (ii) helminth eggs.

Monitoring of the stability of the digester

Monitoring of the operational stability of the treatment system aims at evaluating whether there are signs that the acid fermentation may prevail over the methanogenic fermentation, with the consequent acidification of the digester. In this sense, it is important to determine pH, alkalinity and concentration of volatile acids in the effluent, and compare these values with those in the influent. In addition, a sudden variation in the biogas composition and, especially, an increased percentage of carbon dioxide can be an indication of operational instability.

Monitoring of the sludge quantity and quality

Besides monitoring the efficiency and the stability of the reactor, tests should be performed to establish the quantitative and qualitative development of the sludge in the treatment system. The experimental determination of both presents problems. In systems with attached bacterial growth (immobilised biomass), such as fluidised bed reactors and anaerobic filters, the sludge is present in a form (biofilm) that makes its quantitative determination very difficult. In systems with dispersed bacterial growth, the concentration of sludge will not be uniform, and the determination of samples removed from several points is necessary. The concentration of both total solids (TS) and total volatile solids (TVS) should be determined.

The most important qualitative aspects of the sludge are:

- *Specific methanogenic activity*: reflects the capacity of the sludge to produce methane from an acetate substrate under optimised conditions. Although there are other processes developing in the anaerobic digester, the acetotrophic methanogenesis is the most important one because it is the limiting step in the conversion of the organic matter into methane. The test is performed in a laboratory according to the procedures described in Chapter 25. By knowing the SMA and the sludge mass in the biological reactor, it is possible to estimate the maximum organic load that can be digested in the reactor: this load is equal to the product of the SMA value and the sludge mass.
- *Stability*: aims at establishing which fraction of the sludge mass consists of still undigested biodegradable organic matter. A large fraction of biodegradable material in the sludge is not only an indication of an overloaded system, but it can also cause great problems to the solids–liquid separation of the excess sludge. Based on limited experience, van Haandel and Lettinga (1994) suggest that the fraction of biodegradable solids in the anaerobic sludge should be kept below 3%.
- *Settleability*: can be determined from a specific test described by Catunda and van Haandel (1989). This test is tedious and complicated, and the

application of a simpler, although less accurate method – the determination of the sludge volume index (SVI) or the diluted sludge volume index (DSVI) – is preferable in the operational routine.

The figure and table below (Anaerobic reactor) identify the main points, parameters and frequency of monitoring recommended for an anaerobic reactor. However, it should be highlighted that the monitoring parameters and frequency can be changed in view of local specificities and demands imposed by the environmental control agencies. A more intensive monitoring frequency may be necessary, particularly during the start-up of the system, as focused in the final items of this chapter.

(c) Monitoring of the drying beds

As mentioned previously, optimised operational conditions of the sludge dewatering unit have direct implications on the reduction of the volume of dry sludge to be transported to the final disposal location and also on the quality of the effluent leaving the anaerobic reactor. Thus, to reduce the drying cycles of the excess sludge, a continuous monitoring of the solids should be performed inside the reactor (prior to wastage) and on the drying beds (after the wastage). This monitoring is essential to define the best sludge wastage and dewatering routine, to contribute to reduced drying cycles and to the attainment of a dry sludge with low water content.

The purpose of the sludge dewatering is to reduce the percentage of water in the sludge as much as possible and, at the same time, improve its hygienic quality, maintaining, as much as possible, the organic matter and the nutrients (nitrogen and phosphorus) in the most suitable form to turn the sludge into an organic fertiliser.

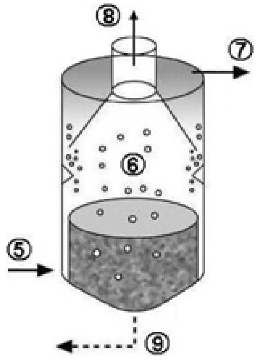
Regarding the operation of the drying beds, the most important parameters are:

- the load of solids applied to the bed
- the percolation time
- the composition and final quality of the dewatered sludge

Regarding the applied load, it is known that the sludge productivity (that is, the sludge mass that can be processed per unit area and per unit time to reach a certain desired final solids level) practically does not depend on the load applied, when it is within the range from 15 to 40 kg TS/m^2 (van Haandel and Lettinga, 1994).

Once the excess sludge is applied, the percolation and evaporation mechanisms start. As the fresh sludge flow is very small in relation to the sewage flow (approximately 0.1 to 0.2%), the composition of the percolated water (that returns to the treatment system) is not very important. The important parameters are the time necessary for percolation and the volume of percolated water. The water percolation is verified daily and, if applicable, the percolated volume is determined from the lowering of the sludge level on the bed (disregarding the water lost by evaporation).

Once percolation is finished, the composition in terms of total solids and the percentages of organic matter, nitrogen (organic and ammonia nitrogen) and phosphorus (total and orthophosphate) are determined at the end of the evaporation drying period.



Anaerobic reactor

Parameter	Unit	Monitoring points and frequency				
		5	6	7	8	9
Treatment efficiency						
Settleable solids	mL/L	Daily	–	Daily	–	–
TSS	mg/L	Weekly	–	Weekly	–	–
COD	mg/L	Weekly	–	Weekly	–	–
BOD	mg/L	Monthly	–	Monthly	–	–
Biogas production	m^3/d	–	–	–	Daily	–
E. coli	MPN/100 mL	Weekly	–	Weekly	–	–
Helminth eggs(1)	N/L	Weekly	–	Weekly	–	–
Operational stability						
Temperature	°C	Daily	Daily	–	–	–
pH	–	Daily	Daily	–	–	–
Bicarbonate alkalinity	mg/L	Weekly	–	Weekly	–	–
Volatile fatty acids	mg/L	Weekly	–	Weekly	–	–
Biogas composition	$\%CO_2$	–	–	–	Monthly	–
Sludge quantity and quality						
Total solids(2)	mg/L	–	–	–	–	Weekly
Total volatile solids(2)	mg/L	–	–	–	–	Weekly
Specific methanogenic activity	gCOD/gVS·d	–	–	–	–	Monthly
Sludge stability	gCOD/gVS·d	–	–		–	Monthly
Sludge volume index (diluted)	mL/g	–	–	–	–	Monthly

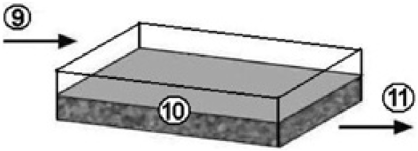
Notes:

(1) The procedures for identification and enumeration of helminth eggs are described in the "Health guidelines for use of wastewater in agriculture and acquaculture". Technical Report Series (WHO, 1989) and in Zerbini and Chernicharo (2001).

(2) The analyses of total solids should be made at several points along the height of the bed and sludge blanket (3 to 6 points), to establish the profile and the mass of solids inside the reactor (see Chapter 25, Example 25.1)

Regarding the hygienic quality, it is convenient to determine the concentration of coliforms and viable helminth eggs. In practice, the end of the evaporation time will be usually established by the need to use the bed again to dry more sludge, or by the transport availability for the dry sludge.

The following figure and table (Sludge drying bed) identify the main points, parameters and frequency of monitoring recommended for drying beds.



Sludge drying bed

		Monitoring points and frequency		
Parameter[(1)]	Unit	9	10	11
Excess sludge applied				
Volume of sludge wasted	m^3	During wastage	–	–
Initial sludge concentration	gVS/L	During wastage	–	–
Initial sludge composition	%VS	During wastage	–	–
Applied load	$kgTS/m^2$	–	During wastage	–
Faecal coliforms	MPN/gTS	During wastage	–	–
Helminth eggs	N/gTS	During wastage	–	–
Sludge on the bed and generation of percolate				
Height of the sludge	cm	–	During wastage	–
Percolation time	d	–	–	Daily[(2)]
Percolated volume	m^3	–	–	Daily
Evaporation time	d	–	–	Daily
Solids percentage	%	–	2 × week	–
Sludge composition	%VS	–	2 × week	–
Total COD	mg/L	–	–	2 × week
Nitrogen (TKN)	% of the TS	–	[(3)]	–
Ammonia nitrogen	% of the TS	–	[(3)]	–
Total phosphorus (P_{tot})	% of the TS	–	[(3)]	–
Orthophosphate	$\%P_{tot}$	–	[(3)]	–
Faecal coliforms	MPN/gTS	–	[(3)]	–
Helminth eggs	N/gTS	–	[(3)]	–

Notes:

(1) The monitoring frequency refers just to the period between the sludge wastage from the reactor and the end of the drying period (removal of the dry sludge)

(2) Check daily whether there is water percolation

(3) The nitrogen, phosphorus and microbiological parameters should be analysed after the end of the drying period (in the dry sludge)

28.2.2.3 Interpretation and recording of the operational data

Some of the most important aspects of anaerobic systems are their simplicity and operational stability. In this sense, the operational database is more used for comparison between certain parameters and historical values. Corrective measures should be taken when a considerable, extended deterioration of the treatment performance is noticed.

The troubleshooting list at the end of this chapter identifies some problems and actions for their solution. These problems can only be identified when comparing the results of the different tests and analyses included in the monitoring programme with the values from previous periods. In many cases, the indication of an operational problem does not result from the absolute value of a parameter, but from its variation. Thus, the need to maintain frequent reports that characterise the performance and general situation of the treatment system is evident.

28.2.3 Sludge measurement and characterisation

(a) Evaluation of the microbial mass

The determination of the biomass in anaerobic reactors presents two main difficulties:

- in some systems, the microorganisms are attached to small inert particles
- the biomass is usually present as a consortium of different morphological and physiological types

As presented in Chapter 25, the amount of biomass is usually evaluated by determining the solids profile, considering that the volatile solids are a measure of the biomass present in the reactors (mass of cellular material). The sludge samples are collected at different levels (heights) of the reactor, further gravimetrically analysed and the results are expressed in grams of volatile solids per litre (gVS/L). These concentration measures of volatile solids, multiplied by the volumes corresponding to each zone sampled, provide the masses of microorganisms over the profile of the reactor, as detailed in Example 25.1 (Chapter 25).

(b) Evaluation of the microbial activity

The success of any anaerobic process, especially the high-rate ones, depends fundamentally on maintaining an adapted shock-resistant biomass with a high microbiological activity inside the reactor. So that this biomass is preserved and monitored, the development of techniques for the evaluation of the microbial activity of anaerobic reactors became imperative, especially of the methanogenic Archaea.

The SMA test can be used as a routine analysis to quantify the methanogenic activity of anaerobic sludge, or also in a series of other applications, as pointed out in Chapter 25, where detailed information is presented on the procedure for this test.

28.2.4 Wastage of sludge from the system

The accumulation of biological solids occurs in anaerobic reactors after some months of continuous operation. The solids accumulation rate depends essentially on the type of effluent being treated, being higher when the influent wastewater presents a high concentration of suspended solids, especially non-biodegradable ones. The accumulation of solids is also due to the presence of calcium carbonate or other mineral precipitates, besides the biomass production itself. When the accumulation of solids other than for bacterial growth prevails, it can be reduced by a pre-treatment (coagulation, flocculation, sedimentation). The accumulation of biomass depends essentially on the chemical composition of the wastewater, being higher for those with high carbohydrate concentrations.

(a) Production of excess sludge and choice of the wastage point

To evaluate the amount of excess sludge produced in UASB reactors treating domestic sewage, an yield coefficient has been usually adopted ranging from 0.10 to 0.20 kgTSS per kgCOD applied to the system (see Chapter 27). In the case of reactor start-ups without seed sludge, the wastage of excess sludge should not be necessary during the initial months of operation of the reactor.

When the wastage is necessary in the case of UASB reactors, it should be done preferentially in the upper part of the sludge bed (less dense, more flocculent sludge, usually with lower specific methanogenic activity). However, attention should be given to the fact that the wastage of this lower concentration sludge will demand the removal of a larger sludge volume, for a given mass to be wasted, directly implying a larger area for the drying beds or a larger dewatering equipment.

An interesting alternative in relation to UASB reactors is to waste the sludge from different heights of the reactor, such as from the bottom (sludge bed) and from half-height of the digestion compartment (sludge blanket). Greater benefits can then be achieved than from the wastage from just a single height:

- The wastage from half-height of the digestion compartment enables the removal of the more disperse excess sludge, usually of lower activity and with poorer settleability.
- To compensate for the larger wastage volumes of this less dense sludge, a smaller portion of the sludge can be wasted from the bottom of the reactor, as it is very concentrated.
- The possible disadvantage to waste part of the bottom sludge, which usually presents higher activity and better settleability, can be compensated by the lower wastage volumes required and a consequent economy in the dewatering devices. Additionally, and depending on the quality of the preliminary treatment that precedes the reactor, the bottom sludge can accumulate inert solids, such as sand, which should be periodically discarded from the reactor. Consequently, the wastage of bottom sludge from the reactor, in small amounts and in a well-managed form, can bring important benefits to the treatment system.

(b) Wastage of excess sludge

An important operational aspect in systems with dispersed growth, such as the UASB reactor, is the wastage of excess sludge. In this case, it is necessary that the sludge mass is maintained between a minimum (dictated by the need to have a sufficient treatment capacity in the system to digest the influent organic load) and a maximum (dependent on the sludge retention capacity of the system) value. The wastage of sludge together with the effluent should be minimised, since this wastage increases the concentration of COD, BOD and suspended solids in the effluent.

On the other hand, the wastage frequency will be dictated by the nature of the dewatering process. In case of a mechanical process, such as a centrifuge, the tendency will be for a daily wastage while the operator is present in the plant. Should there be a drying bed, the tendency will be to apply a large wastage, decreasing the sludge mass in the system from a value close to that of the maximum mass to a value a little higher than that of the minimum mass. Thus, the sludge wastage frequency is reduced to a minimum (and so is the work related to this wastage), while a good performance and operational stability of the digester are ensured. The following routine can be followed to establish the wastage frequency and magnitude (Chernicharo *et al.*, 1999):

- by operating the reactor under normal flow and load conditions, without discharging the excess sludge, the sludge mass in the reactor and the daily sludge production are determined for a reactor “full” of sludge
- the SMA of the sludge is determined
- from the SMA value, the minimum sludge required to maintain a good reactor performance is determined
- the difference between the maximum sludge mass that can be kept in the system and the minimum sludge mass necessary for a good reactor performance is calculated
- after a wastage equal to or lower than the maximum wastage, the loss of sludge together with the effluent is determined again
- the wastage frequency can be determined as the ratio between the sludge mass to be wasted and the sludge accumulation rate in the system

Example 28.1

Aiming at minimising the level of suspended solids in the effluent from a UASB reactor, estimate the wastage frequency of the excess sludge, assuming wastage of 50% of the sludge mass.

Data:

- total reactor volume: $V = 1{,}003.5\ m^3$
- volume of the digestion compartment: $V_{dc} = 750.0\ m^3$

***Example 28.1* (*Continued*)**

- volume of the sedimentation compartment: $V_{sc} = 253.5\ m^3$
- depth of the reactor: 4.5 m
- average influent flowrate: $Q_{av} = 3{,}000\ m^3/d$
- average influent COD concentration: $S_0 = 600$ mg/L
- average effluent COD concentration (in the absence of sludge wastage): $C_{effl} = 198$ mg/L
- average concentration of suspended solids in the effluent (in the absence of sludge wastage): 80 mg/L
- average effluent COD concentration (after sedimentation): 130 mg/L
- specific methanogenic activity of the sludge (at 24 °C): 0.34 mgCOD-CH_4/mgVS·d
- average effluent COD concentration (after wastage of 50% of the sludge mass): 140 mg/L
- average concentration of suspended solids in the effluent (after wastage of 50% of the sludge mass): 20 mg/L

Solution:

(a) Estimate the sludge mass when the reactor is full

Considering the data of Example 25.1, an estimate of 36,950 kgTS and 22,170 kgVS has been obtained (assuming an average fraction of volatile solids in the sludge equal to 60%).

(b) Estimate the sludge production in the system

The concentration of solids (that are considered sludge particles) in the effluent is equal to 80 mgTSS/L. Therefore, the daily sludge production is: $3{,}000\ m^3/d \times 0.080\ kgTSS/m^3 = 240$ kgTSS/d.

The volatile sludge concentration is estimated from the difference between the effluent (without wastage) and the settled effluent: $198 - 130 = 68$ mgCOD/L.

Knowing that 1 mgVS/L has a COD of 1.5 mgCOD/L, the volatile sludge concentration in the effluent is calculated as: (68 mgCOD/L)/(1.5 mgCOD/mgVS) = 45 mgVS/L.

Note that the specific sludge production, that is, the ratio between the daily sludge production (240 kgTSS/d) and the daily organic load applied $(3,000 \times 0,600 = 1,800$ kgCOD/d) is equal to 0.13 $kgTSS/kgCOD_{applied}$, a value considered normal for anaerobic treatment.

(c) Estimate the sludge digestion capacity

From the specific methanogenic activity value and the volatile sludge mass, it is calculated that the sludge digestion capacity is: (0.34 kgCOD-CH_4/kgVS·d) × (22,170 kgVS) = 7,538 kgCOD/d.

***Example 28.1* (*Continued*)**

Note that the sludge digestion capacity is much higher than the influent load: (3,000 m^3/d) × (0.600 $kgCOD/m^3$) = 1,800 kgCOD/d.

(d) Estimate the sludge accumulation in the reactor, after wastage

After wastage of 50% of the sludge, the loss of solids together with the effluent decreases to 20 mg/L, and the daily sludge production is reduced to: 3,000 m^3/d × 0,020 $kgTSS/m^3$ = 60 kgTSS/d.

Therefore, the solids accumulation in the reactor can be estimated taking into account the sludge production before and after wastage: 240 kgTSS/d − 60 kgTSS/d = 180 kgTSS/d.

(e) Estimate the wastage frequency of excess sludge

As the wastage of 50% of the maximum mass represents an amount of: 36,950 kgTSS × 0.50 = 18,475 kgTSS, it is estimated that a period of (18,475 kgTSS)/(180 kgTSS/d) = 102 days will be necessary to fill the reactor with sludge again.

Another approach is to say that the accumulation of 180 kgTSS/d represents an addition of (180 kgTSS/d)/(83.7 $kgTSS/m^3$) = 2.15 m^3/d in the lower part of the reactor (where the concentration is 50.2 gVS/L or 83.7 gTS/L, according to Example 25.1). Therefore, it can be considered that the monthly wastage rate would be 2.15 m^3/d × 30 d = 64.5 m^3 of the sludge from the bottom of the reactor.

Hence, wastage strategies of either 50% of the sludge every 102 days (which represents a volume of approximately 220 m^3 of the bottom of the reactor) or monthly 64.5 m^3 wastages, also from the bottom of the reactor, can be adopted. Alternatively, a more diluted sludge could be wasted in the upper areas, but then the wastage volume would be increased accordingly.

28.2.5 Prevention against the release of foul odours

Until recently, anaerobic processes were associated with foul odours, and this became the main barrier for their larger use in the treatment of liquid effluents. The large number of studies and researches being carried out in the area, notably from the 1970s, resulted in greater knowledge of the microbiology and biochemistry of the anaerobic process and, consequently, of the measures to be adopted for the control of these gases.

The formation of bad smelling gases is usually associated with the reduction of sulfur compounds to hydrogen sulfide (H_2S). Measures should be taken to prevent these gases from escaping to the atmosphere, especially when there are houses close to the treatment area. As the hydrogen sulfide can escape from the reactor both in the liquid (dissolved in the effluent) and in the gas (gas collector), different measures should be taken.

It is necessary to cover the reactor to prevent the H_2S dissolved in the effluent from escaping to the atmosphere. In this case, covering the reactor will also enable a reduced occurrence of corrosion, since the entrance of oxygen will be significantly reduced. The hydrogen sulfide that escapes from the reactor together with the effluent can be removed by some post-treatment method, such as chemical precipitation or chemical or biochemical oxidation. An important aspect to prevent the release of gases dissolved in the effluent relates to the design of the submerged collection system, to avoid turbulence (see Chapter 27).

In relation to the H_2S extracted by the gas collector, together with methane and carbon dioxide, there are some treatment alternatives that can be applied (Belli Filho *et al.*, 2001):

- adsorption, by the passage of the gas through a porous material, such as activated carbon
- absorption, by the contact between the gas and a slightly volatile liquid (solvent), for example in scrubbing towers. In these towers, the gas is applied against the current with the solvent, favouring the maximum contact between gas and liquid
- biological treatment, for example with biological filters and biofilters (for gases). In biological filters, the biogas flow passes through a scrubbing tower containing a high amount of biomass attached to a packing medium. Regarding the biofilters, the biogas is introduced into a tank containing biologically active material (compost) and the microorganisms undertake the reactions, generating innocuous products such as carbon dioxide, water, mineral salts and microbial biomass
- chemical precipitation, by the passage through a hydraulic seal containing some precipitating element, leading, for instance, to chemical precipitation of the sulfide as FeS

28.2.6 Other operational precautions

Besides the precautions previously mentioned, the operational routine of wastewater treatment plants should include other equally important aspects:

- verification and continuous cleaning of the feeding devices of the anaerobic reactors. This measure is particularly important in UASB-type reactors, as the correct wastewater distribution from the upper part to the lower part of the reactors is essential for the appropriate operation of the treatment unit. It is recommended that the wastewater distribution tubes are verified (and, if necessary, unobstructed) daily
- verification of the occurrence of corrosion in the structure of the anaerobic reactor, particularly in steel parts such as gas collectors, guard rails, etc. In case of occurrence of corrosion, the affected structures should be repaired

quickly, aiming at both the integrity of the treatment unit and the safety of the system operators
- correct destination of all solid materials removed in the preliminary treatment (screens and grit chamber) and sludge wasted from the anaerobic reactor
- removal of the floating material layer (scum) that tends to accumulate on the free surface of the sedimentation compartment and inside the gas collector.

28.3 START-UP OF ANAEROBIC REACTORS

28.3.1 Preliminaries

The reduction of the period necessary for the start-up and improved operational control of the anaerobic processes are important factors to increase the efficiency and the competitiveness of the high-rate anaerobic systems. However, a more critical discussion on the similarities, differences and advantages of the different high-rate anaerobic systems regarding start-up, operation and monitoring is difficult, once the behaviour of the process depends fundamentally on the characteristics of the wastewater to be treated.

In general, high-rate anaerobic processes can be operated with organic loads much higher than those of the conventional anaerobic reactors, but frequently these highly efficient processes require longer start-up periods, better operational control and more qualified operators, so that the maximum performance of the system is reached, with minimal risks of process failure. From the practical point of view, it is more economical to operate the reactor under lower loads, thus decreasing the efforts for the control of the operation and the process.

The start-up of the anaerobic reactors and, in a smaller scale, their operation has been considered by technicians as a barrier, possibly due to bad experiences linked to the use of unsuitable operational strategies. Therefore, systematised operational procedures are very important, mainly during the start-up of high-rate systems, notably in the case of UASB reactors.

The start-up of anaerobic reactors is determined by the initial transient period, marked by operational instabilities. The start-up can be basically achieved in three different manners:

- *by using seed sludge adapted to the wastewater to be treated*: the start-up of the system occurs fast, in a satisfactory way, as there is no need for acclimatisation of the sludge
- *by using seed sludge not adapted to the wastewater to be treated*: in this case, the start-up of the system goes through an acclimatisation period, including a microbial selection phase
- *with no use of seed sludge*: this is considered the most unfavourable form to start up the system, once it will be necessary to inoculate the reactor with its own microorganisms contained in the influent wastewater. As the concentration of microorganisms in the wastewater is very small, the time

required for the retention and selection of a large microbial mass can be very long (4 to 6 months)

The start-up and operation of anaerobic filters and UASB reactors are covered in the following items, with special emphasis to the latter ones.

28.3.2 Start-up and operation of anaerobic filters

Usually, the start-up of anaerobic filters for the treatment of domestic sewage has not received much attention, possibly due to the following main aspects:

- anaerobic filters have been primarily applied to the treatment of the sewage from small populations (frequently below 500 inhabitants), and they are not the object of larger operational care in view of the dimension of the systems
- these reactors are provided with a packing medium, ensuring a larger retention of solids and biomass in the system, favouring the start-up process.

However, the anaerobic filters can be started up similarly to the UASB reactors, that is: (i) without seed sludge; (ii) with seed sludge not adapted to the type of wastewater to be treated; and (iii) with seed sludge adapted to the type of wastewater. As such aspects are covered in more detail in the following section, where guidelines for the start-up of UASB reactors are presented, only the aspects inherent to the anaerobic filters are discussed here.

(a) Grease removal

The problem of grease entering into a sewage treatment system results from the characteristics of this material, which tends to accumulate on the upper surface of the treatment units. As they are considered slow and hardly biodegradable materials, they form, together with other floating materials, a thick scum layer, which reduces the useful volume of the tank and tends to harm its operation.

The need for the implementation of grease removal units upstream the anaerobic filters depends intrinsically on the amount of oils and greases present in the wastewater. Although the implementation of these units is not a regular practice, the occurrence of operational problems due to the large presence of grease and the consequent scum formation in anaerobic reactors, particularly in the UASB reactors, has led several new designs of treatment plants to consider the implementation of a grease removal unit upstream the anaerobic reactors.

(b) Coarse solids removal

Like any other sewage treatment system, it is essential that the anaerobic filter is preceded by a preliminary treatment unit intended for the removal of coarse solids. This unit may consist of a screen, or simply of a collecting basket, depending on the size of the system and on the amount of coarse material present in the sewage.

The non-incorporation of coarse solids removal units preceding anaerobic filters contributes negatively to the occurrence of operational problems in these units.

For example, when larger floating solids have access to an anaerobic filter, they can obstruct the holes of the upper slab of the bottom compartment of the filter, which is a problem difficult to correct. In certain situations, when plastic bags, condoms and other similar objects are retained in the bottom compartment, the correction of the problem may require the closure of the filter, the removal of the packing medium and the removal of the bottom slab, to withdraw the material that caused the obstruction. Thus, it is essential to install a screening unit or a collecting basket upstream the anaerobic filters.

Having in mind that the installation of a screening unit or collecting basket has a very low cost compared with the other units of the system, it is recommended that these units are always present in any sewage treatment system.

(c) Wastage of sludge from the system

Young (1991) recommends that the solids should not be wasted from the reactor until the concentration in the sludge zone exceeds 5% (dry solids). Even in these conditions, wastage should only be performed if the sludge blanket penetrates the packing medium or if the concentration of solids in the effluent increases significantly. If the sludge blanket is not distinguished from the sludge bed (uniform distribution), solids should be wasted whenever the solids concentration is approximately 7%, in which case the flow of the solid mass will be hindered, which may favour the formation of preferential routes for the wastewater, besides hindering the removal of excess sludge.

28.3.3 Start-up and operation of UASB reactors

The successful application of the high-rate anaerobic processes is subject to the compliance with a series of requirements, which are mainly related to the concentration and activity of the present biomass, and also to the mixing and flow regime in the reactor, considering that all environmental factors (temperature, pH, alkalinity, etc.) are within the optimum range.

The most common objectives to be achieved in the operation of high-rate anaerobic processes are the control of the solids retention time (independently from the hydraulic detention time), the prevention against the accumulation of inert suspended solids in the reactor and the development of favourable conditions for mass transfer. These objectives are generally achieved when the reactors are well designed and constructed, and when appropriate procedures during the start-up and operation of the system are taken.

(a) Grease removal

The same considerations made in the previous section for anaerobic filters, regarding the importance of the installation of grease removal units preceding anaerobic reactors, are valid for the UASB reactors. The operational problems resulting from the non-removal (or inadequate removal) of grease can be highly detrimental, as these materials may enable the excessive accumulation of scum inside the gas collectors, hindering the release of gases and demanding special devices for its periodical removal.

Although the installation of a grease removal unit upstream UASB reactors is not a regular practice yet, the operational problems that have occurred in units already installed have called the designers' attention towards the inclusion of this unit in the design of new treatment plants.

(b) Removal of coarse solids

As highlighted in the beginning of this chapter and in Chapter 27, the effective removal of coarse solids before the sewage is directed to the UASB reactors is essential. In the particular case of the UASB reactors, the operational problems resulting from the non-removal (or inadequate removal) of coarse solids may jeopardise the whole operation of the treatment system, once these materials can adversely affect the distribution of the influent wastewater at the bottom of the reactor, and generate and accumulate a sludge with poor characteristics, with low activity and difficult to remove.

The concern with the excessive entry of larger dimension solids in the UASB reactors is so great that many of the new designs have considered the installation of sieves, with openings from 1 to 5 mm, to reduce at the most the operational problems resulting from the entry of solids into the reactor.

(c) Considerations and criteria for the start-up of the system

Volume of seed sludge. The volume of seed sludge for the start-up of the system is usually established as a function of the initial biological loading rate applied to the treatment system. The biological loading rate (kgCOD/kgVS·d) is the parameter that characterises the organic load applied to the system in relation to the amount of biomass present in the reactor (see Chapter 27, Equation 27.14). The biological load values to be applied during the start-up depend essentially on the type of seed sludge employed and on its acclimatisation to the wastewater to be treated. It is recommended that whenever possible, the biological load for the start-up be established by means of specific methanogenic activity tests of the sludge (see Chapter 25). Should it be impossible to perform these tests, biological loads in the range from 0.10 to 0.50 kgCOD/kgVS·d, relating to specific methanogenic activities between 0.10 and 0.50 kgCOD-CH_4/kgVS·d, are used during the start-up of the process. These initial loads should be gradually increased according to the efficiency of the system and the improved activity of the biomass.

Volumetric hydraulic load. The volumetric hydraulic load is equal to the amount (volume) of sewage applied daily to the reactor per unit volume (see Chapter 27, Equation 27.8). The hydraulic load produces at least three different effects on the biomass of the reactor during the start-up of the system:

- the hydraulic load removes all the biomass with poor settling characteristics, thus creating space for the new biomass that is growing
- with the removal of part of the new biomass, which does not have good settleability, a selection of the active biomass is made
- the hydraulic load has a strong influence on the mixing characteristics of the reactor, mainly during the start-up of the system

In view of that, the dilution of very concentrated wastewater (COD > 5,000 mg/L) is essential, aiming at obtaining higher hydraulic loads during the initial transient period (Lettinga *et al.*, 1984).

Biogas production. Biogas production is very important in UASB reactors for good mixing of the sludge bed. However, very high gas production rates can adversely affect the start-up of the process because the sludge can expand excessively towards the upper part of the reactor, being lost together with the effluent.

Temperature. The ideal operation temperature for anaerobic reactors is in the range of 30 to 35 °C, when the growth of most of the anaerobic microorganisms is considered ideal. In the case of domestic sewage treatment, this range of temperature is hardly reached, once the average temperature of the influent sewage in warm-climate regions usually ranges from 20 to 28 °C. Under these sub-optimum temperature conditions, the anaerobic reactors are started up more easily with the inoculation of sufficient amounts of anaerobic sludge, preferably acclimatised to the type of sewage.

Environmental factors. For an optimum start-up of the system, it is desirable that the environmental factors are favourable, in accordance with the following main guidelines:

- whenever possible, the temperature inside the reactors should be close to the ideal growth range of anaerobic microorganisms (30 to 35 °C). In the case of domestic sewage treatment, these temperatures are not feasibly reached, which makes the start-up of the system under the ideal temperature conditions virtually impossible
- pH should be always maintained above 6.2 and, preferably, in the range from 6.8 to 7.2
- all the growth factors (N, P, S and micronutrients) should be present in sufficient amounts
- the toxic compounds should be absent in inhibiting concentrations. Otherwise, sufficient time should be provided for acclimatisation of the microorganisms

Acclimatisation and selection of biomass. The first start-up of an anaerobic reactor is a relatively delicate process. In the case of UASB reactors, sufficient, continuous removal of the lightest sludge fraction is essential, to allow the selection of the heaviest sludge for growth and aggregation. The main guidelines for acclimatisation and selection of biomass in UASB reactors are as follows (adapted from: Lettinga *et al.*, 1984):

- do not return to the reactor the dispersed sludge lost together with the effluent
- dilute the influent or recirculate the effluent, when the concentration of wastewater exceeds 5,000 mgCOD/L
- increase the organic load progressively, whenever the BOD or COD removal efficiency reaches at least 60%

- keep the acetic acid concentrations below 1,000 mg/L. In the case of domestic sewage treatment, the expected acetic acid concentrations in the reactor are much lower, and they should be maintained below 200 to 300 mg/L
- provide the necessary alkalinity to the system, to maintain the pH close to 7

(d) Procedure preceding the start-up of a reactor

Characterisation of the seed sludge. Once the use of seed sludge is defined for the start-up of the reactor, analyses should be carried out for its qualitative and quantitative characterisation, including the following parameters: pH, bicarbonate alkalinity, volatile fatty acids, TS, VS, and SMA. Besides the parameters referred to above, a visual and olfactory characterisation of the sludge should be carried out.

Characterisation of the raw sewage. To establish the start-up routine of the anaerobic reactor, a qualitative and quantitative characterisation campaign of the influent raw sewage should be carried out.

Estimation of the seed sludge volume necessary for the start-up of the reactor. Based on the results of the characterisation analyses of the sludge and the influent sewage, the seed sludge volume necessary for the start-up of the reactor can be estimated, as shown in Example 28.2.

Example 28.2

Estimate the amount of sludge necessary for the inoculation of a UASB reactor, knowing the following elements:

Data:

- Influent flowrate: $Q_{av} = 3{,}000\ m^3/d$ (adopted as an average of the characterisation campaign)
- Sewage concentration: $S_o = 600$ mgCOD/L (adopted as an average of the characterisation campaign)
- Concentration of volatile solids in the seed sludge: $C = 30{,}000$ mgVS/L (3%) (adopted as an average of the samples analysed)
- Density of the seed sludge: $\gamma = 1{,}020\ kg/m^3$
- Volume of the reactor: $V = 1{,}003.5\ m^3$
- Biological loading rate adopted during the start-up of the reactor: $L_s = 0.3$ kgCOD/kgVS·d

Solution:

- Applied organic load (L_o):
  $L_o = Q_{av} \times S_o = 3{,}000\ m^3/d \times 0.600\ kgCOD/m^3$
  $L_o = 1{,}800$ kgCOD/d

- Necessary seed sludge mass (M_s):
  $M_s = L_o/L_s = (1{,}800\ kgCOD/d)/(0.3\ kgCOD/kgVS{\cdot}d)$
  $M_s = 6{,}000$ kgVS

Example 28.2 (*Continued*)

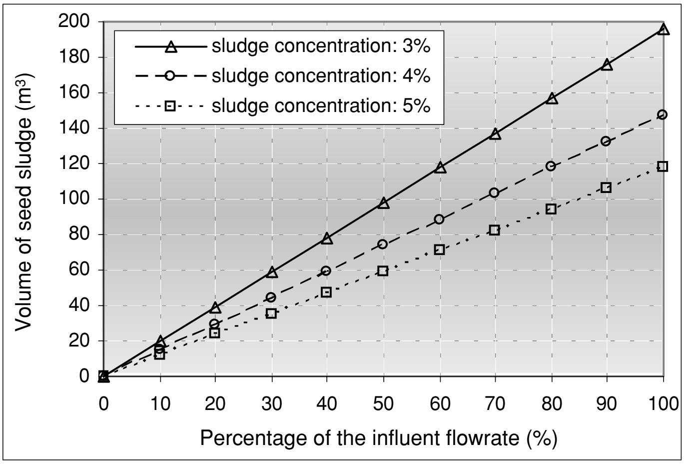
- Resulting seed sludge volume (V_s):
  $V_s = P_s/(\gamma \times C_s)$ – see Chapter 27, Equation 27.27
  $V_s = (6{,}000 \text{ kgVS})/(1{,}020 \text{ kg/m}^3 \times 0.03)$
  $V_s = 196 \text{ m}^3$

As the necessary seed sludge volume is relatively high (196 m^3), equivalent to approximately 32 tank trucks, the possibility of not applying the total organic load can be evaluated, diverting (by-passing) part of the influent sewage to the overflow weir of the treatment plant during the first few days of the reactor start-up.

The following figure enables the visualisation of some alternatives for inoculation and start-up of the anaerobic reactor, taking into consideration the application of different influent flow percentages as a function of the volatile solids concentrations in the sludge.

In the figure, the percentage of applied flow refers to the average flow obtained in the characterisation campaign of the influent (e.g.: 50% refers to the application of an influent flowrate equal to 1,500 m^3/d). Possible alternatives for inoculation of the reactor can be evaluated by means of graphical aid, as exemplified below:

- for application of 100% of the influent flowrate, considering a sludge with a concentration of volatile solids equal to 3%, a seed sludge volume equal to approximately 200 m^3 is necessary
- for application of 50% of the influent flow, considering a sludge with a concentration of volatile solids equal to 5%, a seed sludge volume equal to approximately 60 m^3 is necessary



Graphical representation of seed sludge volumes necessary for the start-up of an UASB reactor, considering the conditions of Example 28.2 and different seed sludge concentrations

(e) Procedure during the start-up of an anaerobic reactor

The procedure during the start-up of the reactor refers mainly to: (i) inoculation, (ii) feeding with wastewater and (iii) monitoring of the process.

Inoculation of the reactor

The inoculation can be done with the reactor either full or empty, although the inoculation is preferable with the reactor empty, to reduce sludge losses during the transfer process. For this second situation, the following procedures can be adopted:

- transfer the seed sludge to the reactor, ensuring that it is discharged into the bottom of the reactor. Avoid turbulence and excessive contact with air
- leave the sludge at rest for an approximate period of 12 to 24 hours, allowing its gradual adaptation to local temperature

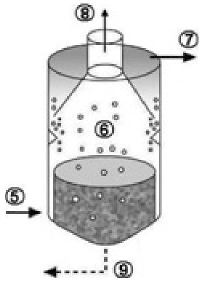
Feeding of the reactor with sewage

- after the end of the rest period, begin the feeding of the reactor with wastewater, until it reaches approximately half of its useful volume
- leave the reactor unfed for a 24-hour period. At the end of this period, and prior to beginning the next feeding, collect supernatant samples from the reactor and analyse the following parameters: *temperature, pH, alkalinity, volatile acids and COD*. Should these parameters be within acceptable ranges, continue the feeding process. *Acceptable values*: *pH between 6.8 and 7.4 and volatile acids below 200 mg/L (as acetic acid*)
- continue the filling process of the reactor, until it reaches its total volume (level of the sedimentation tank weirs)
- leave the reactor unfed again for another 24-hour period. At the end of this period, collect new samples for analyses and proceed as previously stated
- if the parameters analysed are within the established ranges, feed the reactor continuously, in accordance with the amount of seed sludge used and the flow percentage to be applied (see above figure)
- implement and perform a routine monitoring of the treatment process
- increase the influent flow gradually, initially every 15 days, in accordance to the system response. This interval can be either increased or reduced, depending on the results obtained

Monitoring of the treatment process

For the monitoring of the treatment process, the sample collection routine and the physical–chemical parameters to be analysed should be defined during the start-up period. An example of a monitoring programme that has been adopted in the start-up of UASB reactors is presented in Table 28.1.

Table 28.1. Monitoring programme of a UASB reactor during the start-up period



Parameter	Unit	Monitoring points and frequency⁽¹⁾ 5	6	7	8	9
Treatment efficiency						
Settleable solids	mL/L	Daily	–	Daily	–	–
TSS	mg/L	3 × week	–	3 × week	–	–
Total COD	mg/L	3 × week	–	3 × week	–	–
Total BOD	mg/L	Weekly	–	Weekly	–	–
Biogas production	m^3/d	–	–	–	Daily	–
Operational stability						
Temperature	°C	Daily	Daily	–	–	–
pH	–	Daily	Daily	–	–	–
Bicarbonate alkalinity	mg/L	3 × week	–	3 × week	–	–
Volatile fatty acids	mg/L	3 × week	–	3 × week	–	–
Biogas composition	$\%CO_2$	–	–	–	Weekly	–
Sludge quantity and quality						
Total solids⁽²⁾	mg/L	–	–	–	–	Monthly
Total volatile solids⁽²⁾	mg/L	–	–	–	–	Monthly
Specific methanogenic activity	gCOD/gVS·d	–	–	–	–	2 × month
Stability of the sludge	gCOD/gVS·d	–	–		–	Monthly
Sludge volume index (diluted)	mL/g	–	–	–	–	Monthly

Notes:

(1) The analysis frequency can be reduced over the start-up of the process, in accordance with the results achieved

(2) The total solids should be analysed at various points along the height of the bed and sludge blanket (3 to 6 points), to obtain the profile and the mass of solids inside the reactor (see Chapter 25, Example 25.1)

28.4 OPERATIONAL TROUBLESHOOTING

The following items present a set of information that can help detect and correct operational problems in anaerobic reactors, based on the work of Chernicharo *et al*. (1999).

Flow and characteristics of the influent

Observation	Probable cause	Verify	Solution
Flow always lower than the expected one	Population or *per capita* contribution lower than the design value	Flow measuring device	Increase served population
Flow suddenly lower than the expected one	Blockages in sewerage system	Overflow in the contribution area	Unblock sewers
Flow always higher than the expected one	Population or *per capita* contribution higher than the design value	Flow measuring device	Increase treatment capacity
Daily peaks higher than the expected ones	Equalisation lower than the expected one	Flow measuring device	Consider equalisation tank
Sudden irregular peaks	Combined system or cross-connection with stormwater sewers	Coincidence with rains	Disconnect illegal connections
Flow sometimes higher than the expected one	Large infiltration of groundwater	Coincidence with rains	Find the infiltration points
pH higher or lower than normal	Industrial wastewater	Existence of illegal sources	Find and act on the sources to correct the problem
Temperature higher or lower than the normal	Industrial waste	Existence of illegal sources	Find and act on the sources to correct the problem
Settleable solids larger than normal	Illegal dumping of domestic or industrial solid wastes in the sewerage system	Nature of the settleable solids	Find and act on the sources to correct the problem

Source: Chernicharo *et al*. (1999)

Preliminary treatment

Observation	Probable cause	Verify	Solution
Odour or insects at the screen	Long interval between cleanings	Cleaning interval	Increase the cleaning frequency
Sudden increase in the mass of coarse solids retained	Illegal dumping of solid wastes	Existence of illegal sources	Find and act on the sources to correct the problem
Sudden decrease in the mass of coarse solids retained	Retention failure at the screen	Condition of the screen	Repair the screen
Sudden increase in the grit mass retained	Discharge of stormwater into the sewerage system	Sewage flow	Disconnect illegal connection
Sudden decrease in the sand mass retained	Sand dragged from the grit chamber	Flow velocity (dye tracer)	Reduce velocity
Rotten egg odour in the grit chamber	Sedimentation of organic matter	Flow velocity (dye tracer)	Increase water velocity
Sand retained is grey, has a bad odour and contains grease	Sedimentation of organic matter	Flow velocity (dye tracer)	Increase water velocity
Metal and concrete corrosion in the preliminary treatment units	Insufficient ventilation	Ventilation	Improve ventilation

Source: Chernicharo *et al*. (1999)

Performance of the UASB reactor

Observation	Probable cause	Verify	Solution
Unequal influent distribution	Distribution structure unlevelled	Level of the distribution structure	Level the distribution structure
Distribution tube does not receive sewage	Blocking	Blocking	Unblock
Non-uniform effluent collection	Collection structure unlevelled	Level of the collection structure	Level the collection structure
	Surface layer obstructs collection points	Flow conditions	Remove obstruction
High level of settleable solids in the effluent	Excessive hydraulic load	Flow	Reduce flow
	Excessive solids in the reactor	Sludge mass	Waste the excess sludge
Gas production lower than normal	Biogas leakage	Gas collection	Eliminate leakage
	Defective gas meter	Gas meter	Either repair or replace
	Reduced flow	Influent flow	Unblock sewers
	Toxic material in the influent	SMA test	Identify and act on sources of toxic material
	Excessive organic load	SMA and stability test	Reduce organic load
Sludge production higher than normal	Overloaded sludge	Sludge stability	Reduce applied load
	Coarse and/or inorganic solids entering the reactor	Pre-treatment operation	Re-establish operation of the pre-treatment units
Sludge production lower than normal	Small flow	Influent flow	Unblock sewers
	Deficient sludge retention	Phase separator; settleable solids in the effluent	Repair separator
Sludge with high fraction of inorganic solids	Defective grit chamber	Velocity in the chamber	Decrease velocity in grit chamber
	Low upflow velocity in the reactor	Velocity	
Floating sludge grows quickly	Excessive hydraulic load	Organic and hydraulic loads	Reduce load
Reduced efficiency in the removal of organic matter	Excessive load	Load	Reduce load
	Deficient influent distribution	Influent distribution system (tracer studies)	Repair failure

Source: Chernicharo *et al*. (1999)

Characteristics of the sludge in the reactor

Observation	Probable cause	Verify	Solution
SMA lower than the expected one	Entry of inert solids	Settleable solids in influent	Reduce source or revise pre-treatment
	Overload	Sludge stability and removal efficiency of the organic matter	Reduce load
	Presence of toxic material	Test stored sludge	Identify and act on sources of toxic materials
Poor stability	Sludge overload	Specific organic load	Reduce specific load
High sludge volume index	Biodegradable organic matter	Stability	Reduce organic load
	Low hydraulic load	Upflow velocity	Increase dragging temporarily
Poor settleability	Dispersed flocs due to excessive organic load	Sludge stability	Reduce load
	Presence of toxic material	SMA of the sludge	Identify and act on sources of toxic materials
Increased specific sludge production	Flocculation without metabolism	Sludge stability	Reduce specific organic load
Increased inorganic fraction	Entrance of silt and sand	Velocity in the grit chamber	Reduce velocity in the grit chamber
	Low upflow velocity	Upward velocity in the reactor	Increase hydraulic load

Source: Chernicharo *et al*. (1999)

Sludge drying beds

Observation	Probable cause	Verify	Solution
Generation of bad odour when applying sludge to the bed	Sludge instability	Sludge stability (test)	Adjust organic load
Excess sludge wastage tubing blocked	Accumulation of solids and sand	Occurrence of blocked pipes	Clean tubing after use
Excessive percolation time	Excessive load applied	Applied load	Reduce load
	Inadequate bed cleaning	–	Improve maintenance
	“Blind” sand	Verify permeability	Replace sand
	High rainfall	–	Cover bed
	Drainage system blocked	–	Apply upflow washing
	Air trapped in the bed preventing passage of water	Upflow washing with water	Apply water in upward direction, saturating the bed before sludge wastage
Excessive evaporation time	Excessive load applied	Load applied	Reduce load
	High rainfall, low temperatures, high air humidity		Reduce load/cover bed
Very diluted excess sludge	Sludge removal from a very high level in the reactor	Solids concentration profile	Remove the sludge from a lower level (closer to the bottom of the reactor)
Mosquito reproduction on the beds	Semi-permanent water layer	Drainage system	Reduce load, improve permeability

Source: Chernicharo *et al*. (1999)

29

Post-treatment of effluents from anaerobic reactors

29.1 APPLICABILITY AND LIMITATIONS OF THE ANAEROBIC TECHNOLOGY

29.1.1 Applicability for the treatment of domestic sewage

A deep discussion on the evolution and applicability of the anaerobic technology for the treatment of domestic sewage was presented in Chapter 23, where several favourable characteristics of the anaerobic processes were highlighted, such as low cost, operational simplicity, no energy consumption and low production of solids. These advantages, associated with favourable environmental conditions in warm-climate regions where high temperatures prevail practically throughout the year, have contributed to establish the anaerobic systems, particularly the UASB reactors, in an outstanding position.

Nowadays, it can be said that the high-rate anaerobic reactors used for treatment of domestic sewage are a consolidated technology in some warm-climate countries, especially in Brazil, Colombia and India, with several treatment systems operating in full scale (population equivalents from a few thousand up to around one million inhabitants). In Brazil, practically all the wastewater treatment feasibility studies include anaerobic reactors as one of the main options. Undoubtedly, a great contribution to the consolidation and dissemination of the anaerobic technology for the treatment of domestic sewage came from the Brazilian National Research Programme on Basic Sanitation, PROSAB.

Biological Wastewater Treatment in Warm Climate Regions by Marcos von Sperling and Carlos Augusto de Lemos Chernicharo. ISBN: 1 84339 002 7. Published by IWA Publishing, London, UK.

29.1.2 Main limitations

In spite of their great advantages, anaerobic reactors hardly produce effluents that comply with usual discharge standards established by environmental agencies. Therefore, the effluents from anaerobic reactors usually require a post-treatment step as a means to adapt the treated effluent to the requirements of the environmental legislation and protect the receiving water bodies.

The main role of the post-treatment is to complete the removal of organic matter, as well as to remove constituents little affected by the anaerobic treatment, such as nutrients (N and P) and pathogenic organisms (viruses, bacteria, protozoans and helminths).

(a) Limitations regarding organic matter

Limitations imposed by environmental agencies for BOD are usually expressed in terms of effluent discharge standards and minimum removal efficiencies. These constraints are probably the cause that has mostly limited the use of anaerobic systems (without post-treatment) for sewage treatment (see typical values in Table 29.1).

In view of the limitations imposed by environmental legislation for the effluent BOD concentration, or also when the receiving body has limited capacity for assimilating the effluent from the treatment plant (which is frequently the case), it is usually necessary to use aerobic treatment to supplement the anaerobic stage. However, there are situations in which the combination of different anaerobic processes can meet less restrictive requirements regarding efficiency and concentration of the final effluent (e.g. 80% and 60 mgBOD/L, respectively). This is the case for systems consisting of a septic tank followed by an anaerobic filter (usually feasible for small populations, generally fewer than 1,000 inhabitants) or for a UASB reactor followed by an anaerobic filter. Obviously, the application of these combined anaerobic systems is conditioned to an appropriate dilution capacity of the receiving body.

In this sense, in situations in which the receiving body presents a good dilution capacity, the adoption of less restrictive discharge standards could enable the

Table 29.1. Usual effluent BOD and removal efficiencies in anaerobic systems

Anaerobic system	Effluent BOD (mg/L)	BOD removal efficiency (%)
Anaerobic pond	70 to 160	40 to 70
UASB reactor	60 to 120	55 to 75
Septic tank	80 to 150	35 to 60
Imhoff tank	80 to 150	35 to 60
Septic tank followed by anaerobic filter	40 to 60	75 to 85

Source: Chernicharo *et al.* (2001c)

construction of simpler and more economical treatment plants in several small cities by means of a more intensive use of anaerobic reactors, particularly UASB reactors. At a later stage, if it becomes necessary to produce a better quality effluent, a complementary treatment unit can be built after some years. The high costs of sophisticated treatment systems, designed exclusively to meet BOD discharge standards, make their construction at a single stage unfeasible for most cities located in developing countries. On the other hand, the construction in stages could be decisive, in that systems consisting of a UASB reactor and a post-treatment unit become the most feasible ones regarding technical and economical criteria.

(b) Limitations regarding nitrogen and phosphorus

The discharge of nutrients into surface water bodies may cause increased algal biomass as a result of the eutrophication process (abnormal algae growth due to the nutrients discharged, see Chapter 3). It is known that 1.0 kg of phosphorus can result in the reconstruction of 111 kg of biomass, which corresponds to approximately 138 kg of chemical oxygen demand in the receiving body. Similarly, the discharge of 1.0 kg of nitrogen can result in the reconstruction of approximately 20 kg of chemical oxygen demand under the form of dead algae. The problem can be even worsened due to the decreased oxygen levels, by means of the nitrification processes, when at least 4.0 kg of dissolved oxygen are consumed for each kilogram of ammonia discharged into the receiving body.

In cases in which nutrient removal is required to meet the quality standards of the receiving water body, the use of anaerobic processes preceding a complementary aerobic treatment for biological nutrient removal should be analysed very carefully, once anaerobic systems present good biodegradable organic matter removal, but practically no N and P removal efficiency. This certainly causes an adverse effect on biological treatment systems aiming at good nutrient removal, because the effluent from the anaerobic reactor will have N/COD and P/COD ratios much higher than the values desired for good performance of biological nutrient removal processes (Alem Sobrinho and Jordão, 2001).

When the purpose of the treatment plant is also good nitrogen removal, the anaerobic reactor should be used to treat initially only a part of the influent raw sewage (possibly no more than 50 to 70%), and the remaining part (50 to 30%) should be directed to the complementary biological treatment, aiming at nitrification and denitrification, so that there is enough organic matter for the denitrification step. In this case, the great advantage of the use of the anaerobic reactor is that it can receive and stabilise the sludge generated in the complementary treatment, eliminating the need for an anaerobic sludge digester.

On the other hand, when the purpose is the biological phosphorus removal, the use of an anaerobic reactor is not advisable for two main reasons: (i) the effluent from the anaerobic reactor presents a P/COD ratio higher than that of the raw sewage, which harms the performance of the biological phosphorus removal system; and (ii) if the phosphorus-rich sludge generated in the biological phosphorus removal treatment is directed to the anaerobic reactor for stabilisation, the phosphorus incorporated to this sludge will be released under anaerobic conditions and leave with the effluent from the anaerobic reactor. This fact makes efficient phosphorus removal unfeasible in a treatment plant with an anaerobic reactor followed by complementary treatment with biological phosphorus removal.

According to Alem Sobrinho and Jordão (2001), phosphorus removal in treatment plants using an anaerobic reactor will only be effective if chemical products are used for P precipitation (iron or aluminium salts). In this case, the anaerobic reactor has the advantage of stabilising the sludge generated in the complementary biological aerobic treatment.

(c) Limitations regarding microbiological indicators

Regarding the microbiological indicators, low faecal coliform removal efficiencies have been reported in anaerobic reactors, usually amounting to around only 1 log-unit. Regarding other types of microorganisms, such as viruses and protozoans (mainly *Giardia* and *Cryptosporidium*), there are few references covering their reduction or elimination in anaerobic reactors. The removal of helminth eggs in anaerobic reactors, particularly in UASB reactors, has been reported as amounting to 60 to 90%, being therefore insufficient to produce effluents that may be used in irrigation. However, it should be mentioned that these limitations are not exclusive of anaerobic reactors, but are a characteristic of most compact wastewater treatment systems.

As the risk of human contamination by ingestion or contact with water containing pathogenic organisms is high, many times it may be necessary to disinfect the effluents. This fact becomes even more serious due to the poor sanitary conditions in developing countries. On the other hand, the low investments in health and sanitation make the population of these countries bearers of several diseases that can be transmitted by faeces and, consequently, by the sewage generated by this population.

However, although the domestic sewage is an unquestionable source of contamination by pathogenic organisms, it is worth mentioning that the agents used in the disinfection processes can also cause harm to human health and the aquatic environment. It is then concluded that the decision whether or not to disinfect the sewage should be taken from a careful evaluation, based on the specific characteristics of each situation. In other words, there are no universal guidelines ruling sewage disinfection requirements. The decision on the need to disinfect the sewage of a certain locality involves (USEPA, 1986):

- an investigation on the uses of the water downstream the discharge point, and on the public health risks associated with that water
- an evaluation of the alternatives available for control of the sewage contaminated by pathogens
- an evaluation of the environmental impacts the control measures may cause

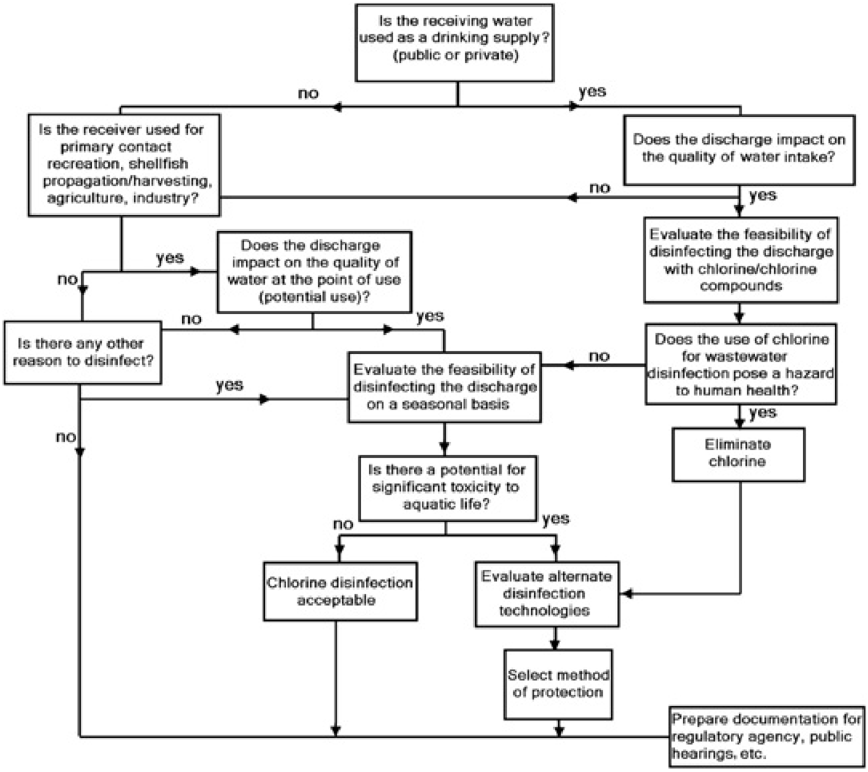


Figure 29.1. Flowchart for local evaluation of the need for and requirements of sewage disinfection (adapted from USEPA, 1986)

Figure 29.1 presents a flowsheet that can aid the decision making on the implementation need and requirements of a sewage disinfection system, taking into account the public health risks involved and the possibility of either reducing or eliminating these risks. Once the risks involved are identified, the environmental aspects start to determine the applicability of the control alternative.

29.1.3 Advantages of the combined (anaerobic/aerobic) treatment

In comparison with a conventional wastewater treatment plant consisting of a primary sedimentation tank followed by aerobic biological treatment (activated sludge, trickling filter, submerged aerated biofilter, or biodisc), with the primary and secondary sludge passing through sludge thickeners and anaerobic digesters prior to dewatering, a treatment consisting of a UASB reactor followed by aerobic biological treatment (with the secondary sludge directed to thickening and digestion in the UASB reactor itself and then straight to dewatering) can present the following advantages (Alem Sobrinho and Jordão, 2001):

- The primary sedimentation tanks, sludge thickeners and anaerobic digesters, as well as all their equipment, can be replaced with UASB reactors, which do not require the use of equipment. In this configuration, besides their main sewage treatment function, the UASB reactors also accomplish the aerobic sludge thickening and digestion functions, requiring no additional volume.
- Power consumption for aeration in activated sludge systems preceded by UASB reactors will be substantially lower compared to conventional activated sludge systems, and especially extended aeration systems.
- Thanks to the lower sludge production in anaerobic systems and to their better dewaterability, sludge volumes to be disposed of from anaerobic–aerobic systems will be much lower than those from aerobic systems alone.
- The construction cost of a treatment plant with a UASB reactor followed by aerobic biological treatment should be no more than 80% of the cost of a conventional treatment plant. In addition, due to the simplicity, smaller sludge production, and lower power consumption of the combined anaerobic–aerobic system, the operational costs also represent an even greater advantage.

Figure 4.24 (Chapter 4) illustrates several possible flowsheets comprising UASB reactors followed by some form of aerobic or anaerobic post-treatment. Section 4.7.2 (Chapter 4) presents a comparison among different treatment flowsheets, including conventional ones and several variants of UASB reactor + post-treatment unit.

29.2 MAIN ALTERNATIVES FOR THE POST-TREATMENT OF EFFLUENTS FROM ANAEROBIC REACTORS

29.2.1 Preliminaries

Taking into consideration the intrinsic limitations associated with the anaerobic systems and the need to develop technologies that are more appropriate to the reality of developing countries, it is important to include a post-treatment stage for the effluents generated in anaerobic reactors. This stage has the purpose of polishing not only the microbiological quality of the effluents, in view of the public health risks and limitations imposed on the use of treated effluents in agriculture, but also the quality in terms of organic matter and nutrients, in view of the environmental damages caused by the discharges of the remaining loads of these components into the receiving bodies.

Considering that the treatment line consisting of *anaerobic reactors+post-treatment units* is an important alternative for developing countries, the main progresses achieved on this subject by the Brazilian National Research Programme on Basic Sanitation, PROSAB (Chernicharo *et al.*, 2001c) are presented in this chapter. The main aspects of the most important post-treatment alternatives being applied in Brazil are herein discussed.

29.2.2 Anaerobic filter

29.2.2.1 Preliminary considerations

The main innovative purpose of the research was to evaluate the applicability of an anaerobic process (anaerobic filter) used for the polishing of domestic sewage, whose previous treatment stage is also performed by another anaerobic process (UASB reactor). This association of anaerobic processes contributes greatly to the reduction of power and operational costs of the treatment plant.

Until recently, the anaerobic filters were limited to small populations, usually treating effluents from septic tanks. Nowadays, anaerobic filters after UASB reactors are being used to produce a final effluent with BOD lower than 60 mg/L, even in cities with population larger than 50,000 inhabitants. The complementary organic matter removal achieved in the second anaerobic reactor (anaerobic filter) occurs by:

- the retention of solids in the anaerobic filter, reflecting on the removal of particulate organic matter. In this case, physical removal mechanisms prevail through the combined effects of coarse filtration in the packing medium and sedimentation along the column
- the formation of biofilm on the packing medium and removal of the remaining soluble organic matter. In this case, the formation of biofilm and the removal of carbonaceous matter by biochemical means depend on the amount of organic matter present in the effluent from the UASB reactor.

29.2.2.2 Typical configuration

Wastewater treatment plants using UASB reactors followed by anaerobic filters represent a very simple flowsheet (Figure 29.2). Besides the preliminary treatment

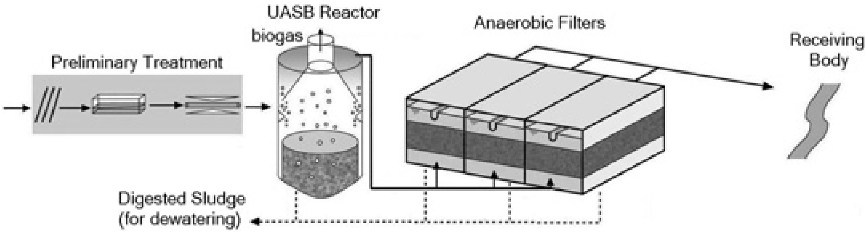


Figure 29.2. Typical configuration of a treatment plant with a UASB reactor and an anaerobic filter

units (screen and grit chamber), the flowsheet comprises basically the two sequential anaerobic treatment units (UASB reactor and anaerobic filter) and the dewatering unit. This is because the sludge produced in the anaerobic units are already thickened and stabilised. Sludge drying beds have been frequently used for sludge dewatering in small plants. UASB reactor+anaerobic filter facilities have already been installed in some locations in Brazil, as shown in Figures 27.3 and 27.4.

29.2.2.3 Design criteria

A deep discussion on the main design criteria and parameters for anaerobic filters is presented in Chapter 27. These criteria were obtained from pilot-scale research and from operational results from full-scale plants.

29.2.3 Polishing ponds

29.2.3.1 Preliminary considerations

Facultative ponds are largely used for post-treatment of effluents from anaerobic ponds. These systems have the advantage of removing at a higher efficiency the pathogenic organisms present in the sewage, but their main disadvantage is the high concentration of algae in the final effluent, which leads to serious restrictions by some environmental agencies.

When an efficient anaerobic pre-treatment is applied prior to the sewage discharge into a pond, the concentrations of organic matter and suspended solids are largely reduced, and consequently only a complementary removal of these two constituents will be required, needing much lower hydraulic detention times. In these conditions, the limiting factor that determines the minimum detention time (and, therefore, the volume and the area of a pond system) will usually be the removal of pathogenic organisms, and not the stabilisation of the organic matter. For this reason, the nomenclature **polishing pond** has been adopted to name those ponds intended for the post-treatment of effluents from efficient anaerobic systems, thus distinguishing them from the **stabilisation pond,** which treats raw sewage (Cavalcanti *et al.*, 2001).

The UASB reactor+polishing pond configuration is a very interesting alternative from the technical–economical–environmental point of view, mainly when there are area limitations for the construction of only stabilisation ponds. In addition, the problems related to odours from anaerobic ponds can be avoided in plants utilising a UASB reactor and polishing pond, since the anaerobic reactor can be installed with odour control. This alternative is even more attractive when the effluent from the pond can be used for agricultural purposes, since the polishing ponds aim mainly at the removal of pathogenic organisms. Because of its advantages, the post-treatment of effluents from anaerobic reactors through ponds has been common in developing countries.

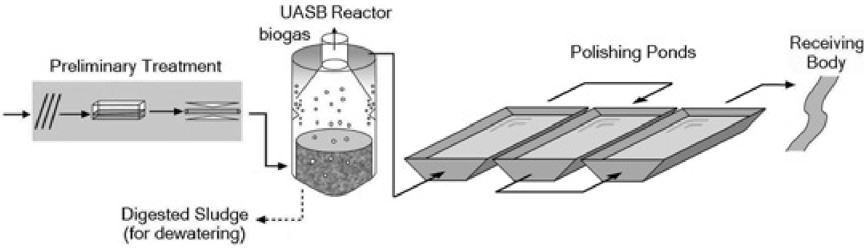


Figure 29.3. Typical configuration of a treatment plant with a UASB reactor and polishing ponds

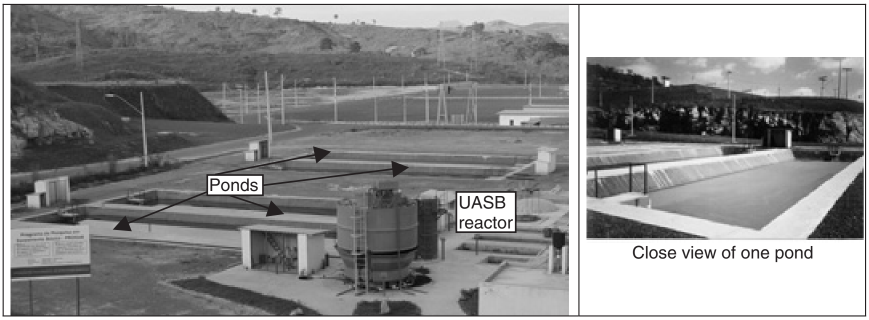


Figure 29.4. View of a UASB reactor followed by four polishing ponds in series (250 inhabitants, Arrudas Experimental WWTP, UFMG/COPASA, Brazil)

29.2.3.2 Typical configuration

Wastewater treatment plants using UASB reactors followed by polishing ponds also have a very simplified flowsheet (Figure 29.3). Besides the preliminary treatment units (screen and grit chamber), the flowsheet comprises the anaerobic treatment unit, the polishing pond (either a single baffled pond or ponds in series) and the dewatering unit for the sludge produced in the UASB reactor. The same considerations made for the UASB reactor+anaerobic filter system are valid here in relation to the characteristics of the anaerobic sludge, which is already thickened and stabilised. Thus, dewatering units using drying beds are also usual in smaller plants. Figure 29.4 illustrates a research unit implemented by the Federal University of Minas Gerais, Brazil.

29.2.3.3 Design criteria

A further discussion on the main design criteria for polishing ponds used as post-treatment units for effluents from UASB reactors is presented in Chapter 19. These criteria were obtained from research at pilot and demonstration scales and from operational results from full-scale plants.

29.2.4 Land disposal

29.2.4.1 Preliminary considerations

Land disposal of sewage is an ancient practice, in which filtration and the action of microorganisms take place. The microorganisms have the capacity to convert the organic matter into simpler compounds. A treated effluent and a revitalised soil are obtained as a final result of this process, since the compounds generated by the microorganisms can be beneficial for the growth of plants and vegetables. The main methods that use the soil for treatment and/or final disposal of sewage are briefly described in Section 4.5.3.

The current section covers only *overland flow* systems as a means of post-treatment of effluents from UASB reactors. A detailed description, the typical configuration and the main design criteria for the other systems can be found in Coraucci Filho *et al.* (2001).

Sewage treatment by the *overland flow* method is the one that presents the least dependence on the types of soil. In this method, the vegetation, associated with the top soil layer, acts as a filter, removing the nutrients and providing conditions for the retention and transformation of the organic matter contained in the sewage. Besides that, it protects the soil against erosion and creates a support layer on which the microorganisms settle. The main mechanisms through which organic matter and solids are removed are biological oxidation, sedimentation and filtration. The main characteristic that differentiates this method from the others is the fact that the effluent flows downward on a slightly inclined vegetated ramp and the remaining water (effluent), which is neither absorbed nor evaporated, is collected downstream and directed for disposal. For more permeable soils, the process is similar to that of irrigation, but with the generation of effluent.

In comparison with other land disposal methods, *overland flow* presents the following characteristics as its main advantages (Coraucci Filho *et al.*, 2001):

- it is appropriate for the treatment of sewage from rural communities and from seasonal industries that generate organic wastewater
- it provides an advanced secondary treatment, with a relatively simple, cheap operation
- the vegetable covering can be reused or commercially used
- it presents the minimum restriction regarding the characteristics of the land, requiring only relatively impermeable soil for its installation and an adequate slope

The disadvantages are:

- the method is limited by the climate, culture tolerance in relation to water and slope of the land
- the application may be limited during wet weather

- the loading rates may be restricted by the growth pattern of the culture
- flat or very steep land is not suitable for this type of treatment

Therefore, the method consists in applying the liquid in the highest part of the ramp. The effluent then drains all over the slope by gravity, where part of it is lost by evapotranspiration and the remaining part is collected on the base of the ramp. Percolation can be insignificant because this system is initially conceived for low-permeability soils. In spite of that, its use has been also reported for soils with medium permeability and impermeable underground (USEPA, 1981). Sewage application is intermittent and the following types of feeding can be adopted: (i) high-pressure sprinklers; (ii) low-pressure sprinklers; and (iii) distribution piping or channels with spaced openings.

Organic matter removal. The effluent produced by overland flow treatment systems usually presents low BOD concentrations. BOD is removed by the biofilm that grows on the surface of the soil and plants. The biofilm can eventually become very thick due to excessive growth. The bacterial cells close to the surface of the soil and plants die due to the lack of oxygen. Different from other attached growth treatment systems, the dead mass of biological solids is not significantly removed from the system, being eventually degraded as time goes by. The complete development of the biofilm after the system start-up may take some time, even 1 year in some cases (WPCF, 1990).

The experiences using the *overland flow* method for the post-treatment of anaerobic effluents have indicated BOD and COD removal efficiencies in the ramps ranging from 48 to 52%, depending on the applied loading rates (Chernicharo *et al.*, 2001a). The overall efficiency of the *anaerobic reactor+ overland flow system* usually amounts from 80 to 90%.

Suspended solids removal. The removal of suspended solids is very efficient in overland flow systems, due to the reduced flow velocities over the ground (between 0.3 and 3 cm/s). The solid material removed works as a substrate for the biofilm, being virtually degraded.

Nitrogen removal. The mechanisms responsible for the removal of nitrogen in overland flow systems include absorption by plants, nitrification/denitrification and ammonia stripping. The plants are capable of removing between 20 and 30% of the total N (e.g. Martel *et al.*, 1980). The removal rate by plants depends on the vegetation culture selected, on the depth and distribution of the roots, on the N loading rate, on the movement of water in the soil and other factors. In general, a type of grass that takes time to develop and presents high nitrogen absorption rates is chosen. It is recommended that the vegetation is periodically harvested, to obtain higher efficiencies.

The losses by ammonia volatilisation are very variable and present a close relation with the evaporation rate and the sewage loading technique. The application of effluents by means of high-pressure sprinklers results in the loss of approximately

7 to 11% of nitrogen in the form of ammonia, while ammonia stripping during the flow of the effluent on the soil is usually lower than 5% (Khalid *et al.*, 1978).

The nitrification process is mainly affected by the amount of oxygen available, the loading rate, the pH and temperature. In mild climates, the limiting factors are the amount of available oxygen and the loading rate. The ratio between the wet and dry periods controls the availability of oxygen in the medium and the time necessary for nitrification. The loading rate is inversely proportional to the ammonia removal, that is, the higher the loading rate, the lower the ammonia removal efficiency. The denitrification process is affected by the degree of treatment of the wastewater applied; once that happens, the higher the concentration of influent BOD to the treatment system, the larger the probability of development of anaerobic conditions and the presence of carbonaceous matter sufficient for denitrification. The BOD_5:N ratio should be approximately 3:1, to favour better removal efficiencies.

The experiences with the use of the *overland flow* process for the post-treatment of anaerobic effluents in Brazil have indicated nitrogen removal efficiencies ranging from 75 to 90%, depending on the temperature, sewage loading rates, and feeding and resting times.

Phosphorus removal. Phosphorus removal in overland flow systems occurs by sedimentation and adsorption in the soil and plants. Removal rates vary between 20 and 60%, although values in the range of 84 to 89% have already been reported (Lee *et al.*, 1976; Martel *et al.*, 1980). Approximately 10% of the phosphorus, corresponding to the insoluble part, is removed in the previous treatment system (in this case, the anaerobic reactor). Except for the component that is incorporated to the biomass, the additional phosphorus removal is minimum in the conventional biological treatment systems, since most of the phosphorus present after the primary treatment is in soluble form. Phosphorus removal in overland flow systems is not usually high, due to the limited contact existing between water and soil, hindering the adsorption process.

Pathogenic organism removal. The survival of pathogenic bacteria in the soil is subject to several factors, including the antagonism of the microflora, moisture content, water retention capacity, organic matter concentration, pH, solar radiation and temperature (Feachem *et al.*, 1983). In overland flow systems, the main microorganism removal mechanisms include: sedimentation; filtration through the biofilm formed on the stems of plants and on the upper layer of the soil; adsorption by soil particles; predation; solar irradiation and desiccation.

In general, and according to experimental results obtained in the past, it can be said that overland flow systems are not efficient regarding the removal of microbial indicators, such as faecal (thermotolerant) coliforms (WPCF, 1990). Peters and Lee (1978) observed a reduction of just one logarithmic unit (or a 90% reduction) in the faecal coliform levels after the application of raw wastewater to an overland flow system. Chernicharo *et al.* (2001a) obtained slightly better results in experiments conducted in a *UASB+overland flow system* treating domestic sewage, in which the removals of faecal coliforms were one log-unit for the UASB reactor and one to two log-units for the overland flow system, resulting in a final effluent with concentrations in the range of 10^4 to 10^5 MPN/100 mL.

The existing knowledge on virus survival in the soil, which is not very comprehensive yet, suggests that the protein nature of these microorganisms favours their adsorption onto the surface of the soil particles (mainly if the soil is of clayey nature), where they are protected from adverse environmental conditions (e.g. Goyal and Gerba, 1979). Schaub *et al.* (1978) observed enteric virus removal rates of up to 85% in overland flow systems.

Helminth eggs remain viable in the soil during long periods, although this varies from species to species. For instance, it is known that *A. lumbricoides* and *T. saginata* eggs can survive in the soil for periods longer than those necessary for plant growth. Vegetable cultures irrigated with wastewater from regions where ascariasis and teniasis are endemic are a potential disease transmission risk (WHO, 1985). Stien and Schwartzbrod (1990) concluded from an experimental study in laboratory scale that the survival time of *Ascaris* eggs in the soil decreases quickly after 20 days from the date of contamination by artificial wastewater. The egg elimination process in the soil depends essentially on two factors: exposure to sunlight and type of soil. Eggs were not found in the vegetable samples after 10 days from wastewater application. The survival time of the eggs in the roots depends on the type of vegetable culture but, in general, it decreases quickly after 45 days from the contamination. Chernicharo *et al.* (2001a) observed no helminth eggs in the final effluent of an overland flow system fed with domestic sewage previously treated in a UASB reactor.

The main characteristics and results of experiments with overland flow systems used for the post-treatment of effluents from anaerobic reactors in Brazil are presented in Table 29.2 (Coraucci Filho *et al.*, 2001).

29.2.4.2 Typical configuration

The typical configuration of a wastewater treatment plant consisting of a UASB reactor and post-treatment by overland flow has a very simple flowsheet (Figure 29.5). Besides the preliminary treatment units (screens and grit chambers), the flowsheet comprises the anaerobic treatment unit, the land treatment system and the dewatering unit for the sludge produced in the UASB reactor. The same considerations made for the systems previously discussed, regarding the characteristics of the anaerobic sludge that is already thickened and stabilised, are also valid here. Dewatering units using drying beds can be used in small-sized plants.

29.2.4.3 Design criteria

The main criteria for the design of overland flow systems applied to the post-treatment of effluents from anaerobic reactors are as follows (adapted from USEPA, 1981; WPCF, 1990 and Coraucci Filho *et al.*, 2001):

Table 29.2. Characteristics and results of experiments with post-treatment systems by overland flow

Parameter	Experiment 1	Experiment 2	Experiment 3	Experiment 4
Type of pre-treatment system	Anaerobic filter	Anaerobic filter	UASB reactor	UASB reactor
Width of the slope (m)	4.2	4.2	3.0	3.0
Length of the slope (m)	35	35	25	25
Gradient of the slope (%)	3.5	3.5	4	4
Hydraulic loading rate (m^3/hour·m)	0.10 and 0.20	0.30 and 0.40	0.20 to 0.60	0.48$^{(a)}$
Feeding period (hour/d)	8	8	8	8
Feeding frequency (d/week)	5	5	5	5
Vegetation cover	*Tifton 85*	*Tifton 85*	*B. humidicola*	*Tifton 85*
Average characteristics of the final effluent				
BOD (mg/L)	30	60	48 to 62	60
COD (mg/L)	116	–	98 to 119	–
TSS (mg/L)	40	–	17 to 57	–
TKN (mg/L)	13	–	–	14 to 18
P (mg/L)	0.5	–	–	–
E. coli (MPN/100 mL)	–	–	–	10^4 to 10^5
Helminth eggs (egg/L)	–	–	0.2	0

(a) Average rate (variable flow over the day, due to the transient hydraulic feeding system to the slopes)
Source: Adapted from Coraucci Filho *et al.* (2001)

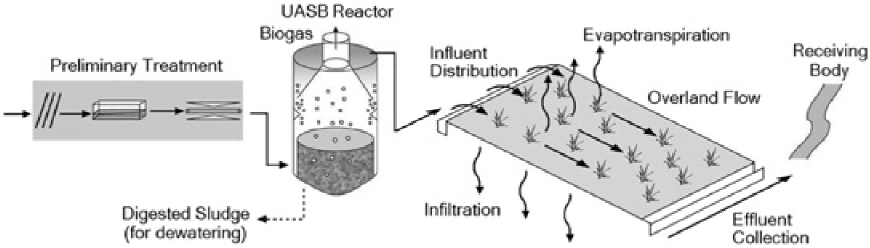


Figure 29.5. Typical configuration of a treatment plant with a UASB reactor and overland flow system

Length of the slope. The length is the longitudinal dimension of the physical surface of the soil, defined by the flowing direction of the effluent. For the low-pressure wastewater application technique, the length of the slope ranges from **30 to 45 m**. Lengths between 45 and 60 m are used for the high-pressure distribution systems.

Ground slope. A ground slope between 1 and 12% is recommended, with an optimal interval between **2 and 8%**. A slope lower than 1% is not recommended, due to the possible formation of pools with sewage and the consequent proliferation of flies. Very high slopes cause the decrease of the flow time and the treatment efficiency, besides favouring the development of erosive processes.

Classification of the soil. The overland flow system was initially developed for soils with low permeability, lower than 15 mm/hour. In spite of that, the system can be used in locations with moderate permeability (**15 to 50 mm/hour**). This is because the void spaces of the soil can be filled with influent solids (clogging) and vegetable growth over time. The permeability can also be changed by soil compaction during the construction of the system.

Operation cycle. The operation is intermittent, with a *feeding period* between **8 and 12 hours/d**, followed by a *dry period* ranging from **16 to 24 hours/d**. Operational cycles with **4 days feeding and 2 days resting (dry)** avoid the propagation of insects.

Hydraulic loading rate. The loading rate is considered the main parameter for the design of the system, defined as the volume applied to the treatment module divided by the loading period in hours. There is a tendency to standardise this parameter, expressing it in terms of unit-width of the module, in $m^3/hour \cdot m$ (Paganini, 1997; Coraucci Filho *et al.*, 2001). This parameter is dependent on the effluent discharge regime, on the sewage pre-treatment level, on the depth and slope of the ground, as well as on the climate. For the post-treatment of anaerobic effluents, the use of loading rates between **0.2 and 0.4 m^3/hour·m** of width of the slope has been usual.

29.2.4.4 Construction aspects

The following main aspects in relation to the construction of overland flow systems should be taken into consideration (USEPA, 1981; WPCF, 1990 and Coraucci Filho *et al.*, 2001):

Storage. It is necessary to build a storage tank sufficient to store the effluent on the days when there is no application. The liquid should be stirred during this period.

Distribution of the sewage. The uniform distribution of the wastewater on the whole width of the ramp is a critical factor in the performance of the system. Its application by either low- or high-pressure sprinklers or by perforated tubes should be started from the top of each slope. The effluent can be distributed by three different techniques (see also Table 29.3):

- *piping with spaced openings:* piping similar to that used for irrigation. The influent is applied under low pressure (2 to 5 N/cm^2). An adjustment should

Table 29.3. Distribution methods: advantages and limitations

Method	Advantage	Limitation
Piping with adjustable openings	• Easy cleaning • Low power consumption • Little generation of aerosols • Smaller safety areas • Easier water balance control	• Possibility of sedimentation inside the tubes • Difficult uniform distribution • Possibility of erosion • Blocking of the orifices
Cut or perforated piping	• Low power consumption • Little generation of aerosols • Smaller safety areas	• Difficulty to ensure uniform distribution • Possibility of erosion • Difficulty to control the water balance • Blocking of the orifices
Bubbling orifice	• Low power consumption • Little generation of aerosols • Smaller safety areas • Less susceptibility to sedimentation	• Difficulty to achieve uniform distribution • Possibility of erosion • Difficulty in maintenance when blocked
Distribution channels	• Low power consumption • Little generation of aerosols • Smaller safety areas • Easy operation	• High initial construction cost • Possibility of erosion • Formation of preferential routes
Low-pressure sprinklers	• More uniform sewage distribution • Low power consumption • Production of less aerosols than high-pressure sprinklers	• Possibility of orifice obstruction by large particles • Generation of aerosols
High-pressure sprinklers	• More uniform sewage distribution • Fewer maintenance requirements	• High power consumption • Larger generation of aerosols • Larger safety areas

Source: Adapted from Araújo (1998)

be made to obtain a uniform distribution. This type of distribution is not recommended for influents with high concentration of suspended solids due to the potential deposition of solids close to the discharge point

- *low-pressure sprinklers:* used with pressures between 5 and 15 N/cm^2. In this type of distribution, the solids can cause the blockage of the sprinkler openings
- *high-pressure sprinklers:* used with pressures between 35 and 60 N/cm^2. This type of distribution covers larger areas than those previously mentioned. As the effluent can reach longer distances, the construction of longer slopes is recommended, to have an appropriate treatment. However, care

should be taken in the use of this type of sprinkler in the case of domestic sewage, in view of the contamination risks by aerosols

Selection of the vegetation. The covering vegetation is essential to the good performance of the system. Perennial and water resistant grasses are those that adapt better to overland flow systems. Their main functions are: protection against erosion, redistribution of the flow (which avoids short circuits), support for microorganisms and removal of nutrients.

Monitoring. The flow, the applied rates, the period and frequency of sewage loading, and the quality of the influent and effluent should be constantly monitored. If there is significant infiltration into the soil, the groundwater shall also be monitored.

Example 29.1

Design an overland flow system acting as post-treatment of the effluent from a UASB reactor, with the following design elements being known:
Data:

- Population: P = 20,000 inhabitants
- Average influent flow: $Q_{av} = 3{,}000\ m^3/d$ ($125\ m^3/hour$)
- Average influent BOD (S_o) = 350 mg/L

The anaerobic reactor was designed in Example 27.2.

Solution:

(a) Calculation of the required area

Design parameters (see Section 29.2.4.3):

- Loading rate: $q_L = 0.35\ m^3/hour{\cdot}m$
- Length of the slope: Z = 35 m
- Feeding periods (feeding hours per day in each slope): $L_p = 8$ hours/d
- Feeding frequency (loading days per week): f = 5 d/week

Net area required:

$$A = \frac{Q_{av} \times Z}{q_L \times L_p} \times \left(\frac{7}{f}\right)$$

$$= \frac{(3{,}000\ m^3/d) \times (35\ m)}{(0.35\ m^3/m{\cdot}hour) \times (8\ hours/d)} \times \left(\frac{7\ d/week}{5\ d/week}\right) = 52{,}500\ m^2$$

Total area (assuming a 20% increment for urbanisation, roads, laboratory, interconnections, etc):

Total area = $1.2 \times 52{,}500\ m^2 = 63{,}000\ m^2$ = (6.3 ha)

Per capita land requirement = $(63{,}000\ m^2)/(20{,}000$ inhabitants$) = 3.2\ m^2/$inhabitant

***Example 29.1* (*Continued*)**

(b) Dimensions of each slope

Number of slopes (initial trial value; this value can be revised, to allow more favourable dimensions and a better adjustment among units in terms of load, daily rest and weekly rest): n = 25

Area of each slope: $A_u = A/n = (52{,}500\ m^2)/25 = 2100\ m^2$

Length of each slope: Z = 35.00 m (previously defined, design parameter)

Width of each slope: $W = A_u/Z = (2100\ m^2)/(35.00\ m) = 60.00\ m$

Gradient of the slopes: s = 4% (design parameter, see Section 29.2.4.3)

Level difference between the upper and the lower parts of each ramp: $H = (Z \cdot s/100) = 35.00\ m \times 4/100 = 1.40\ m$

(c) Operational regime of the slopes

Weekly cycle:

- Number of slopes in rest: $n_r = n \cdot (1 - f/7) = 25 \times (1 - 5/7) = 7$

Daily cycle:

- Number of slopes in operation: $n_{op} = n - n_r = 25 - 7 = 18$
- Number of slopes in loading (at each instant): $n_{load} = n_{op} \cdot L_p/24 = 18 \times 8/24 = 6$
- Number of slopes in resting (at each instant): $n_r = n_{op} - n_{load} = 18 - 6 = 12$

(d) Concentration of effluent BOD

Effluent concentration of the UASB reactor (assuming E_{UASB} efficiency = 75%):
$BOD_{efflUASB} = 350\ mg/L \cdot (1-75/100) = 88\ mg/L$ (see Example 27.2)

Effluent concentration of the overland flow (assuming E = 50%):
$BOD_{effl} = 88\ mg/L \cdot (1-50/100) = 44\ mg/L$

Overall efficiency of the system:
$E = (350 - 44)/350 = 0.87 = 87\%$

29.2.5 Trickling filter

29.2.5.1 Preliminary considerations

As described in Chapter 43, a trickling filter consists basically of a tank filled with a highly permeable material, onto which wastewater is loaded in the form of drops or jets. Wastewater percolates towards the bottom drains, allowing bacterial growth on the surface of the packing material, in the form of a fixed film (biofilm). Wastewater passes over the biofilm, allowing a contact between the microorganisms and the organic matter.

Although the trickling filters (TF) are wastewater treatment systems with great potential and numerous advantages, mainly because of their simplicity and low operational costs, few units have been implemented so far with the purpose of performing the post-treatment of effluents from anaerobic reactors.

The main and innovative purpose of the researches carried out in the past years was to evaluate the applicability and behaviour of the trickling filters, when used for polishing of effluents from anaerobic reactors, particularly UASB reactors. This association (UASB reactor+TF) may contribute significantly to the reduction of the power and operational costs of the treatment plant.

29.2.5.2 Typical configuration

Wastewater treatment plants that use UASB reactors followed by trickling filters present a simple flowsheet (Figure 29.6). Basically, besides the preliminary treatment units (screens and grit chambers), the flowsheet comprises the sequential anaerobic and aerobic biological treatment units (UASB reactor, trickling filter and secondary sedimentation tank), as well as the dewatering unit. Notice that, in this configuration, the excess aerobic sludge removed from the secondary sedimentation tank is returned to the UASB reactor for thickening and anaerobic digestion. Therefore, with this flowsheet, primary sedimentation tanks and separate units for thickening and anaerobic digestion of the excess aerobic sludge are not required, different from the conventional treatment plants that use trickling filters (Figure 43.3).

The sludge wasted from UASB reactors is already thickened and stabilised, and can be sent directly for dewatering and final disposal. Drying beds have been frequently used for dewatering of the sludge in small-sized plants.

An innovative and compact configuration of this treatment system was developed by the Federal University of Minas Gerais (Brazil) for sewage treatment in

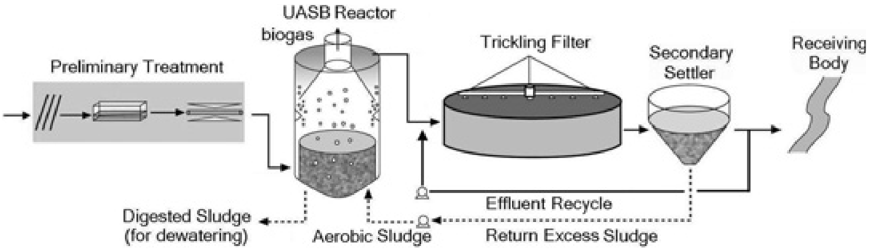


Figure 29.6. Typical configuration of a treatment plant with UASB reactor and trickling filter

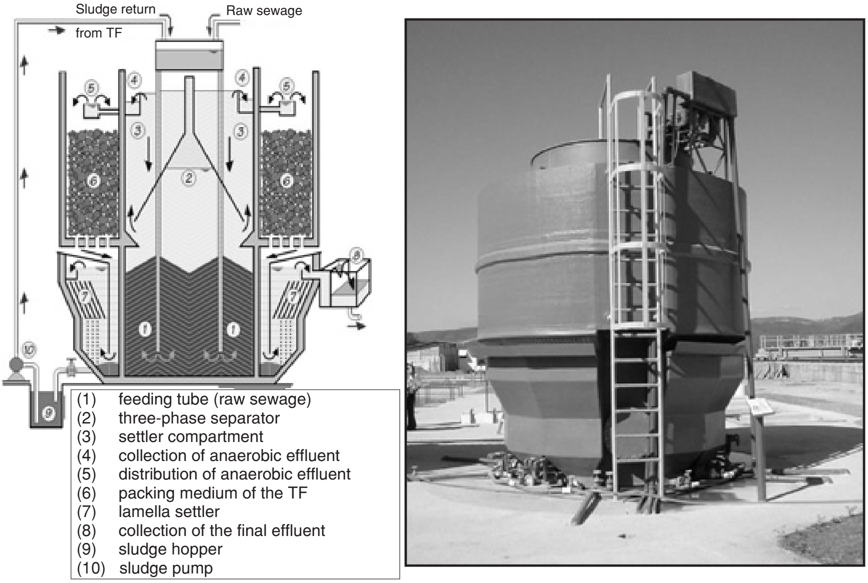


Figure 29.7. Compact configuration of a UASB reactor and trickling filter system (module for 500 inhabitants, Arrudas Experimental WWTP, Brazil)

small communities. The compact system comprises the three main units (UASB reactor and TF reaction and settling compartments) in a single treatment module, as illustrated in Figure 29.7.

29.2.5.3 Design criteria

A detailed discussion on the main design criteria for trickling filters used as post-treatment units for effluents from UASB reactors is presented in Chapter 43. These criteria were obtained from pilot-scale research and operational results from full-scale plants.

29.2.6 Submerged aerated biofilter

29.2.6.1 Preliminary considerations

As described in Chapter 45, a submerged aerated biofilter consists of a tank filled with porous material, through which sewage and air flow permanently. In almost all the existing processes, the porous medium is maintained totally submerged by the hydraulic flow. The biofilters are characterised as three-phase reactors consisting of:

- solid phase: consisting of the support medium and colonies of microorganisms present in the form of a biofilm

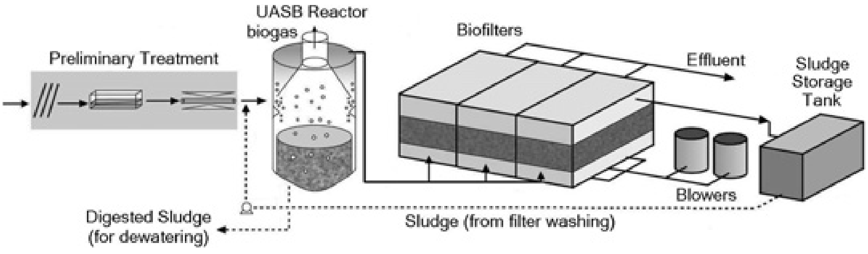


Figure 29.8. Typical configuration of a treatment plant with UASB reactor and submerged aerated biofilters

- liquid phase: consisting of the liquid in permanent flow through the porous medium
- gas phase: formed by artificial aeration and, in a reduced scale, by the gases derived from the biological activity

Several small wastewater treatment plants with UASB reactors followed by submerged aerated biofilters filled with granular material, without secondary sedimentation tanks, and with backwashing removal of sludge from the biofilter, are already in operation in Brazil. Most of the plants have been designed for organic matter removal (effluent BOD < 30 mg/L), without nitrification.

29.2.6.2 Typical configuration

Sewage treatment plants that use UASB reactors followed by submerged aerated biofilters also present a simple flowsheet (Figure 29.8). Besides the preliminary treatment units (screens and grit chambers), the flowsheet comprises the sequential anaerobic and aerobic biological treatment units (UASB reactor and submerged aerated biofilter), as well as the aeration, sludge accumulation and dewatering units. Also in this configuration, the excess aerobic sludge removed from the biofilter is returned to the UASB reactor for thickening and anaerobic digestion. Therefore, with this flowsheet, primary sedimentation tanks and separate units for thickening and anaerobic digestion of the excess aerobic sludge are avoided, different from the conventional treatment plants that use submerged aerated biofilters (Figure 45.1).

The sludge wasted from the UASB reactor is already thickened and stabilised, and can be directly sent for dewatering and final disposal. Sludge drying beds have been frequently used in small-sized plants.

29.2.6.3 Design criteria

A detailed discussion on the main design criteria for submerged aerated biofilters used as post-treatment units for effluents from UASB reactors is presented in

Chapter 45. These criteria were obtained from pilot-scale research and operational results from full-scale plants.

29.2.7 Activated sludge

29.2.7.1 Preliminary considerations

The essence of the *continuous flow* activated sludge process is the integration of the aeration tank (aerobic biological reactor), secondary sedimentation tank and sludge recirculation line. These three components are maintained in the alternative of activated sludge systems acting as post-treatment of effluents from anaerobic reactors.

The *intermittent flow* activated sludge system (sequencing batch reactors) can also be adopted as post-treatment, requiring, in this case, only the tanks that alternate in the functions of reaction and sedimentation.

A discussion on the applicability, advantages and disadvantages of this configuration is presented in Chapter 30, in which this alternative is compared to the usual alternatives (conventional activated sludge, extended aeration and sequencing batch reactors).

29.2.7.2 Typical configuration

When the activated sludge system acts as post-treatment of anaerobic effluents, the anaerobic reactor is used instead of the primary sedimentation tank (which is an integral part of the conventional activated sludge system). The aerobic sludge is recirculated in the usual manner, that is, from the bottom of the secondary sedimentation tank to the entrance of the aerobic reactor (aeration tank).

The excess aerobic sludge generated in the activated sludge stage, not yet stabilised, is sent to the UASB reactor, where it undergoes thickening and digestion, together with the anaerobic sludge. As the return flow of the excess aerobic sludge is very low compared with the influent flow, there are no operational disturbances in the UASB reactor. The sludge treatment is largely simplified: there is no need for

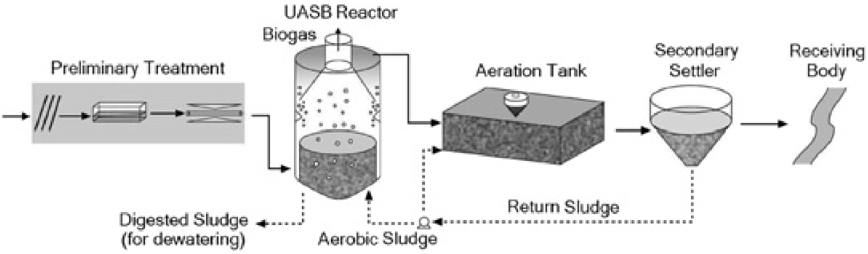


Figure 29.9. Typical configuration of a treatment plant with UASB reactor and activated sludge system

separate thickeners and digesters, and just the dewatering stage is necessary. The mixed sludge removed from the anaerobic reactor is digested, has solids concentrations similar to those from sludge thickeners and presents good dewaterability. Figure 29.9 presents the flowsheet of this configuration.

29.2.7.3 Design criteria

A detailed discussion on the main design criteria of activated sludge systems acting as post-treatment of effluents from UASB reactors is presented in Chapter 38. These criteria were obtained from pilot-scale research and operational results from full-scale plants.

Part Four

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