Environmental Sanitation, Wastewater Treatment and Disposal

2nd Edition

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বাংলাদেশ
বিশ্ববিদ্যালয় মঞ্জুরী কমিশন

Environmental Sanitation, Wastewater Treatment and Disposal

First Published : November 2013, Agrahayan 1420

Second Published : December 2014, Poush 1421

Published by : University Grants Commission of Bangladesh
29/1, Agargaon, Dhaka-1207, Bangladesh
Phone : 88-02-8181279, 8181618
E-mail : chairmanugc@yahoo.com
Web : www.ugc.gov.bd

UGC Publication No. : 158

ISBN : 978-984-8920-16-9

Printing layout & Supervised by : Md. Kamal Hossain, Deputy Director (P & I)
Md. Liakat Hossain, Assistant Director (Pub)

Cover and Design : Tahmid Ritu, Abdullah Al-Muyeed

Drawing : Tahmid Ritu, Tanjil Hasan

Printing : Quality Printers
6, Rozoni Boshak Lane,
Armanitola, Dhaka-1100
Phone :

Price : BDT : 470.00
US $: 10.00

Environmental Sanitation, Wastewater Treatment and Disposal by Tanveer Ferdous Saeed, Abdullah Al-Muyeed & Tanvir Ahmed, Published by University Grants Commission of Bangladesh, Dhaka.

ISBN : 978-984-8920-16-9

Cover Picture : Internet

Dedicated to

Our Parents

Foreword

It is my earnest privilege to present this book titled Environmental Sanitation, Wastewater Treatment and Disposal from the University Grants Commission, Bangladesh (UGC). The explosion of population has exerted undue pressure on the environment and its resources, a problem which is more severe for developing countries due to their limited resources in providing adequate sanitation, handling of wastes and wastewater generated and preventing pollution. This book offers primarily theoretical concepts in environmental sanitation, treatment of wastewater and water pollution, that form a core part of the curricula for civil/civil and environmental engineering discipline at the undergraduate and postgraduate levels. A comprehensive textbook to address these issues in the undergraduate and postgraduate curricula was critically necessary, and this book certainly fills up that void to some extent. With the emergence of civil and environmental engineering disciplines in different universities around the country, I believe that this book will receive its due credit. Apart from that, postgraduate research students in environmental engineering, and practicing environmental engineers in relevant fields will also find the contents of this book useful as reference materials.

The authors of the book are three young professionals in the field of environmental engineering working as faculty members in different universities. I wholeheartedly admire their courage and enthusiasm in embarking on this tedious journey to write this textbook. The University Grants Commission of Bangladesh has a long history for promoting research and publication works from scholars in Bangladesh and therefore, is pleased to promote such an endeavour from the young authors. From my point of view, the real beneficiaries are the thousands of students studying in this field in various universities in the country. It is my belief that the students as well as the respected peers will appreciate their effort.

The University Grants Commission of Bangladesh is happy to publish the book. I expect that the book will be of immense benefit to the students, researchers and teachers of relevant fields. I express my sincere thanks to the officers and staff of the Publication Section of the UGC for their strenuous efforts in publishing the book.

Dhaka
September 2013

Prof. Dr. A K Azad Chowdhury
Chairman (State Minister)
University Grants Commission of Bangladesh

Preface

This text book has been designed to cover the curricula of undergraduate and postgraduate environmental engineering and related courses that are generally offered by the Civil Engineering Departments of the technological universities in Bangladesh. More specifically, this book covers several sub divisions of environmental engineering, such as sanitation engineering, wastewater (municipal and industrial) treatment technologies, water pollution control that are usually taught at the 3rd or 4th level of the undergraduate programs or as core courses in the postgraduate level.

This book is comprised of 12 chapters covering a wide range of topics related to sanitation, wastewater treatment, disposal and water pollution. Chapter 1 gives a brief description on the importance of sanitation, wastewater treatment and safe disposal, with an aim to provide the reader a scope to think about the necessity of sustainable environmental solutions for the country. Chapter 2 describes engineered sanitation technologies in context of Bangladesh. Chapters 3-7 describe municipal wastewater transportation into treatment plants, and different treatment stages for achieving safe disposal criteria. Several design examples have also been included in these chapters to allow the students, as well as the professionals for developing a clear understanding of the design process of treatment plants. Chapter 8 describes different approaches to provide treatment of industrial wastewater. Two case studies on the treatment of tannery and textile wastewater (i.e. major sources of water pollution in Bangladesh) have been included in this chapter, with an aim to giving the reader a conceptual view for preventing water pollution from such sectors in Bangladesh. Chapter 9 includes a brief description on sludge treatment (for safe disposal), generated from wastewater treatment plants. Chapter 10 focuses on low-cost natural treatment technologies, such as ponds and constructed wetlands. Chapter 11 gives a brief description on the factors associated with water pollution. Several case studies regarding water pollution analysis and control have been incorporated in this chapter which will allow the reader to acquire an in-depth knowledge on such factors. Finally Chapter 12 gives brief information and required guidelines for the construction, operation and maintenance of decentralized wastewater treatment systems.

During the writing phase, we received feedbacks and suggestions from national and international experts and professionals. We are greatly indebted to them for their useful comments, that helped us enrich the quality and content of this book. We are grateful to the University Grants Commission (UGC) of Bangladesh, for making the publication of this book possible. Despite our wholehearted effort there may be some inaccuracies or mistakes for which we are wholly responsible. We would be extremely grateful if you let us know by e-mail when you detect any such errors in the text.

We are also indebted to our colleagues, friends and family members who supported us during the stressful writing period. Finally, we would like to express our gratitude to our parents for showing us the light of knowledge throughout our life. This book is dedicated to our parents.

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List of Abbreviations

ABR	Anaerobic Baffled Reactor
ANAMMOX	Anaerobic Ammonium Oxidation
BABE	Bio-Augmentation Batch Enhanced
BCFS	Biological Chemical Phosphate Nitrogen Removal
BOD	Biochemical Oxygen Demand
CANON	Completely Autotrophic Nitrogen Removal Over Nitrite
CBFP	Continuous Belt Filter Presses
CETP	Central Effluent Treatment Plant
COD	Chemical Oxygen Demand
CSTR	Continuous Flow Stirred Tank Reactor
DAF	Dissolved Air Floatation
DCC	Dhaka City Corporation
DEAMMOX	Denitrifying Ammonium Oxidation
DO	Dissolved Oxygen
DoE	Department of Environment
DPAO	Denitrifying Phosphate Accumulating Organisms
DPHE	Department of Public Health Engineering
DWASA	Dhaka Water Supply and Sewerage Authority
DWWT	Decentralized Wastewater Treatment
EQS	Effluent Quality Standards
ETP	Effluent Treatment Plant
FSS	Fixed Suspended Solid
GoB	Government of Bangladesh
HCB	Hexa Chloro Benzene
HRT	Hydraulic Retention Time
HSSF	Horizontal Sub Surface Flow
KW	Kilo Watt
LDPE	Low Density Poly Ethylene
MDG	Millennium Development Goal
ML	Million Liter
MLD	Million Liter Per Day
MLSS	Mixed Liquor Suspended Solid
MLVSS	Mixed Liquor Volatile Suspended Solid
NTU	Nephelometric Turbidity Unit
PAO	Phosphate Accumulating Organisms
PCB	Polychlorinated Biphenyl
PE	Population Equivalent

PF	Plug Flow
POP	Persisting Organic Pollutants
RAS	Returned Activated Sludge
RBC	Rotating Biological Contactors
ROEC	Reed Odourless Earth Closet
SBR	Sequencing Batch Reactor
SDI	Sludge Density Index
SHARON	Single Reactor for High Activity Ammonium Removal Over Nitrite
SOR	Surface Overflow Rate
SS	Sanitation Secretariat
STP	Sewerage Treatment Plant
SVI	Sludge Volume Index
TDS	Total Dissolved Solid
TKN	Total Kjeldahl Nitrogen
TN	Total Nitrogen
TP	Total Phosphorus
TS	Total Solids
TSS	Total Suspended Solids
UASB	Upflow Anaerobic Sludge Blanket
UNEP	United Nations Environment Programme
UPVC	Unplasticized Poly Vinyl Chloride
VIP	Ventilated Improved Pit
VSS	Volatile Suspended Solid
VSSF	Vertical Sub Surface Flow
WAS	Waste Activated Sludge
WSSD	Water Supply and Sanitation Decade
WWTP	Wastewater Treatment Plant

Chapter 1

Introduction

Environmental engineering is concerned with the protection of the environment and natural resources. Environmental engineering imparts special attention on biological, chemical and physical reactions of contaminants in air, land and water, and focuses on developing improved technologies for minimizing the adverse impacts of such components on the environment.

The people who are professionally involved with environmental engineering discipline, are referred as environmental engineers. However, the descriptive title "environmental engineer" was not used until the 1960s, when academic programs in engineering and public health schools broadened their scope, and required a more accurate title to describe the people practising this discipline. The roots of this profession, however, go back as far as recorded history reaching into several major disciplines including civil engineering, sanitary and wastewater engineering, public health, ecology, chemistry, and meteorology.

Sanitation, wastewater treatment and impact of pollutants on aquatic environments – all interrelated subdivisions within the environmental engineering discipline, are subjected to intensive research worldwide due to their rigorous impact on environmental sustainability. In Bangladesh, their impact on environment is severe, and currently gaining attention in both government and public sectors. As such, this chapter provides a brief summary on these topics, with an aim to provide the reader a scope to think about the necessity of sustainable environmental solutions for the country. Section 1.1 provides current scenario of sanitation practices in Bangladesh, and their impact on the environment. Sections 1.2-1.4 deliver the basic knowledge on wastewater treatment theory, and fate of major pollutants in aquatic environments. Finally, section 1.5 assists the readers, in terms of understanding the scope and contents of this book.

1.1 Environmental Sanitation

The term environmental sanitation has received numerous interpretations in different countries. If restricted to narrower perspectives, such term refers to the safe disposal of human wastes. However, in many cases, environmental sanitation includes safe water supply, and disposal of human refuse in a particular community. In recent years, the concept of environmental sanitation also extended towards controlling the elements, that generally affect the environment, or human health.

Sanitation often does not receive rigorous attention by the policy makers of developing countries, or aid donors (Ahmed and Rahman, 2000). For such reasons, the global scenario of sanitation coverage is not satisfactory. More than 2.5 billion people, representing almost half of the world population, do not have access to adequate or improved sanitation facilities (WHO, 2008). Figure 1.1 depicts the proportion of the population across the world, without sanitation access. As observed in Figure 1.1, the proportion of population without sanitation access was very high in South Asia.

In South Asia, open defecation practice (by the population of the country) is still more than 50% in India, Nepal and Pakistan (Figure. 1.2). Approximately 668, 50 and 14 million people in India, Pakistan and Nepal respectively, practice open defecation. In Bangladesh, about 18 million (11% of the population) people practice open defecation; 37% people use unimproved sanitary means, and only 36% people use improved means for excreta disposal.

Table 1.1 provides a brief comparison of sanitation progress (i.e. hygienic latrine coverage) in Bangladesh, as reported by Sanitation Secretariat (SS) of Department of Public Health Engineering (DPHE), and other organizations.

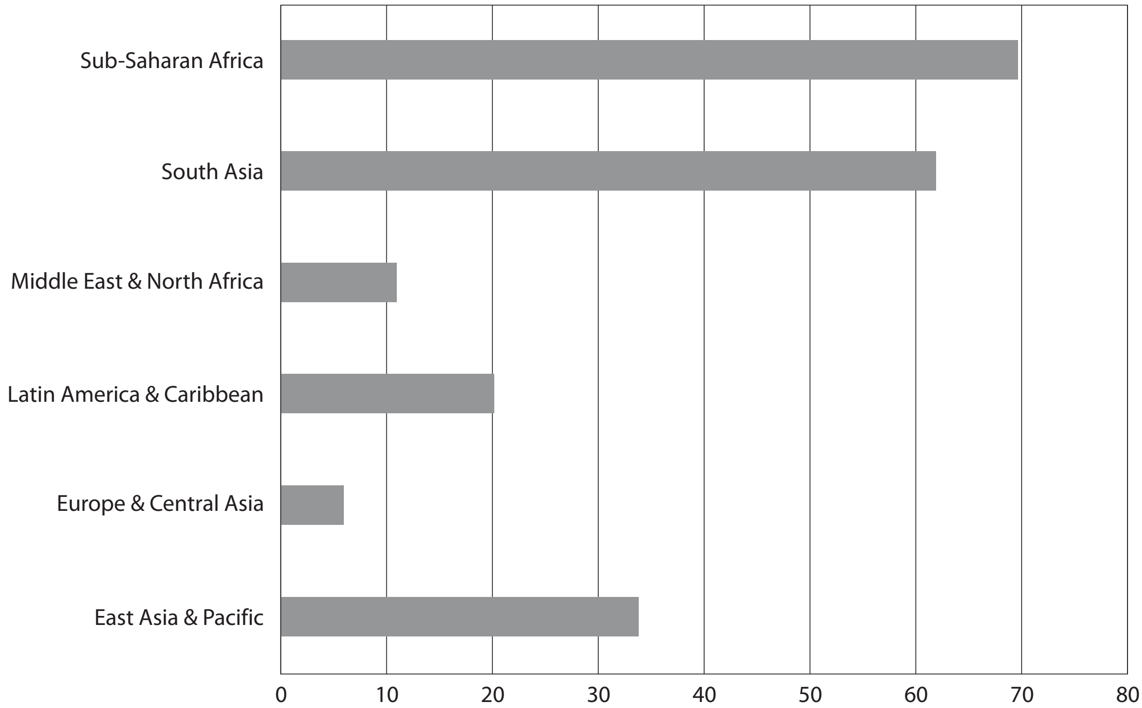

Figure 1.1 **Population distribution (%) without sanitation access throughout the world in the year 2010 (GMR, 2013).**

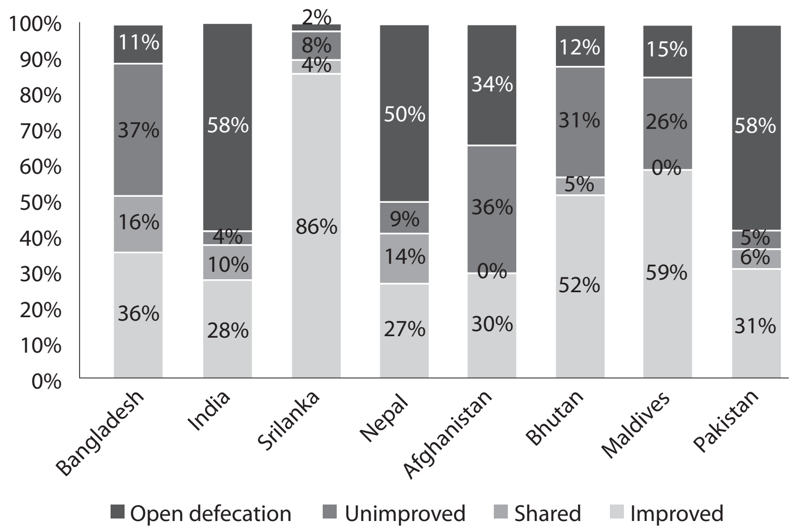

Figure 1.2 **Sanitation coverage of South Asia in 2006.**

It is evident from Table 1.1 that a large number of population in Bangladesh are associated with unsanitary systems, such as unhygienic latrine and open defecation. Even the households in rural and slum areas, where sanitary facilities are available, children rarely use them due to lack of health education. In addition, the generated faecal sludge from these latrines are often discharged into open water bodies, thereby posing severe environmental

Table 1.1 Comparison of Sanitation progress reported by Sanitation Secretariat (SS) and other organizations.

Organization	Reporting period	% of household using hygienic latrine	% of household using hygienic latrine + latrine without water seal	% of household using unhygienic latrine	% of household practicing open defecation
NGO Forum	2006	33.2	69.7	24.8	5.5
SS	2006	81.6	-	-	-
UNICEF	2006	39.2	73.1	19.4	7.5
BRAC	2007	31.1	-	-	-
SS	2007	85.6	-	-	-
Dishari	2007	60.6	-	-	-
JMP	2011	55	27	14	4

threats (Rahman and Bache, 1993). According to Rahman and Bache (1993), slum wastewater (in Bangladesh) comprises substantial pollutants (eg. BOD_5 = 400-1700 mg/L, suspended solids = 500-3000 mg/L), which can pose severe threats to the surrounding environment. Such findings indicate the necessity of adequate wastewater transportation and treatment systems, prior to environmental disposal in Bangladesh.

1.2 Wastewater Engineering

Wastewater, also referred as sewage, is produced from household wastes, human and animal wastes, industrial wastes, storm water runoff, and infiltration of groundwater. Wastewater possesses certain physical, chemical and biological properties, depending on water usage and discharge characteristics of a community. Physical properties of wastewater include color, temperature and solid contents. The chemical components of wastewater include organic and inorganic materials. Organic components comprise of carbohydrates, fats, oil, grease, proteins, surfactants, pesticides, agricultural, and toxic chemicals. Inorganics often include heavy metals, nutrients (nitrogen and phosphorus), pH, alkalinity, chlorides, sulphur, etc. Gases, for example carbon dioxide, nitrogen, oxygen, methane, and hydrogen sulphide can also be present in wastewater. Different types of microorganisms such as bacteria, fungi, protozoa, viruses, microscopic plants, and animals are also found in wastewater. Some of these organism species can cause severe impact to human health.

Since wastewater contains many harmful elements, it requires treatment, prior to environmental disposal (in order to protect the environment). Such objectives are often achieved by wastewater treatment plants, that speed up the degradation of pollutants, resulting in end products that do not interfere with the natural environment of receiving water bodies. The treatment of wastewater can be classified into three categories: (a) preliminary; (b) secondary; and (c) tertiary treatment process. Each category has specific functions, for removing particular constituents of wastewater.

Figure 1.3 describes different stages of a typical wastewater treatment process.

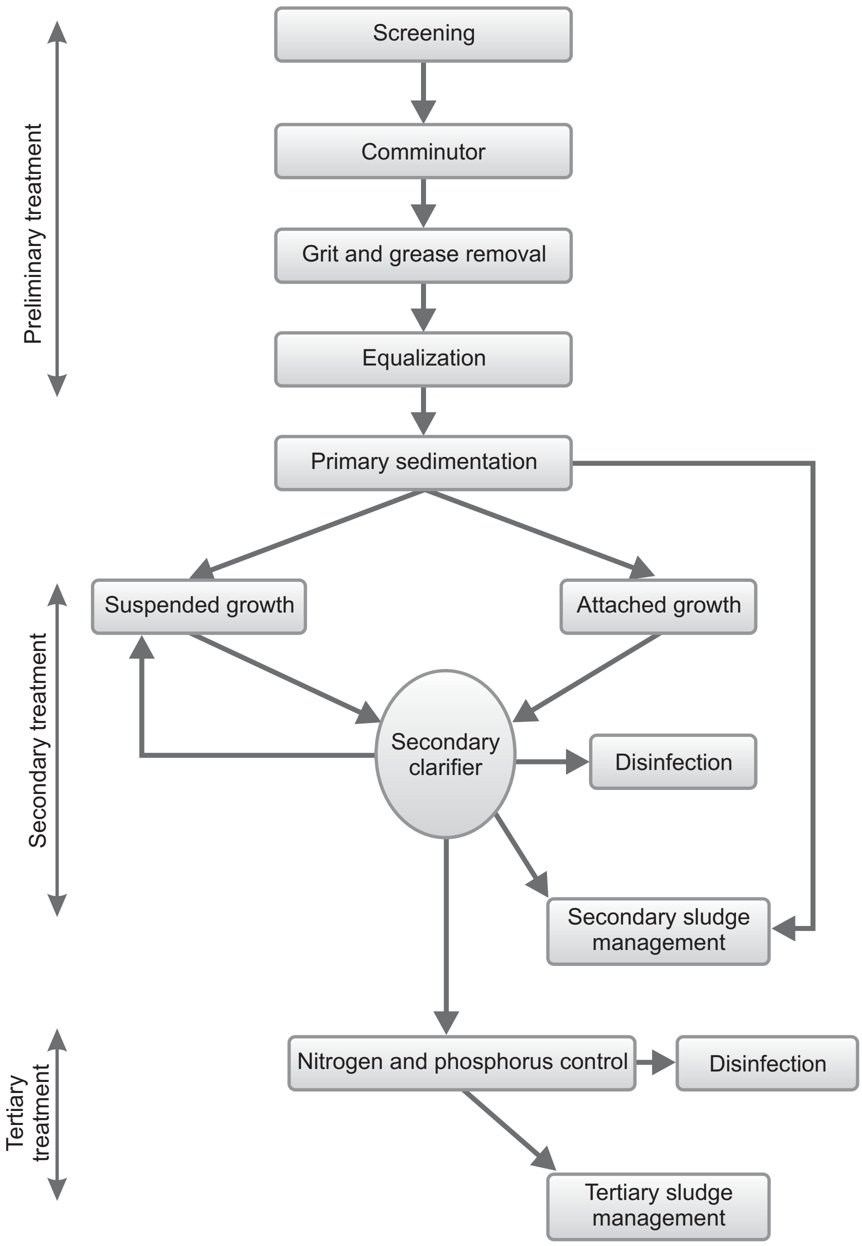

Figure 1.3 **Flow chart of wastewater treatment process.**

In Bangladesh, the only sewerage system has been developed and maintained by Dhaka Water Supply and Sewerage Authority (DWASA) to serve the inhabitants of the nation's capital. However, the sewerage network has a very small coverage (only 18% of the city, and 25% of the population) and the treatment plant of DWASA is not adequate for the treatment of sewage to a satisfactory level. The overloading of the existing treatment plant and uncontrolled discharge of wastewater into the surrounding water bodies are severely degrading the aquatic environment of Bangladesh.

1.3 Wastewater Characteristics: Physical, Chemical and Biological

The pollutants of wastewater are generally classified into three categories: (a) physical; (b) chemical; and (c) biological. Some common wastewater pollutants and their sources have been illustrated in Table 1.2.

Table 1.2 **Classification and sources of common wastewater pollutants.**

Properties	Parameter	Sources
Physical	Temperature	Domestic, and industrial wastes.
	Solids	Domestic, industrial wastes, and infiltration.
	Color	Domestic, industrial wastes, and decay of organic compounds.
	Odor	Industrial wastes.
Chemical (organic)	Carbohydrates, proteins, fats, oils, and grease	Domestic, commercial, and industrial wastes.
	Pesticides	Agricultural wastes.
	Phenols	Industrial wastes.
Chemical (inorganic)	Alkalinity, chlorides	Domestic wastes, and infiltration.
	pH	Domestic, commercial, and industrial wastes.
	Nitrogen	Domestic and agricultural wastes.
	Phosphorus	Domestic, commercial, and industrial wastes.
	Heavy metals	Industrial wastes.
	Sulfur	Domestic, commercial, and industrial wastes.
Chemical (priority pollutants)	Surfactants	Domestic, commercial, and industrial wastes.
	Volatile organic compounds	Domestic, commercial, and industrial wastes.
Biological	Bacteria	Domestic wastes, surface water infiltration, and treatment plants.
	Viruses	Domestic wastes.
	Animals, plants	Open watercourses, and treatment plants.

1.4 Fate of Major Environmental Contaminants

The efficiency of treatment plants is associated with the necessity of acquiring profound knowledge on the physico-chemical and biological removal mechanisms, such as: degradation of contaminants. Decomposition or biodegradation may take place (in presence of microorganisms) in one of the two distinctly different ways: aerobic (using free oxygen), and anaerobic (in the absence of free oxygen).

A schematic representation of the aerobic cycle for carbon, nitrogen, sulfur and phosphorus has been illustrated in Figure 1.4. This figure shows only the basic phenomena and simplifies the actual steps and mechanisms. Carbon dioxide and water are two end products of aerobic decomposition of organic materials. Both are stable, low in energy and are used by plants during photosynthesis. Phosphorus, sulfur and nitrogen compounds are often included in the general discussion of decomposition, because the breakdown and release of these compounds during decomposition of organic matter can contribute to water quality problems. In aerobic environments, sulfur compounds are oxidized to sulfate ion (SO_4^{2-}) and phosphorus is oxidized to phosphate (PO_4^{3-}). Any phosphate not rapidly taken up by microorganisms is bound by physical or chemical attraction to suspended sediments and metal ions, making it unavailable to most aquatic organisms. Nitrogen, another major pollutant, is oxidized through a series of steps in the progression:

$$\textit{Organic Nitrogen} \rightarrow NH_4^+ \ (\textit{ammonium}) \rightarrow NO_2^- \ (\textit{nitrite}) \rightarrow NO_3^- \ (\textit{nitrate})$$

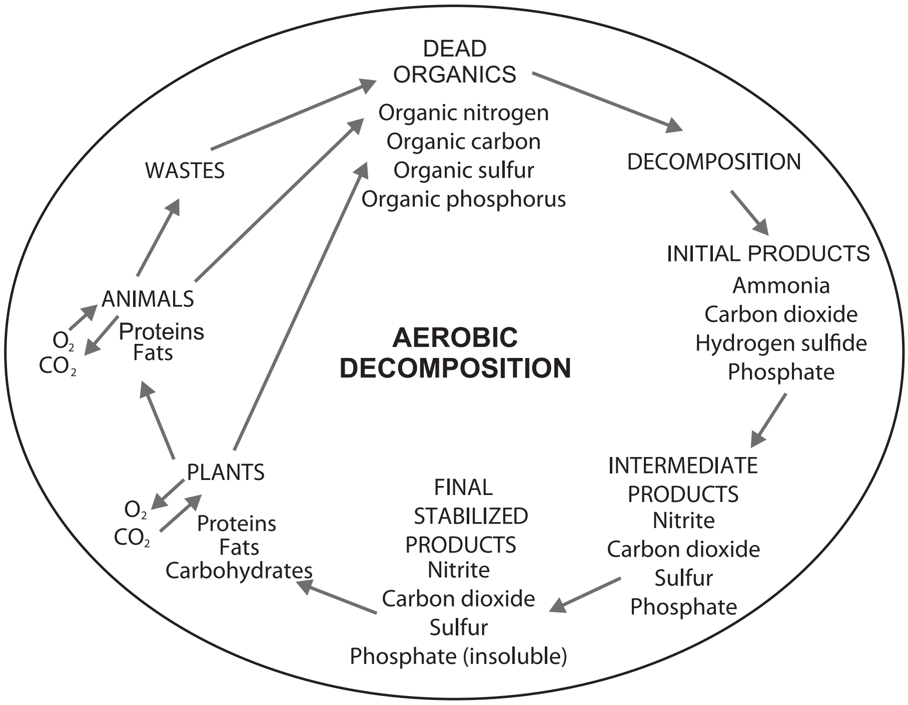

Figure 1.4 **Aerobic carbon, nitrogen, phosphorus, and sulfur cycles (De Camp 1963).**

Anaerobic decomposition is usually performed in absence of oxygen by a completely different set of microorganisms. Such microorganisms include obligate anaerobes (that can only survive in anaerobic environments) and facultative anaerobes (which can survive in aerobic or anaerobic environments).

Figure 1.5 represents a schematic of anaerobic decomposition. Note that the left half of the cycle is identical to the aerobic cycle. Many of the end products of anaerobic decomposition are biologically unstable. Methane (CH_4) for example, is a high-energy gas commonly called "marsh gas" (or "natural gas" when burned as fuel); methane can be oxidized as an energy source (food) by a wide variety of aerobic bacteria. Sulfur is anaerobically biodegraded to hydrogen sulfide (H_2S) which can be employed as an energy source by aerobic bacteria. Phosphates released during anaerobic decomposition are soluble in water and do not bind to metal ions or sediments. Soluble phosphate is easily taken up by plants and used as a nutrient.

Biologists often speak of certain compounds as hydrogen acceptors. When energy is released from high-energy compounds, a C = H or N=H bond is broken and the freed hydrogen must be attached somewhere. In aerobic decomposition, oxygen serves as hydrogen acceptor and forms water. In anaerobic decomposition, oxygen is not available. The next preferred hydrogen acceptor is nitrate (NO_3^-) or nitrite (NO_2^-), forming ammonia (NH_3). If no appropriate nitrogen compound is available, sulfate (SO_4^{2-}) accepts hydrogen to form sulfur (S) and H_2S, the compound responsible for toxic effects.

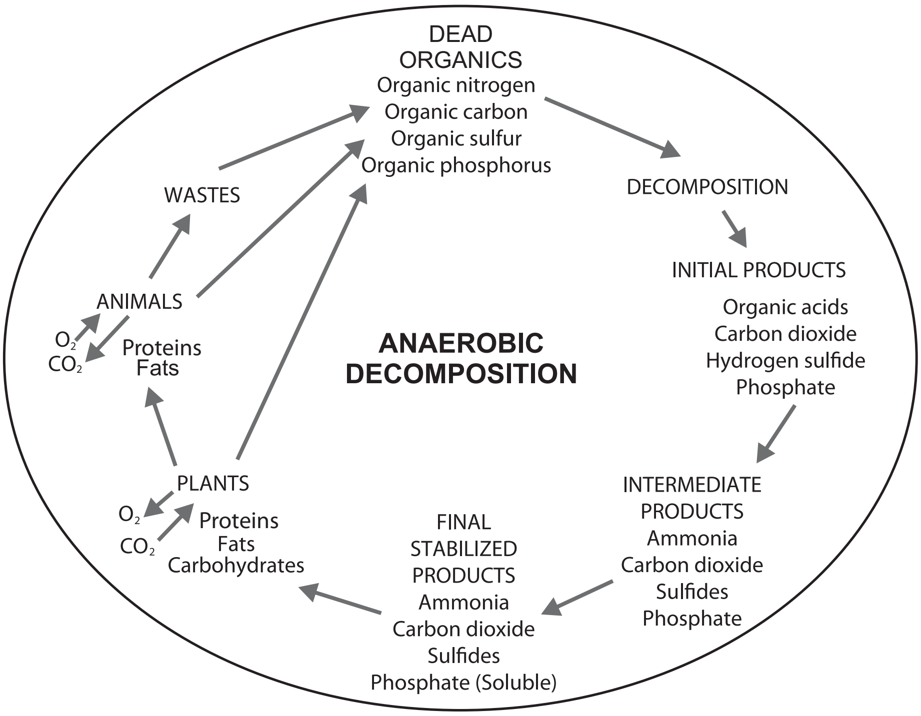

Figure 1.5 **Anaerobic carbon, nitrogen, phosphorus, and sulfur cycles (De Camp 1963).**

The removal mechanisms of various heavy metals (from wastewater) such as: arsenic (As), cadmium (Cd), lead (Pb), mercury (Hg), zinc (Zn) etc. are achieved via chemical precipitation, ion-exchange, adsorption, coagulation, electro-coagulation and reverse osmosis. In precipitation process, metal salts are converted into insoluble form, by adding correct anions; the process requires addition of other chemicals. An ion exchanger is a solid that is capable of exchanging either cations or anions from the surrounding materials; commonly used matrices for ion exchange are synthetic organic ion exchange resins. Electro-coagulation is an electrochemical method that uses an electrical current to remove metals from solution. The contaminants present in wastewater are maintained in solution by electrical charges. When these ions and other charged particles are neutralized with ions of opposite electrical charges provided by electro-coagulation system, they become destabilized and precipitate in a stable form. Reverse osmosis involves the use of semi-permeable membranes for the recovery of metal ions from wastewater.

1.5 Organization of the Book

Tsections 1.1-1.4 illustrate that that sanitation and wastewater treatment must be integrated properly, to protect the current degradation of aquatic environment in Bangladesh. Considering such factors, this book has been written focusing on environmental sanitation, wastewater treatment and disposal. The book consists of 12 chapters. Chapter 2 describes engineered sanitation technologies in context of Bangladesh. Chapters 3-7 describe wastewater transportation, and different treatment stages (i.e. preliminary, primary, secondary, and tertiary process) for municipal wastewater treatment. Chapter 8 focuses on industrial wastewater disposal information in Bangladesh; two case studies such as: chemical constituents of tannery and textile effluents (two major industrial pollution sources in Bangladesh), along with possible treatment options have also been provided. Chapter 9 provides information on available technologies for sludge treatment, generated from conventional treatment plants. Chapter 10 provides brief information on natural treatment technologies. Chapter 11 denotes the impact of major pollutants, on the aquatic environments of Bangladesh. Finally, Chapter 12 provides information and design process of decentralized wastewater treatment systems. Figure 1.6 describes a block diagram, illustrating the structure, and organization of the book.

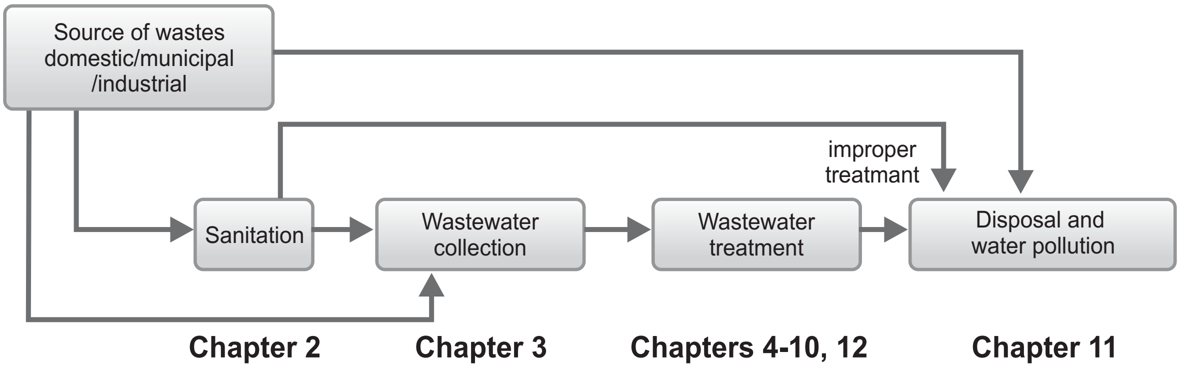

Figure 1.6 **Block diagram of book organization.**

Questions

1. Write short notes on: (a) environmental engineering; and (b) environmental engineers.
2. Explain the current scenario of sanitation systems in Bangladesh.
3. Provide a block diagram of wastewater treatment stages.
4. Distinguish between aerobic and anaerobic fate of major pollutants in environment.

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Chapter 2

Sanitation

2.1 Introduction

Adequate sanitation is defined as the safe management of human excreta and includes both 'hardware' (sanitation technologies, such as toilets and hygienic latrines) and 'software' (hygiene promotion, such as hand washing with soap). Inadequate provision of sanitation is directly and indirectly related to the communicable diseases, health risk, poor health and environmental pollution. Inadequate sanitation facilities and poor hygiene practice can cause continued transmission of disease through several routes even after safe water supply is secured at the point of use. If we consider, faeces as the source of pollution in the environment, there are several pathways (e.g. ingestion through drinking water, food and utensils, host intermediates, direct contact) through which contaminants can reach humans and cause diseases(Figure 2.1). Sanitation is considered the first line of defense (primary intervention) for the protection of the quality of the environment and control of all diseases. Sanitation facilities, in the form of proper treatment and disposal of all forms of wastes both liquid and solid including human excreta, interrupt the transmission of faecal pollution to the environment (air, water and soil). Sanitation include all off-site or on-site waste/ wastewater treatment options the goal of which is to retain and confine materials of faecal origin for a very long time, stabilize the organic matter through aerobic or anaerobic processes eventually causing the destruction of pathogens.

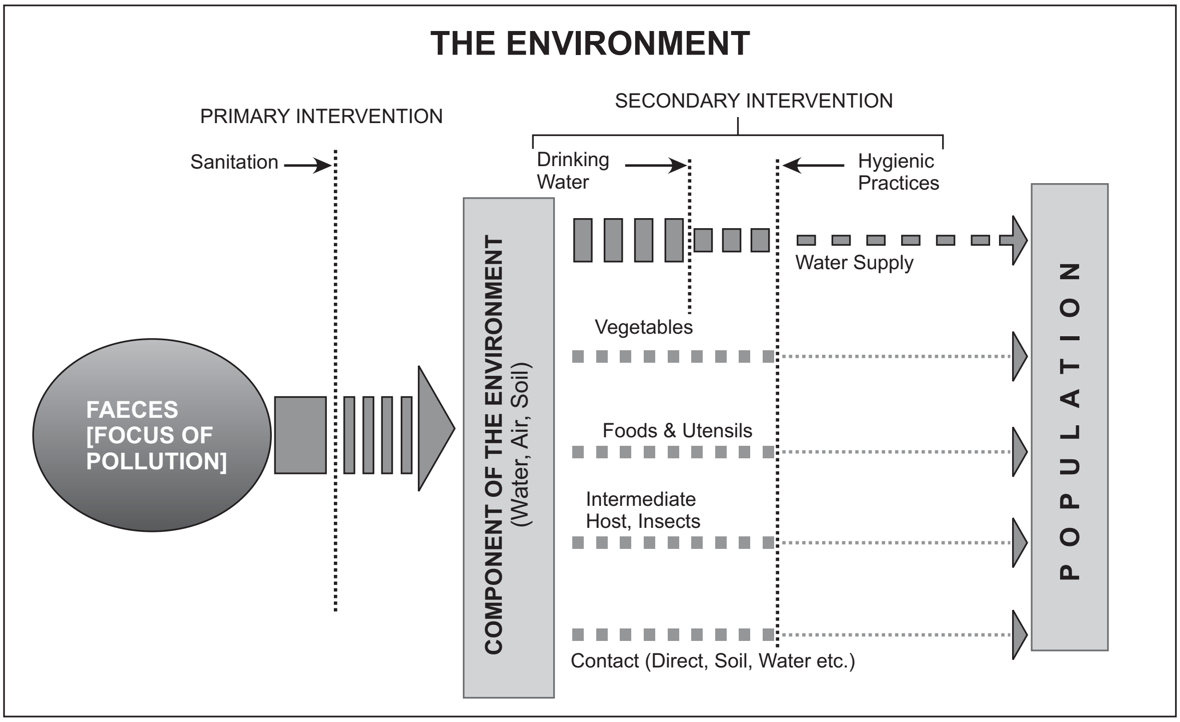


Figure 2.1: **Transmission routes of pollution and diseases in the human environment and their intervention options.**

2.2 International Commitments and the Global Sanitation Scenario

Inadequate sanitation is not a new concern – indeed, the 1980s were the United Nations International Drinking Water Supply and Sanitation Decade (WSSD). During that time period, the international community set a target of achieving 100% coverage in water supply and sanitation by 1990. It was an ambitious target and progress over the decade could not keep up with population growth. By 2000, the World Health Organization estimated that 1.1 billion people still lacked access to a safe water supply, but over twice as many people, 2.4 billion lacked access to basic sanitation. Development agencies believe the new sanitation target agreed at the WSSD is more realistic, but still presents a significant challenge. The World Summit on Sustainable Development in Johannesburg, South Africa, in September 2002, greatly emphasized on safe water and sanitation and urged that the population without sanitation in developing countries be reduced to half by the year 2015. In 2010, the United Nations General Assembly and the UN Human Rights Council recognized access to safe drinking-water and sanitation as a human right. The UN General Assembly proclaims the period from 2005-2015 the International Decade for Action, "Water for Life", to commence on World Water Day, 2005. It is an international drive with focus on water related issue at all levels to meet many of the 2015 Millennium Development Goals MDGs[1])- particularly MDG 7, target 10, which calls for the world to halve the proportion of people without sustainable access to safe drinking water and basic sanitation. This provides an international commitment for an integrated approach to sanitation, water supply, and hygiene promotion. It is recognised that delivering the new sanitation target will require considerable political will together with significant financial, technical, and human resources.

Since 1990, almost 1.9 billion people have gained access to an improved sanitation facility (JMP, 2013). By the end of 2011, there were 2.5 billion people who lacked access to an improved sanitation facility. Of these, 761 million use public or shared sanitation facilities and another 693 million use facilities that do not meet minimum standards of hygiene (unimproved sanitation facilities). The remaining 1 billion (15% of the world population) still practise open defecation. (JMP, 2013) The world remains off track to meet the Millennium Development Goal (MDG) sanitation target, which requires reducing the proportion of people without access from 51% in 1990, to 25% by 2015.Fulfilling this target remains a substantial challenge. Unless the pace of change in the sanitation sector can be accelerated, the MDG target may not be reached until 2026[2].

Box 2.1

Status of Sanitation in Bangladesh

In Bangladesh, much attention was given to provide ample safe drinking water to the people for health promotion during the last few decades but unfortunately sanitation did not receive equal attention. As a result, the population coverage by proper sanitation remains low. Although rate of diarrhoeal mortality has decreased significantly (due to improved clinical interventions), the incidence of diarrhoea still remains high. The strategy of providing safe water without adequate sanitation has failed to create expected health impact in Bangladesh. Consequently Bangladesh is not on track in achieving the

[1]The United Nations Millennium Summit agreed a set of time-bound and measurable goals called Millennium Development Goals (MDGs) aimed at combating poverty, hunger, illiteracy, child mortality, environmental degradation and discrimination against women.

[2]http://www.un.org/waterforlifedecade/sanitation.shtml.

MDG target for improved sanitation coverage, in fact at the current rate of sanitation coverage Bangladesh will fall around 6% short of the MDG target for 2015. Of course, this has much to do with the definition of 'improved sanitation' as per JMP. JMP has a stringent guideline of determining 'improved sanitation' and does not consider shared facilities (e.g. public toilets) as such. Bangladesh has made improvements in the coverage of both 'improved sanitation' and 'shared facilities' in the last two decades and if combined together, it may have reached the MDG targets. According to experts, with the stringent definition for 'improved sanitation', it will be hard to achieve MDG targets for a densely populated country like Bangladesh where in many urban slums, having individual toilet facilities is quite an impractical proposal. In such cases, shared facility is the best possible option that can be provided.

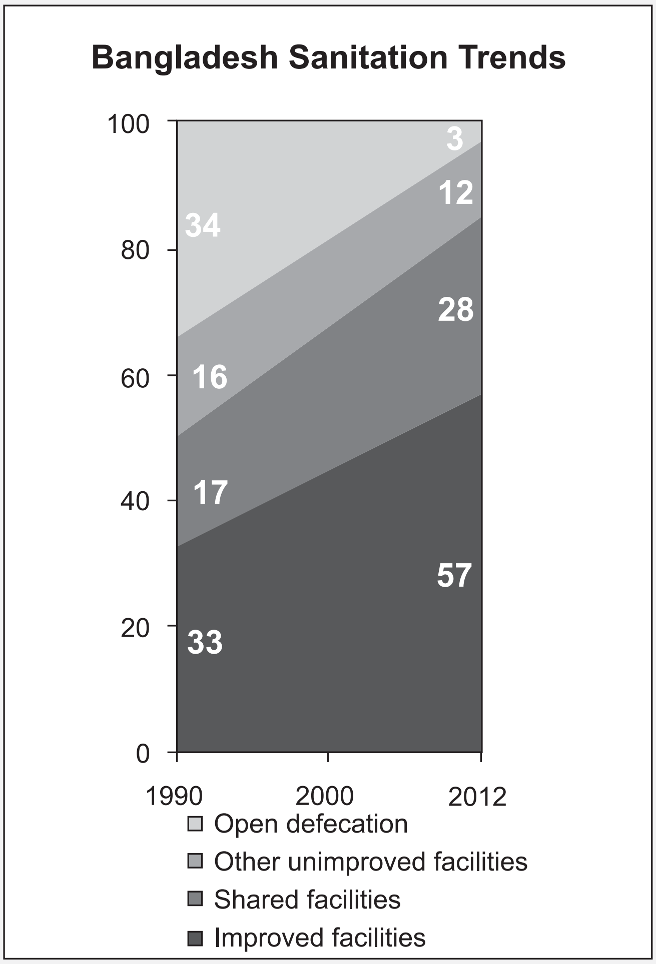


Figure: **Sanitation coverage in Bangladesh (JMP, 2014) As per JMP definition, Flush or pour-flush toilets (to Piped sewer system, Septic tank and pit latrine), Pit latrine, Ventilated Improved Pit-latrine, Pit latrine with slab and Composting toilet together comprise of improved sanitation. Unimproved sanitation include Flush or pour-flush to elsewhere, Pit latrine without slab or open pit, Bucket, Hanging toilet or hanging latrines and no facilities or bush or field (i.e. open defecation). Shared latrines (public toilets) are also considered as unimproved as they require travel, waiting time or a fee in the case of sanitation.**

2.3 Disease Transmission Pathways and the Impact of Sanitation on Public Health

Inadequate sanitation can cause several diseases, which are transmitted from faeces to human through contaminated hands, soil, water, animals and insects (Figure 2.2). Water and excreta borne diseases can be classified into various groups via:

- Water related infections can be of following types -
  - Water borne
  - Water washed
  - Water based
  - Insect vector route
- Excreta related infections can be of following types -
  - Transmission via infected excreta
  - Transmission by an excreta related insect vector.
- Water and excreta related infections

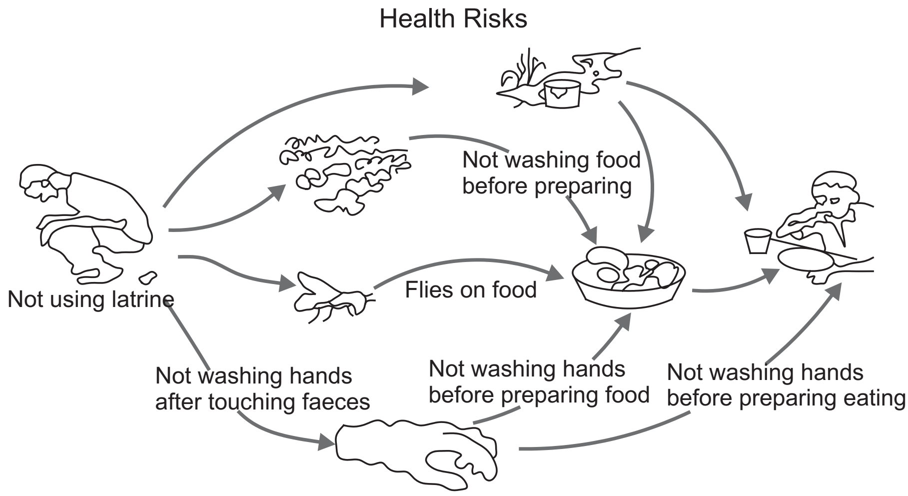


Figure 2.2 Transmission of diseases due to inadequate sanitation.

Water related infections. Water related diseases are those that in some gross way related to unclean environment and impurities in water. Water related disease may be broadly divided into those caused by a biological agents (pathogens), and those caused by some toxic chemical substances in water. The first group is called the water related infections which is most common in developing countries. However, second group is also slowly gaining importance in Bangladesh, particularly due to rapid industrial development and urbanization.

There are four transmission routes that are water related:

Water- borne route: Water-borne transmission occurs when a pathogen is present in water which when drunk by a person he/she gets infected. Potentially water-borne diseases include the classical infections, like cholera and typhoid, but also include a wide range of other diseases, such as infective hepatitis, some diarrheas and dysenteries.

Water-washed route: There are many infections of the intestinal tract and of the skin that are significantly reduced following improvements in the domestic and personal hygiene as well as increased volume of water. They depend on the quantity of water used such as bacterial skin sepsis, scabies, fungal infections of the skin, trachoma etc.

Water-based route: A water-based diseases are ones in which the pathogen spends a part of its life-cycle in a water snail or other aquatic animal. All these diseases are due to infection by parasitic worms (Helmenthses) which depend on the aquatic intermediate hosts to complete their life-cycles. Important examples are schistosomiasis and Guinea worm.

Insect Vector route: The fourth route is via insects that either breed in water or bite near water, eg. Malaria, yellow fever, dengue etc.

Excreta- related infections. Excreta-related infections are ones that are related to human excreta (meaning urine and faeces). Two transmission mechanisms that are excreta related are:

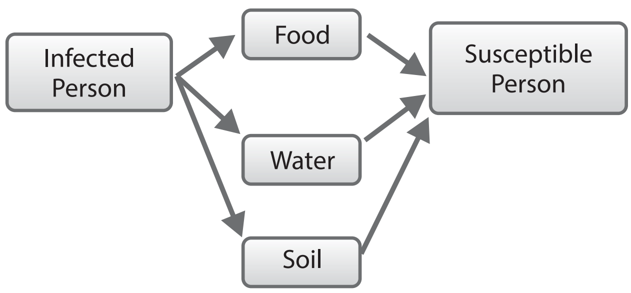
- Transmission via infected excreta:The pathogen is released into the environment in the faeces or urine of an infected individual.
- Transmission by an excreta- related insect vector:An insect that visits excreta to breed or feed may mechanically carry the pathogens to food.

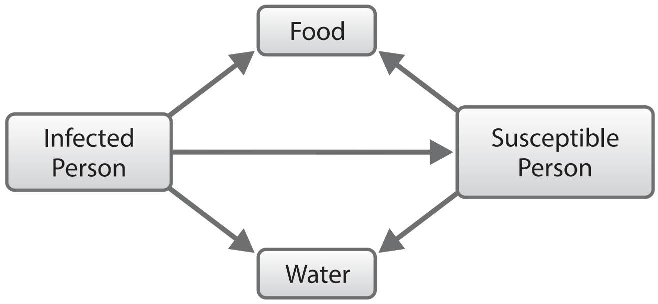
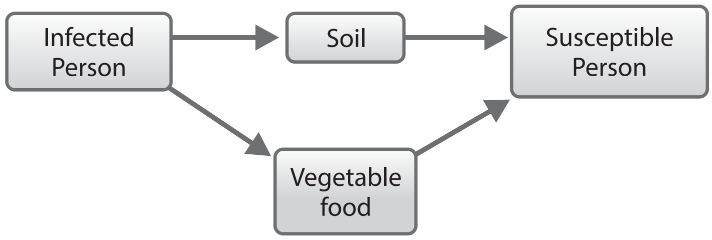
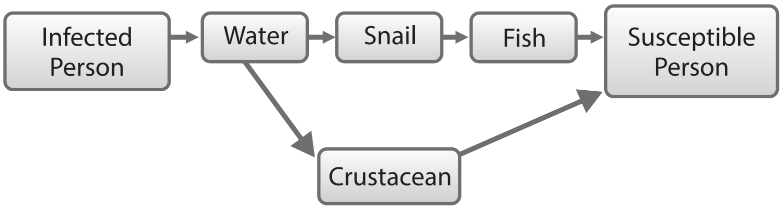
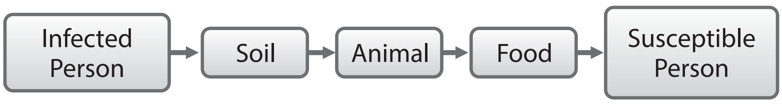
Water and excreta related infections. The water and excreta related classifications are overlapping because many water-related infections are excreta related and most excreta-related infections are water-related. Some examples are:

- The Diarrhea- causing infections and enteric fevers
- Viral Diseases
- Worms with no intermediate host
- Worms with intermediate stages in the pig or cow
- Worms with aquatic intermediate stage
- Water borne re-infection
- Skin and eye infections and louse-borne infections
- The infections transmitted by water-related insects

The following table 2.1 summarizes the water and excreta related infections and their transmission pathways:

Table 2.1 **Transmission routes of water and waste related diseases.**

Diseases	Transmission Route
The Diarrhea- causing infections and enteric fevers **Diarrhoeal diseases:** • Cholera • E.ColiDiarrhoeas • Viral Diarrhoeas • Other Diarrhoeas **Dysenteries:** • Amoebic Dysentery • Bacillary Dysentery	Transmission is faecal-oral; both water-borne and water-washed transmission occur 

<table>
<tr><th>Diseases</th><th>Transmission Route</th></tr>
<tr><td>Enteric Fever :
• Typhoid
• Para-typhoid</td><td></td></tr>
<tr><td>Viral Diseases:
• Poliomyelitis
• Hepatitis-A
Poliomyelitis and hepatitis A & E both caused by viruses are entirely different infections but they have several epidemiological features in common.</td><td>Transmission is faecal-oral

</td></tr>
<tr><td>Worm Infection with no intermediate host
• Ascariasis (round worm)
• Pinworm
• Hookworm
The adult worms live in the human intestine, and their eggs or larvae are passed in the faeces</td><td>They must remain in a suitable environment (usually warm, moist soil) for 5-6 week before they become infectious re-infect by oral route (ingesting food). Hookworm larvae cause re-infection by penetrating the unbroken skin, usually of the foot.

</td></tr>
<tr><td>Worm Infection with aquatic host:
• Schistosomiasis (transmitted through snails, not in India or Bangladesh)
• Guinea Worm</td><td>
</td></tr>
<tr><td>c. Worm Infection with animal host:
• Tape Worm
The beef and pork tapeworms are parasites. The adult worms live in the intestine of humans.</td><td>Eggs present in the faeces are eaten by a cow or pig following which they hatch and form encysted larvae in the muscle, tongue, liver, or other sites. Humans are re-infected by eating inadequately cooked beef or pork containing cysts.

</td></tr>
</table>

Diseases	Transmission Route
Water/waste related insect-borne diseases: • Malaria • Dengue & Yellow Fever • Kalazar • Filariasis • Sleeping sickness	Infected Person → Mosquitoes Flies → Susceptible Person
Skin, Eye and other diseases: • Skin Infection • Scabies • Eye Infection • Louse-borne Typhus	Infected Person → Susceptible Person

Sanitation provides a barrier to faecal diseases, by isolating human excreta and removing traces of faecal material from hands after contact. The following diseases can largely be prevented with basic sanitation and hygiene:

- Diarrhea, that causes an estimated two million deaths per year, mostly among children under the age of five.
- Cholera.
- Schistosomiasis (bilharzia), which infects 100 million people per year, of which 20 million people suffer severe consequences. Improved water and sanitation may reduce it by ~77%.
- Trachoma causes blindness in 6-9 million people per year. Access to sanitation may reduce it by ~25%.
- Intestinal worms infect about a third of the population in developing countries; improved sanitation would control their transmission.
- Hookworms cause malnutrition. Using concrete slabs to cover pit latrines can prevent them from being transmitted to humans.

2.4 Benefits of Improving Access to Sanitation

Increasing access to sanitation is a key component of development and poverty reduction, as it has major health benefits as well as associated social, economic and environmental benefits. These include:

- public health – diseases related to inadequate sanitation and poor hygiene are among the highest causes of illness and death in developing countries, especially among children under five. Providing sanitation is also instrumental in meeting international health targets.
- public services – the public health consequences of inadequate sanitation puts pressure on health services in developing countries.
- human dignity – sanitation facilities provide privacy, safety, dignity, a cleaner environment and greater convenience to users.

- gender – without access to household sanitation women and girls face safety and dignity issues. They may only be able to defecate at certain times to ensure privacy and/or avoid harassment and sexual assault. Lack of school sanitation is a barrier to girls enrolling and staying at school, especially during menstruation.
- poverty elimination and economic growth – illness and death from poor sanitation results in lost economic activity, which reduces household income and the productivity of the local economy. The contamination of rivers and aquifers from human excreta can also damage agricultural production and tourism, which can impact national economies.
- water supply – when human excreta enters a drinking water supply, it compromises safety. Improving sanitation and hygiene practices maximises the benefit of investments in water supply.

2.5 Hygienic Latrine

The linkage between sanitation and health leads to an understanding that the primary focus of sanitation should be on the environmental transmission routes of excreta related diseases. Based on this understanding, a 'hygienic latrine' is defined as a sanitation facility the use of which effectively breaks the cycle of disease transmission. Improved hygienic practice is to be emphasized and proper use of hygienic latrines be ensured because both play the vital role in breaking the cycle of disease transmission.

The most fundamental health objective of sanitation must be achieved through proper design, installation, and use of a sanitary or hygienic latrine. There is no universal design of a hygienic latrine that could be effectively used under all socio-economic and hydrogeological conditions. It is therefore important that a wide range of sanitary or hygienic latrine technologies is available to suit different conditions.

A 'hygienic latrine' would mean to include all of the following:

- Confinement of faeces away from the environment.
- Sealing of the passage between the squat hole and the pit to effectively block the pathways for flies and other insect vectors, thereby breaking the cycle of disease transmission.
- Venting out of foul gases generated in the pit through a properly positioned vent pipe to keep the latrine odor free and encourage continual use of the hygienic latrine.

2.6 Challenges in Sanitation

While sanitation appears to be a basic problem with a seemingly relatively easy solution, recent international efforts have shown it to be a complex issue involving political, financial, technical and institutional issues (Figure 2.3).

Political commitment. Sanitation, hygiene and safe water supply need to be integrated to maximise their effectiveness to meet public health and environmental goals. However, sanitation has not received the same level of investment as water supply. The difference in investment between water supply and sanitation is partially responsible for the gap between water and sanitation coverage. Political commitment for sanitation is seen by many as

Figure 2.3 Challenges of sanitation.

important in shaping government policy and investment priorities, and in implementing the programmes required to meet the target.

Financing. Sanitation's public health and environmental benefits make it a public good, but sanitation is also a private good at the household level. Until recently, most countries and donor agencies treated sanitation only as a public good that could not be provided by the market, and which needed to be subsidised to provide greater incentives to expand coverage. Inappropriate targeting of government subsidies has, however, affected government plans for increasing access to sanitation, as subsidies did not reach those who needed them most. In response, a demand approach to sanitation was developed. Most of the financing for meeting the target is likely to come from users of the facilities, either through their purchasing of materials and providing labour, or through cost recovery schemes. Low interest bank loans are one option to help to ensure that poor families can generate enough money to purchase adequate sanitation facilities. Some NGOs and community groups have resisted full cost recovery for basic services to poor people, as they see this as exacerbating poverty, but others note that many basic services are already paid by users.

Technology transfer and innovation. There are a number of known sanitation technology options that are appropriate in many different contexts in developing countries. To facilitate the appropriate transfer of technologies, there is a need for information to be disseminated to local decision-makers as well as the technical capacity to adapt them to local circumstances. This requires both networks for information exchange and skilled technicians to design and market locally appropriate sanitation solutions. One way of increasing local capacity for technical innovation is to assist developing countries' institutions to adapt solutions to suit local conditions. Some locations may require innovative solutions, for example in wetland areas where groundwater contamination is an issue, or, in extremely poor areas, where technologies might need to be altered to be more affordable. Technical innovation can also aid sanitation suppliers by improving their products and incorporating local materials and building practices into the design of new technologies.

Institutional capacity. Sanitation programmes need planners, decision-makers, and sector professionals who are trained in evaluating different approaches to providing, operating and maintaining sanitation. However, many point to a severe shortage of engineers and field workers to provide the technical and social scientific skills to develop sanitation programmes. This shortage could jeopardise efforts to meet the sanitation target. Historically the sanitation sector has been divided between different government agencies such as health, water, and education. This has led to poor coordination. Some point out that meeting the sanitation target and sustaining its progress require an increase in the capacity and accountability of the public sector to promote, coordinate and regulate sanitation provision. The private sector, mainly in the form of local enterprises, but also including large corporations, has a role in improving access to sanitation. Local small scale suppliers who deliver sanitation products to consumers include local masons, builders and market owners. Individuals entering the sanitation sector may need help and training to provide effective sanitation services to households. The banking sector could also play an important role in providing low cost loans for sanitation improvements.

2.7 Sanitation and Waste Categorization

The major waste streams into the environment are excreta, grey water and solid wastes. Waste streams have varying characteristics depending on the source and interaction with the environment. Pollution in slums is a result of pathogens, nutrients, micropollutants and other trace organics in waste streams. Understanding the components of the waste streams is important in applying technology to overcome the unique challenges in the urban slum environment.

Excreta. Excreta refer to urine and faeces. They form a major health risk due to the presence of pathogens in faeces and the mobility of nutrients and micro-pollutants in urine. In urban slums, where open defecation and excreta disposal in the open storm water drains is very common, this health risk is a reality. Even when sanitation services are provided, the health risk is not eliminated. Pit latrines, which are the simplest form of latrine, are designed to let the liquids percolate into the soil. In the densely populated areas, pollution of soil and groundwater is therefore another major public health risk.

Major contaminant constituents in sludge from pit latrines and septic tanks (see Chapter 3) are organic matter in the form of COD, nutrients and pathogens (bacteria, viruses and parasites). Their concentration is lower in the municipal wastewater treatment sludge compared to pit latrine sludge and septage as shown in Table 2.2.

Grey water. Grey water is wastewater of domestic origin from bathroom, kitchen, and laundry use, excluding wastewater from the toilets. Grey water accounts for 65% to 75% of the domestic wastewater consumption in peri-urban areas of developing countries where water consumption ranges from 20 L/ca.d to 30 L/ca.d(Morel and Diener, 2006). The amount of water consumed can thus be used to estimate the return factor of grey water generated in the urban poor areas or slums. It can be argued that the potentially negative impacts from grey water disposal are in urban slums with water supply services and where little or no consideration has been given to the planning and management of grey water. Grey water contains contaminants of concern that include suspended solids, pathogens,

Table 2.2 Major contaminant constituents in sludge from pit latrines and septic tanks.

Parameter	Faecal and WWTP sludge characteristics (mean and range values)			References
	Public toilet sludge	Septage	WWTP sludge	
Total solid, TS (mg/L)	52,500	12,000– 35,000		Koné and Strauss (2004).
	30,000	22,000		NWSC (2008).
	≥3.5%	<3%	1%	Heinss et al. (1998).
Total volatile solids (% TS)	68	50–73	–	Koné and Strauss (2004).
	65	45	–	NWSC (2008).
COD (mg/L)	49,000	1200–7800	–	Koné and Strauss (2004).
	30,000	10,000	47–608	NWSC (2008).
	20,000– 50,000	<10,000	500–2500	Heinss et al. (1998).
BOD_5 (mg/L)	7600	840–2600	–	Koné and Strauss (2004).
			20–229	NWSC (2008).
Total nitrogen, TN (mg/L)	–	190–300	–	Koné and Strauss (2004).
			32–250	NWSC (2008).
Total Kjeldahl Nitrogen, TKN (mg N/L)	3400	1000		
NH_4–N (mg/L)	3300	150–1200	–	Koné and Strauss (2004)
	2000	400	2–168	NWSC (2008).
	2000–5000	<1000	30–70	Heinss et al. (1994).
Nitrates, NO_3^- (mg N/L)	–	–	–	Heinss et al. (1998).
	–	–	–	Koné and Strauss (2004)
Total phosphorus, TP (mg P/L)	450	150	9–63	NWSC (2008).
Faecal coliforms (cfu/100mL)	1×10^5	1×10^5	6.3×10^4 –6.6×10^5	NWSC (2008).
Helminth eggs	25,000	4000–5700		Heinss et al. (1998).
	20,000–60,000	4000	300–2000	Heinss et al. (1998).
		600–6000		Ingallinella et al. (2002).

Adapted from Katukiza, A., Y. (2012).

nutrients, grease and also organic micro pollutants (Table 2.3) from household chemicals and pharmaceuticals that may be present due to urine contamination (Elmitwalli and Otterpohl, 2007; Eriksson et al., 2009; Li et al., 2009). Kitchen grey water contains a higher level of COD and total suspended solids (TSS) than grey water from the bathroom and laundry (Li et al.,2009). It has a nutrient content close to the COD:N:P ratio of 100:20:1 (Metcalf and Eddy, 2003) while other streams of grey water have low concentrations of nitrogen and phosphorus. The nutrient content is a limiting factor in application of conventional biological treatment processes for grey water treatment (Jefferson et al., 2001). Typically, all grey water types have a good biodegradability as indicated by their BOD_5 :COD ratio which is close to 0.5 (Knerr et al., 2008; Li et al., 2009).

Solid waste. Solid waste management is one of the major problems to the environment and to public health faced by developing countries. The growth of cities in developing countries

Table 2.3 Characteristics of grey water.

Parameter (units in mg/L unless otherwise stated)	Types of grey water										
	Christova-Boal et al. (1996)	Li et al. (2009)	Surendran and Wheatley (1998)	Li et al. (2009)	Christova-Boal et al. (1996)	Surendran and Wheatley (1998)	Li et al. (2009)	Siegrist et al. (1976)	Hargelius et al. (1995)	Li et al. (2009)	Gerba et al. (1995)
	Bathroom	Bathroom	Bathroom	Laundry	Laundry	Laundry	Kitchen	Kitchen	Bathroom & Kitchen	Mixed	Mixed
Turbidity (NTU)	60–240	44–375	92	50–444	50–220	108	298	–	–	29–375	15.3–78.6
TSS	–	7–505	631	68-465	–	658	134–1300	2410	–	25–183	
pH	6.4–8.1	6.4–8.1	7.6	7.1-10	9.3–10	8.1	5.9–7.4		–	6.3–8.1	6.7–7.6
Electrical cond. (μScm^{-1})	82–250	–	– –	–	190–1400	–	–		–	–	– –
BOD_5	76–200	50–300	–	48–472	48-290	–	1460	1460	–	47-456	
COD	–	100–633	425	231–2950	–	725	26–2050		–	100-700	–
TOC	–	–	104	–	–	110	–	880	–	–	–
Oil and grease	37–78	–	–	–	9.0–88	–	–	–	–	–	–
Total N	–	3.6-19.4	–	1.1–40.3		–	11.4–74	74	–	1.7–34.3	–
TKN	4.6–20	–	–	–	1–40	–	–	–	–	–	–
NH_4–N	<0.1–15	–	1.56	–	<0.1–1.9	10.7	–	6	–	–	–
NO_3–N	–	–	0.9	–	–	1.6	–	0.3	–	–	1.8–3.0
Total P	–	0.11–>48.8	–	ND–>171	–		2.9->74	74	–	0.11–22.8	–
PO_4–P	0.11–1.8	–	1.63	–	0.062–42	10.1	–	31	–	–	–
Total coliforms (CFU/100ml)	$500–2.4\times10^7$	$10–2.4\times10^7$	6×10^6	$200.5–7\times10^5$	2.3×10^3 - 3.3×10^5	7×10^5	$>2.4\times10^8$	–	–	$56–8.03\times10^7$	$10^{7.2}–10^{8.8}$
Faecal coliforms (CFU /100ml)	$170–3.3\times10^3$	$0–4.4\times10^5$	600	$50–1.4\times10^3$	$110–1.09\times10^3$	728	–	–	40×10^6	$0.1–1.5\times10^8$	$10^{5.4}–10^{7.2}$
E. coli (CFU/100ml)	–	–	–	–	–	–	–	–	236×10^6	–	–

Source: adapted from Katukiza, A., Y. (2012).

has resulted in growth of periurban areas that generate large amounts of organic solid waste. However, this growth has not been matched with the provision of suitable solid waste management practices at affordable costs in terms of collection, transport and treatment or safe disposal (Chanakya et al., 2009; Okot-Okumu and Nyenje, 2011). In developing countries, less than 50% of the solid waste generated in urban areas is collected centrally by the municipalities and private sector with limited recycling or recovery of recyclable materials (Jingura and Matengaifa, 2009; Okot-Okumu and Nyenje, 2011). This is a missed opportunity for resource recovery, economic benefits, and to reduce the waste quantities disposed in landfills. The substantial amount of solid waste that remains uncollected is likely to result in environmental pollution and negative public health effects (Bhatia and Gurnani, 1996). Solid waste generated in urban slums has a large portion that is organic (Table2.4). Poor management of organic solid waste can lead to emission of methane from solid waste dumps as well as leaching of nutrients, micro-pollutants and organic matter to the natural environment. The increasing volume of solid waste generated with varying characteristics calls for a system approach from source separation to treatment options that promote resource recovery and finally disposal to a landfill of solely the inert fraction for which at the time no useful application can be found.

Table 2.4 **Composition of municipal solid waste in developing countries.**

Solid waste type	Composition (%)				
	Henry et al. (2006)	KCC (2006)	Ogwueleka (2009)	Rajabapaiah (1998)	Sharholy et al. (2007)
Food residues	53	74	76	65	45.3
Paper	16.8	11	6.6	8	3.6
Textile	2.6	1	1.4		2.2
Plastic	12.6	12	4	6	2.9
Grass/wood	5.6	–	–	7	
Leather	1	–	–	–	
Rubber	1.5	–	–	–	
Glass	2	2	3	6	0.7
Metal	2.3	0.4	2.5	3	2.5
Other waste (leather, rubber, cardboard, wood, bricks, ash and soil)	2.6	0.4	6.5	5	42.8

Adapted from Katukiza, A., Y. (2012).

2.8 Sanitation Technologies

Treatment of waste streams in using sustainable technologies preserves their reuse potential to recover resources. Examples of resources are bio-energy generated from transformation of organic material, plant nutrients (nitrogen, phosphorus, potassium as macro nutrients; calcium and sulfur as micro-nutrients) and water treated to the desired discharge standards. Innovative waste disposal and treatment technologies have been developed in an attempt to address the sanitation crisis in urban slums, but they can only be sustainable if they function as elements within a sanitation system.

CHAPTER 2

A sanitation system is complete when it has a defined flow stream for each of the products. Technologies and institutional aspects complement each other in the collection, transport, treatment and disposal or reuse of the waste. The settlement dynamics in urban slums have triggered new thinking because improved onsite sanitation will still play a key role in the foreseeable future in order to meet the Millennium Development Goal (MDG).

Sanitation systems may have varying combinations of these functions depending on the local conditions. Sanitation systems can, however, be classified based on various other criteria. For instance, the classification of on-site or off-site sanitation systems depends on whether the waste is stored, treated and disposed of at the point of generation or transported to somewhere else for treatment and/or disposal. Sanitation systems can also be classified as wet or dry systems based on the methods of collection and conveyance of wastes produced in a community. Wet and dry systems can be either on-site or off-site systems.

On-site systems. When the wastes are collected, treated and disposed of at the point of generation it is called an on-site system e.g., pit latrines and septic tank systems. On-site sanitation systems are widely used in rural areas of both developed and developing countries, and in the absence of more costly sewerage system, is also extensively used in urban areas of developing countries. The simplest on-site sanitation system is a pit latrine, which consists of a manually dug pit covered by a concrete, wooden or bamboo slab with a squatting hole. Some form of superstructure is erected over the pit to ensure privacy to the user.

The on-site sanitation system has, over the years, been developed and improved into a lot of different designs (e.g., ventilated improved pit latrine, pour-flush single and double-pit latrines, aqua privies, septic tanks and so on) which will be discussed in detail in the following article. The basic principles of on-site systems, however, remain the same: liquids infiltrate into the soil and the solids are retained, anaerobically digested and have to be removed, or a new pit has to be dug at regular intervals. The basic on-site systems are primarily designed to dispose of human excreta. Wastewaters from cooking, clothes washing, and bathing are collected in small drains and disposed of in soakaways for infiltration.

Where on-site sanitation systems are not feasible because of high population density, high water consumption, low infiltration rate of soil or high groundwater table, wastes have to be collected and transported off-site for treatment and/ or disposal.

Off-site systems. When the waste is collected and transported to somewhere else for treatment and disposal, the system is called off-site, e.g., bucket latrine systems and conventional sewerage system. The basic elements of off-site sanitation systems therefore include collection, transportation, treatment, disposal and/ or reuse. The waste is collected either through house sewers or manually using buckets or vaults, transported either by cart, truck or sewer system to a suitable distant place where it is treated prior to disposal or reuse.

Collection and transportation of the wastes through a sewer reticulation system requires that the waste be diluted by water. Hence it is essential that piped water supply be available in areas where this system is to be applied. This waterborne system is by far the most satisfactory system of waste disposal provided sufficient funds are available for its construction and maintenance.

Since costs of waterborne sanitation systems are prohibitive, it may be preferable to introduce such systems gradually, and where possible existing sanitation systems should be upgraded and reused for improvement. For instance, the so-called small bore sewerage system can be employed by making use of the existing septic tanks or upgraded aqua privies. In this system, the costs can be significantly reduced because of smaller sewer sizes and lower gradients.

Dry systems. In dry systems no water is used for the dilution of the waste. They are usually applied in unsewered areas with no piped water supply, e.g., pit latrine systems (on-site) and bucket latrine systems (off-site).

Wet systems. In the wet system the waste is diluted with flushes of water. Wet systems are suitable where piped water supply systems are available, e.g., septic tank systems (on-site) and conventional sewerage systems (off-site).

Sometimes a sanitation system can be classified as either permeable or confined depending on whether the system allows infiltration. Confined systems do not allow infiltration of the liquid portion of the wastes into the ground, e.g., aqua privies, septic tanks etc. In the unconfined or permeable system the liquid part of the wastes is allowed to infiltrate, causing potential pollution of the groundwater, e.g., pit latrines.

Sanitation systems may be considered as a combination of several components of unit operations and processes. Each component may imply several different alternatives for

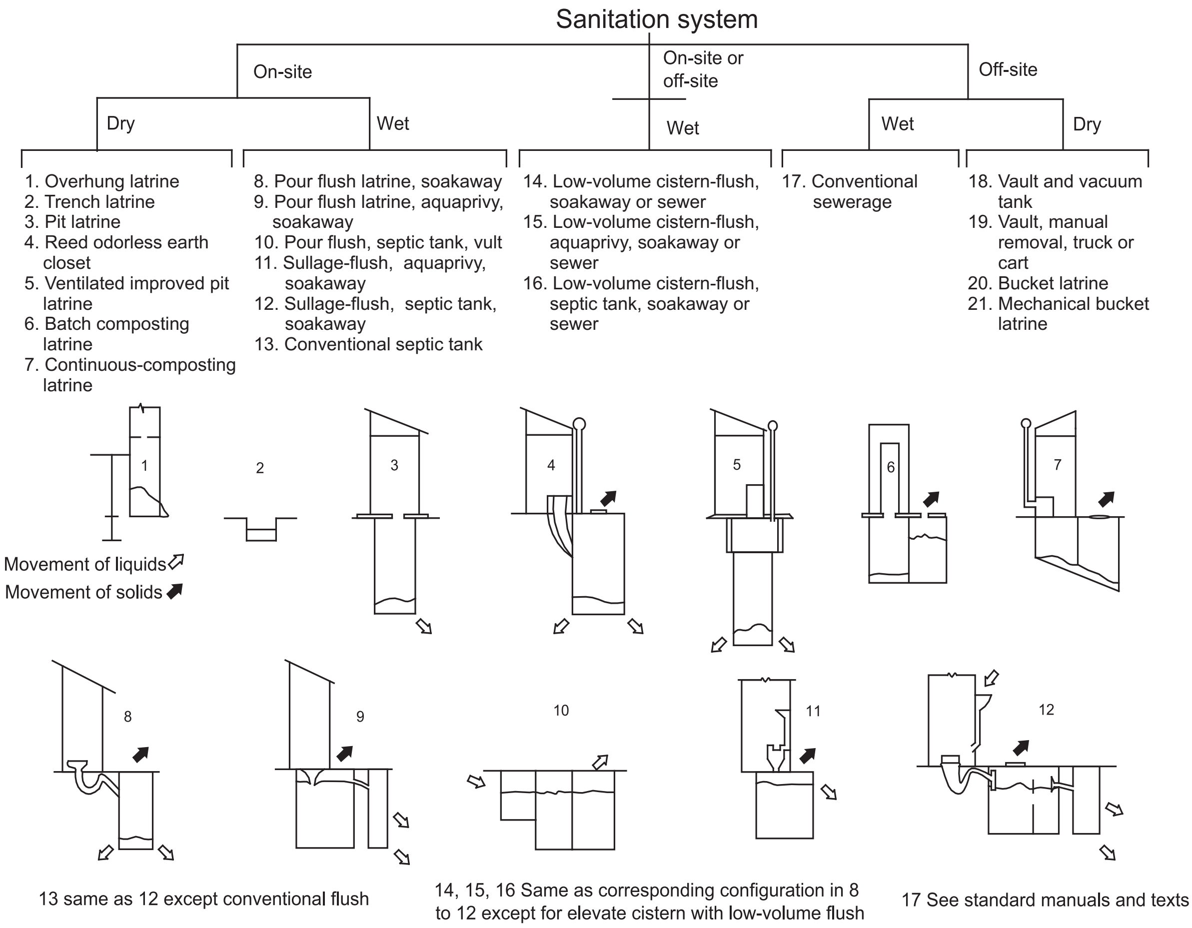


Figure 2.4 Categorization of sanitation technologies.

choice of unit operation and/ or facility. Thus, a sanitation system can be combined in many ways. For instance, effluents from septic tanks can be disposed of into a soakage pit where feasible or can be connected to a small bore sewer system for off-site treatment and disposal. Figure 2.4 illustrates the categorization of sanitation technologies.

2.9 Major Sanitation Technologies

Pit latrines are basic structures that can be adapted easily into different types of latrines such as VIP latrines and ecological sanitation systems. Other latrines share many common features of simple pit latrines; therefore, focusing first on pit latrines will help to understand the other sanitation technologies. This section considers a selected range of low-to medium-cost sanitation technologies beginning with the least-cost pit latrine technology, focusing on its design considerations, merits, demerits and suitability.

Pit latrine with slab. Pit latrines are the simplest form of dry latrine. They consist of a pit dug in the ground and a cover slab or floor above the hole (Figure 2.5). Pit latrines must have a cleanable cover slab in order to be considered as improved sanitation systems. The excreta (both faeces and urine) drop through the hole to enter the dry pit. Pit latrines should be constructed on a slight mound so they are higher than the surrounding ground and water at the surface will flow away from the hole. They should also have a lid that can be placed over the hole to reduce problems with flies and odours. They may have a squat pan or a raised footrest to make using the latrine more convenient. The pit is often lined but the bottom remains open, allowing the liquid to drain into the soil and leaving the solids behind.

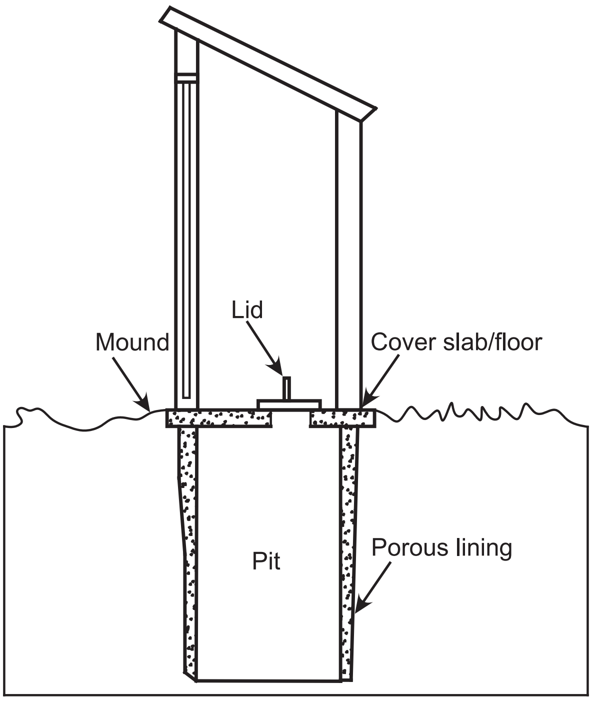


Figure 2.5 Diagram of a simple pit latrine.

Pit latrines should also have an upper part, called the superstructure, to provide protection from the rain and sun, and privacy and comfort for the user. Pit latrines can have a single pit or double pit. In double pits, while one is filling with excreta, the second pit remains out of service. When the first pit is filled with excreta up to about 50 cm below the slab, it is taken out of use and the remaining space is filled with grass and vegetation materials that can be composted. One then use the second pit until that is full. Meanwhile, the first pit will stay sealed for a period of 6–9 months, during which time the waste will decompose and any pathogenic microorganisms will die. After this period, the material (humus soil) in the first pit can be taken out manually. Humus or humic substance is used to describe organic matter that has been stabilised by decomposition processes. It is safe to handle and readily used as fertiliser in agriculture or can be disposed of safely. This is the main principle of ecological sanitation.

In general, pit latrines with a slab isolate human excreta from the surrounding environment and prevent the transmission of faeco-orally transmitted diseases. They also have other advantages:

- They do not require water so are appropriate in areas where there is no adequate water supply.
- Squatting is normal to many people and thus is acceptable to users.
- Alternating double pits will allow the excreta to drain, degrade and transform into a nutrient-rich, safe humic material that can be used to improve soils.
- They avoid contamination of surface water and top soil if properly installed and maintained.
- They can be constructed with minimum cost using local material and local skills.
- The presence of properly constructed slabs will allow easy cleaning and avoid flies and unsightliness.

However, pit latrines are not without limitations. There may be a foul odour from the pit and they can be a favourable place for the breeding of flies and mosquitoes. With single pits, a new pit needs to be dug every time one gets full. They can be susceptible to failure/overflowing during floods. Other disadvantages can be overcome by proper design, construction and usage. For example, if the superstructure is not properly constructed, it may discourage use of the latrine by family members. Children may be discouraged from using the latrine if the slab is not designed with them in mind and is too big for them. Use of excess water or less compostable materials for anal cleansing should be avoided because it may affect the decomposition rate of human excreta.

The design of a pit latrine should follow the following criteria:

- The site of a latrine should preferably be in the backyard of the house and away from an alley in the village. It should not be nearer than 6 m or farther than 50 m from the house. The direction of the wind should be away from the main house.
- If there is a well in the compound, the latrine should be located as far away from it as possible on the downhill side to avoid possible seeping and contamination of groundwater.
- The faecal microorganisms may migrate from the pit through the soil, however, the degree that this happens varies with the type of soil, moisture levels and other environmental factors. It is, therefore, difficult to estimate the necessary distance between a pit and a water source, but 30–50 m is the recommended minimum, with an absolute minimum of 15 m.

- The size of the pit depends on the number of people using it and the design period, i.e. the length of time before it is full. Typically, the pit should be at least 3 m deep for a family of five for a design period of three to five years. The diameter should be at least 1 m; up to 1.2 m diameter will make it easier to dig but if it exceeds 1.5 m there is an increased risk of collapse, especially in sandy soils.
- In flood-prone areas, it is advisable to raise the mound of the latrine and prepare diversion ditches around it. When the soil condition is rocky and it is impossible to dig a deep pit, the depth of the pit can be extended by building upwards with concrete rings or blocks. However, care must be taken to ensure the structure remains watertight.
- The level of the water table must also be taken into consideration. The pit must be entirely above the water table at all times of the year. If the water table is near the surface of the ground, the waste in the pit may contaminate the groundwater.
- Lining the pit prevents it from collapsing and provides support to the superstructure. The pit lining material can be brick, rot-resistant timber, concrete, stones, or mortar plastered on to the soil. If the soil is stable (i.e. no sand or gravel deposits or loose organic materials), the whole pit need not be lined. The bottom of the pit should remain unlined to allow the percolation of liquids out of the pit.
- The superstructure should be built using locally available materials. These may include a masonry wall made of cement blocks, bricks, or stone with cement or mud bindings; or a wooden structure covered with timber, bamboo, grass/thatch, sticks, leaves of banana or enset trees, or canvas made of sacks. However, the type of superstructure depends on several factors such as a household's financial capacity, the availability of construction material locally, local customs and traditions, and the availability of skilled artisans.

The cover slab needs to be strong and have a smooth surface so it can be cleaned easily. It may be made of concrete or termite- or rot-resistant timber, with or without stones and mud covering. Various designs of slab are used.

Pit latrines must be properly maintained to function properly. It is advisable to keep the squatting or standing surface clean and dry. This will help to prevent pathogen/disease transmission and limit odours.

If the pit has been dug to an appropriate size for the number of users, then it may never become full. The liquid will drain into the soil and the solid waste will slowly decompose so the volume remains stable.

Ventilated improved pit (VIP) latrine. The VIP latrine is an improvement over the simple dry pit latrine. The distinctive feature that gives the VIP latrine its name is the vent pipe installed into the pit, which is used to exhaust the foul odour from the pit and control flies (Figure 2.6).

The principle is that a continuous flow of air comes in through the superstructure and enters the pit through the hole. This cold air will go down into the pit displacing (pushing up) the hot smelly air upward through the vent pipe. The other advantage of the vent is controlling flies. As we discussed earlier, dry pit latrines potentially serve as breeding places for flies.

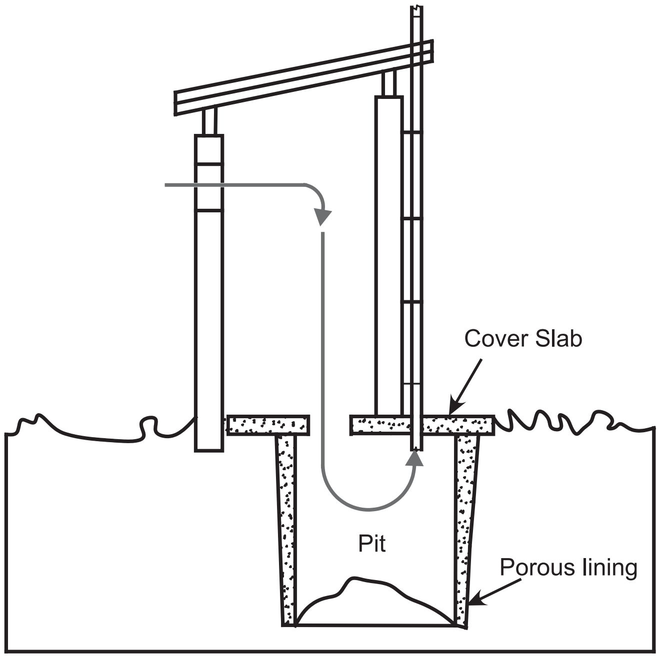


Figure 2.6 **Diagram of VIP latrine.**

Newly-emerging adult flies will try to escape through the vent pipe because the pipe allows sunlight to enter into the pit and flies are photopositive (meaning they move towards light) by nature. A mesh screen tied at the top of the vent pipe will prevent flies from escaping to the outside of the latrine.

VIP latrines can have a single pit or double pit. They share the advantages of simple pit latrines with slabs described above but they also have unique advantages that improve on the limitations, namely, that flies and odours are significantly reduced. It should be noted, however, that the health risks from flies are not completely removed by ventilation.

As these latrines are based on a simple pit latrine, discussion is focused on only the improved features of VIP latrines. The vent pipe should have an internal diameter of 110–150 mm and reach more than 300 mm above the highest point of the superstructure. The vent works better in windy areas but where there is not much wind its effectiveness can be improved by painting the pipe black. This makes the vent pipe warmer and the heat difference between the pit (cool) and the vent (warm) creates an updraft that pulls the air and odours up and out of the pit. To test the efficacy of the ventilation, a small, smoky fire can be lit in the pit; the smoke should be pulled up and out of the vent pipe and not remain in the pit or the superstructure. The mesh size of the fly screen must be large enough to prevent clogging with dust and allow air to circulate freely. Aluminum screens with a holesize of 1.2–1.5 mm have proved to be the most effective.

The maintenance requirements are similar to simple latrines. In addition, dead flies, spider webs, dust and debris should be removed from the ventilation screen to ensure a good flow of air.

Reed Odourless Earth Closet (ROEC). The Reed odourless earth closet (ROEC) is a variation on the ventilated improved pit latrine. With ROEC, the pit is fully off-set from the superstructure and is connected to the squatting plate by a curved chute as shown in Figure 2.7.

The ROEC is fitted with a vent pipe to control odour and insect nuisance. It is claimed that the chute, in conjunction with the ventilation stack, encourages vigorous air circulation down the latrine, thereby removing odours and discouraging flies. This latrine is common in southern Africa. The design considerations and design principles of ROEC are similar to those of a single pit VIP latrine.

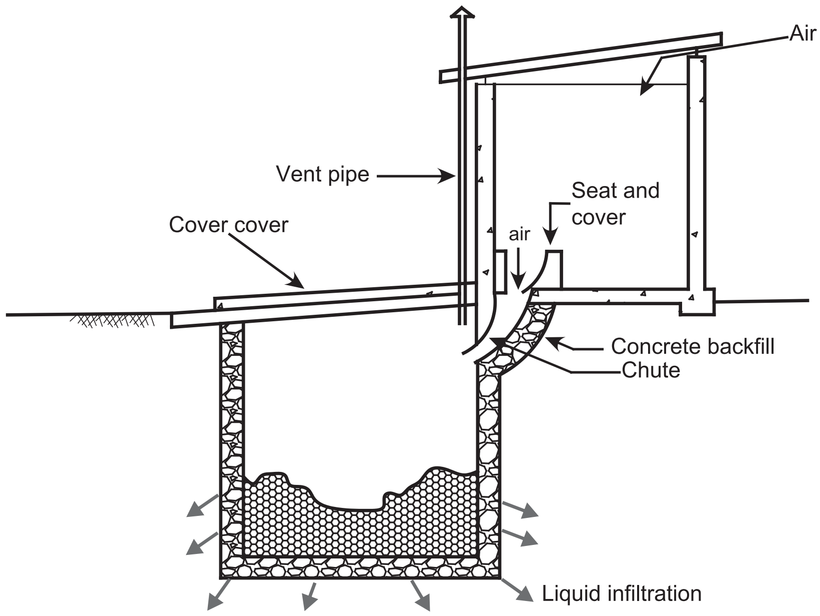


Figure 2.7 **ROEC sanitation technology.**

The important advantages of ROECs are as follows:

- ROEC pit can be made larger as the superstructure is fully off-set and thus can have a longer life than VIP latrine;
- pit can be easily emptied without disturbing the superstructure and it can be a permanent facility;
- there is no danger of users, particularly children, falling into it;
- it may be more acceptable to users because the excreta cannot be seen.

The main disadvantages are as follows:

- the ROEC chute easily becomes fouled with excreta, thereby providing a possible site for fly breeding and odour nuisance;
- the chute has to be regularly cleaned with a long-handled brush or a small amount of water.

Ecological sanitation. Also known as ecosan, it describes an approach to human waste management rather than a single method. In ecosan systems, human excreta is considered to be a resource, not waste. The principle is to make use of excreta by transforming it into an end product that can be used as a soil improver and fertiliser for agriculture. Ecological sanitation aims to decrease contamination of the environment caused by human excretion and to prevent faeco-orally transmitted diseases. An additional benefit of using waste in this way is that the amount of artificial fertiliser used in cultivation of fields is decreased. This saves money for the farmer and protects lakes and other water bodies from eutrophication caused by runoff of these additional fertilisers.

There are, however, some constraints for communities to consider before adopting the ecosan approach. Ecosan systems require a little more space than conventional latrines. At the end of the process the decomposed waste, known as compost or ecohumus, has to be dug out before it can be spread on the land. There may be a cultural taboo against handling of excreta, even though it should be more like soil than waste by this stage. Some people may be unwilling to use the crops and foods produced. Nonetheless, ecological sanitation is a more sustainable approach to waste management than other systems and should therefore be promoted as the preferred option.

Arborloo – a single pit ecosan method. A simple form of ecological sanitation is the Arborloo (Figure 2.8). This consists of a single, unlined shallow pit with a portable ring beam (circular support), slab and superstructure. It is used like a normal latrine but with the regular addition of soil, wood ash and leaves. When it is full, it is covered with leaves and soil and a small tree is planted on top to grow in the compost. (The tree gives the system its name; 'arbor' is Latin for 'tree'.) Another pit is dug nearby and the whole structure is relocated over the new pit. No handling of the waste is required. If a fruit tree or other useful variety is grown there is the added benefit of food or income.

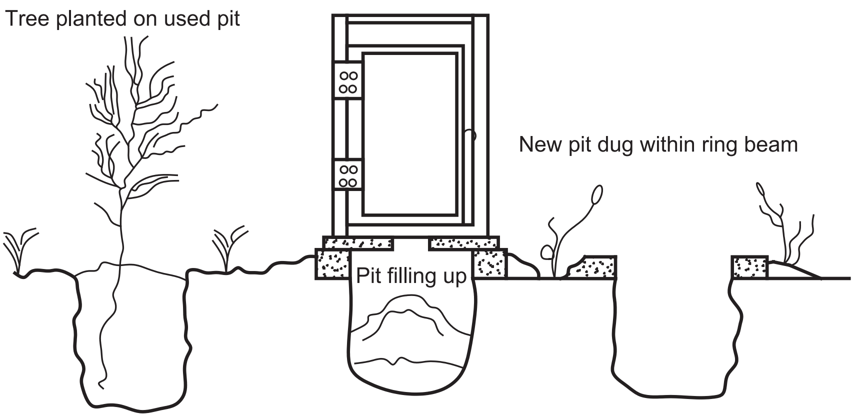


Figure 2.8 **Arborloo – a single pit ecosan system.**

Fossa Alterna – a double pit method. The double pit latrine system can be constructed to be an ecosan system. The alternating waterless double pit is also known as Fossa Alterna, which means alternate ditch. The physical structure is constructed in a similar way to a single pit latrine except that it has two pits and they are shallower than a normal pit with a maximum depth of 1.5 m. The slab and superstructure may be movable between the two pits (Figure 2.9) or may be a larger permanent structure that covers both pits.

Like the Arborloo, soil, wood ash, vegetable kitchen waste and leaves are added regularly. A small amount should be added after each defecation (not urination). This introduces necessary plant material to mix with the human waste and also adds a variety of organisms like worms, fungi and bacteria that help in the degradation process.

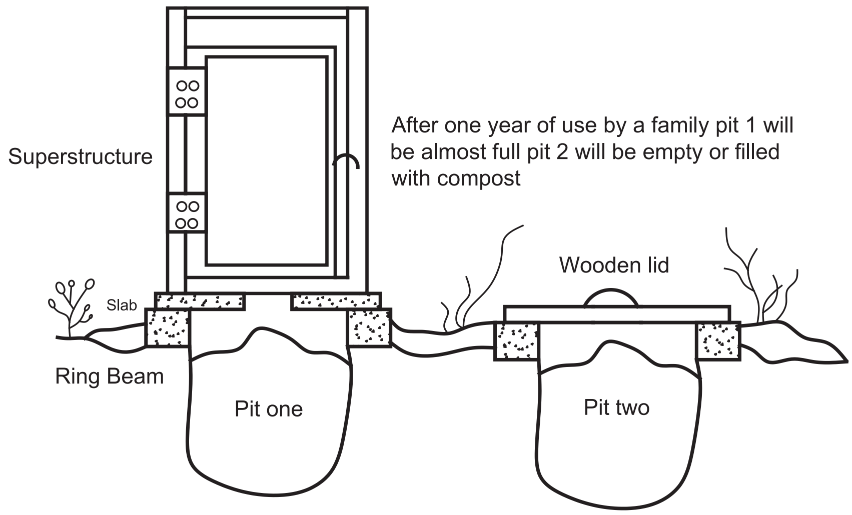


Figure 2.9 **Fossa alterna – double alternating compost pit toilet.**

When the first pit is full, after about 12–24 months depending on the size of the pit and the number of users, everyone starts using the second pit instead. The first pit is covered and the material in it will degrade into a dry, earth-like mixture. This takes about 6–12 months. After this time, the composted mixture is dug out manually and can be used to spread on soil. It is important in the construction to make sure the slab is movable or has a manhole large enough to allow access to the pit for digging out. The health risk for the people who empty the compost is minimal if the pit has been left for over one year. However, good personal hygiene should always be promoted in activities related to sanitation.

2.10 Water Carriage Systems

Cistern flush toilet. The cistern flush toilet, also known as a water closet or WC, is usually made of ceramic material (Figure 2.10). The flush toilet consists of two parts: a tank (cistern) that supplies flushwater for carrying away the excreta and a bowl into which the

Figure 2.10 **Cistern flush toilet cistern or tank is behind the raised lid.**

excreta are deposited. It also needs connection to constant running water and a discharge pipe to take the wastewater away to a sewer or septic tank. WCs are rarely found in rural households but are quite common in government offices, some schools and health facilities.

The attractive feature of the flush toilet is that it has a water seal to prevent odours from coming back up through the plumbing. A skilled plumber is needed to install a flush toilet. From the users' perspective, it is a safe and comfortable toilet to use provided that it is kept clean, but the high capital cost for installation and the need for skilled personnel makes it not affordable by every family, especially those living in rural areas.

Pour-flush toilets. A pour-flush toilet is like a cistern flush toilet except that instead of the water coming from the cistern above, it is poured in by the user. When the water supply is not continuous, any cistern flush toilet can become a pour-flush toilet. Water is simply poured into the bowl manually from a bucket or a jug to flush the excreta; approximately 2–3 litres of water is usually sufficient. Pour-flush toilets share all the advantages of cistern flush toilets but use a lot less water. The wastewater should be disposed of to a septic tank or seepage pit, also known as a leach pit (Figure 2.11).

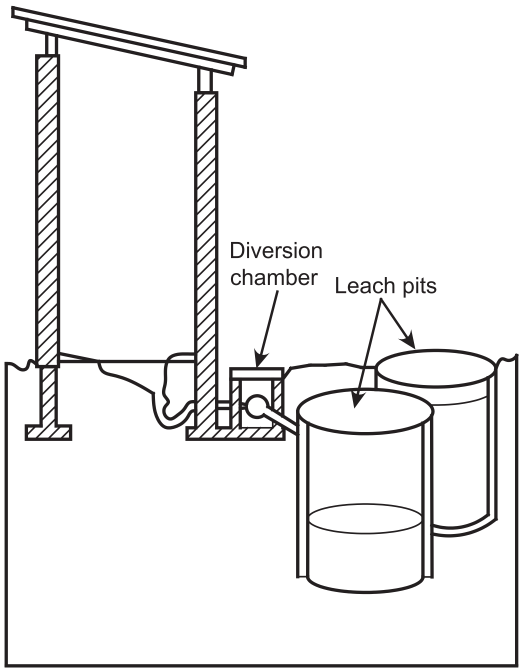


Figure 2.11 Pour-flush latrine design.

The pit will contain excreta, cleansing water and flush water. As this leaches from the pit and migrates through the soil, faecal organisms are removed. In some geological conditions, there is a risk of groundwater pollution; therefore, this method is not always recommended.

The basic functions of a pour-flush latrine are:

- After each use, the latrine is manually pour-flushed through the pan and trap with about 2-3 litres of water. Some of the clean flushwater remains in the trap and maintains the water seal, thus providing the barrier against odours and insects as in the case of conventional cistern-flush toilets which use between 10 and 20 litres of water per flush.

- From excreta and flushwater around 5-10 litres per capita per day (lcd) of wastewater enter the pit, together with an additional usually equal amount if water is used for anal cleansing. The pit has to provide sufficient volume for solids storage, as well as sufficient area for the wastewater to infiltrate into the soil, which requires that the soil has sufficient long-term infiltrative capacity. If the soil is unsuitable for infiltration, the liquid effluent can be removed by other means e.g., by connecting to the sewerage system if available.

Aqua privy. The aqua privy is a single pit latrine which has a watertight pit filled with water. Excreta drop into the pit and wastewater is displaced into a storage chamber, a seepage pit or a sewer line. It needs to be topped up regularly, so a nearby water supply is required.

Urinals. Urinals, used by men and boys, are only used for collecting urine. Urinals are either wall-mounted units or a drainage channel constructed on the floor in connection with the wall. Most urinals use water to flush although waterless urinals are now becoming popular. In public places and schools, urinals for men and boys help to keep toilets cleaner and decrease the demand for more toilet-seats.

2.11 Handwashing Facilities

Every latrine or toilet must have handwashing facilities. As you know, hygiene is an essential component of health promotion and one of the critical times for handwashing is after visiting the toilet. A latrine without a proper handwashing facility will not serve its ultimate objective of disease prevention.

If there is no running water, handwashing stations can be made using jerrycans, tin cans, wooden bowls, or pottery depending on the local culture and custom of the community. Simple devices/jars can be made using very basic materials. They should have also the provision of soap or ash for washing.

2.12 People's role in Latrine Construction

Sanitation contributing to attaining the MDGs suffers from one major difficulty; any sanitation facility shared by more than one household is considered to be unimproved by WHO and UNICEF (2013). An improved sanitation facility is defined by WHO as one that hygienically separates human excreta from human contact and not shared by more than one household.

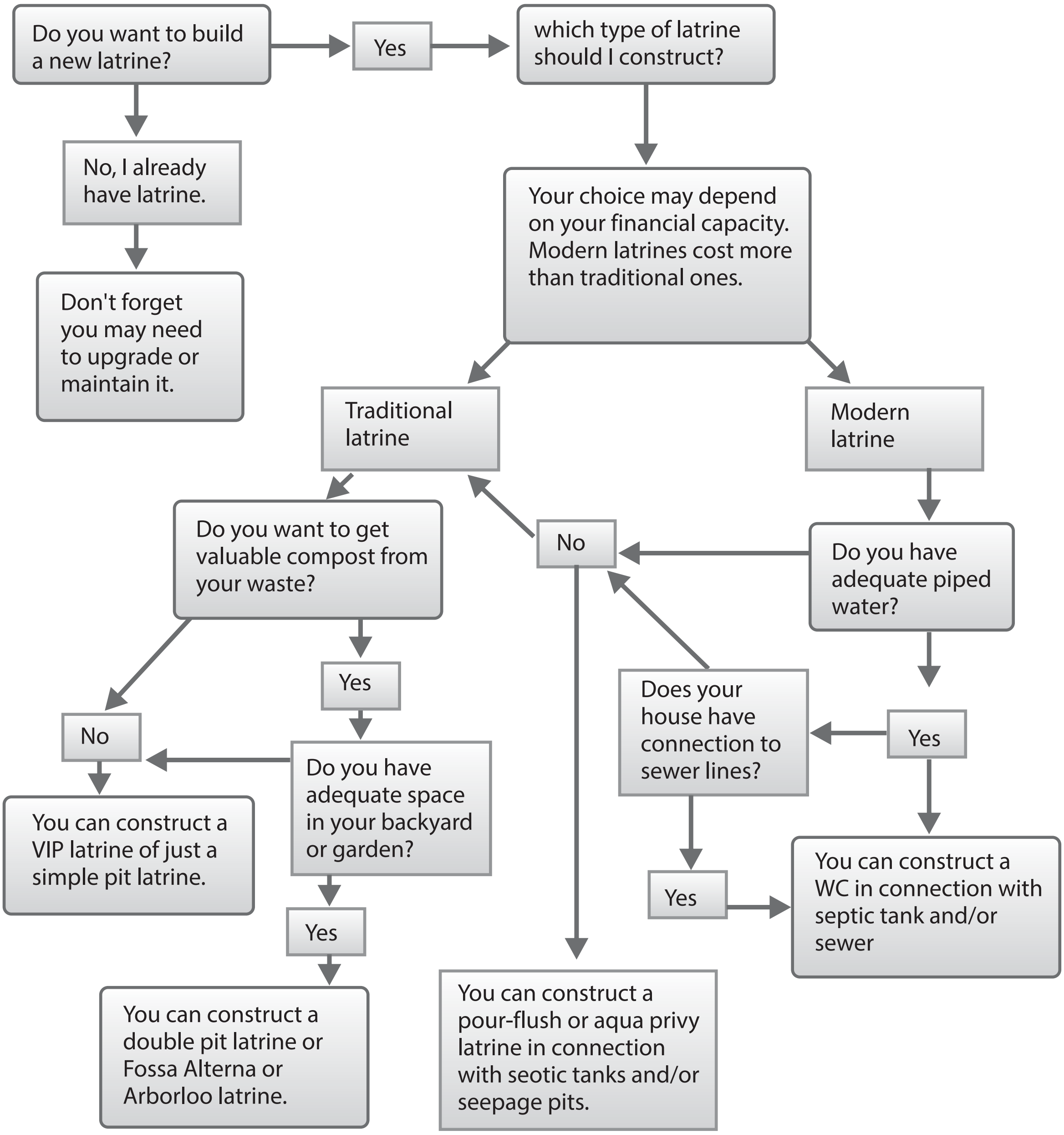


Figure 2.12 Decision tree for latrine options (Source: http://labspace.open.ac.uk).

CHAPTER 2

Box 2.2

The national Sanitation Campaign of Bangladesh

The Government of Bangladesh has taken up an extensive program of "National Sanitation Campaign" in order to ensure construction of sanitary latrines, its use and personal hygiene practice by 100 % of the population to achieve MDGs. The aims of this campaign are to:

- Change the attitude and practice of population towards use of sanitary latrines by creating awareness through cooperation and collaboration of the government, and Non-Government Organizations, Development Partners and better-off people of the society with full commitment at all levels;
- Encourage setting out targets by Local Government Institutions (LGIs) and NGOs in three phases of the years 2005, 2008 and 2010 to achieve the goal of 100 % sanitation coverage;
- Discourage open defecation; and
- Provide importance to maintenance of personal hygiene and capacity building of the population.

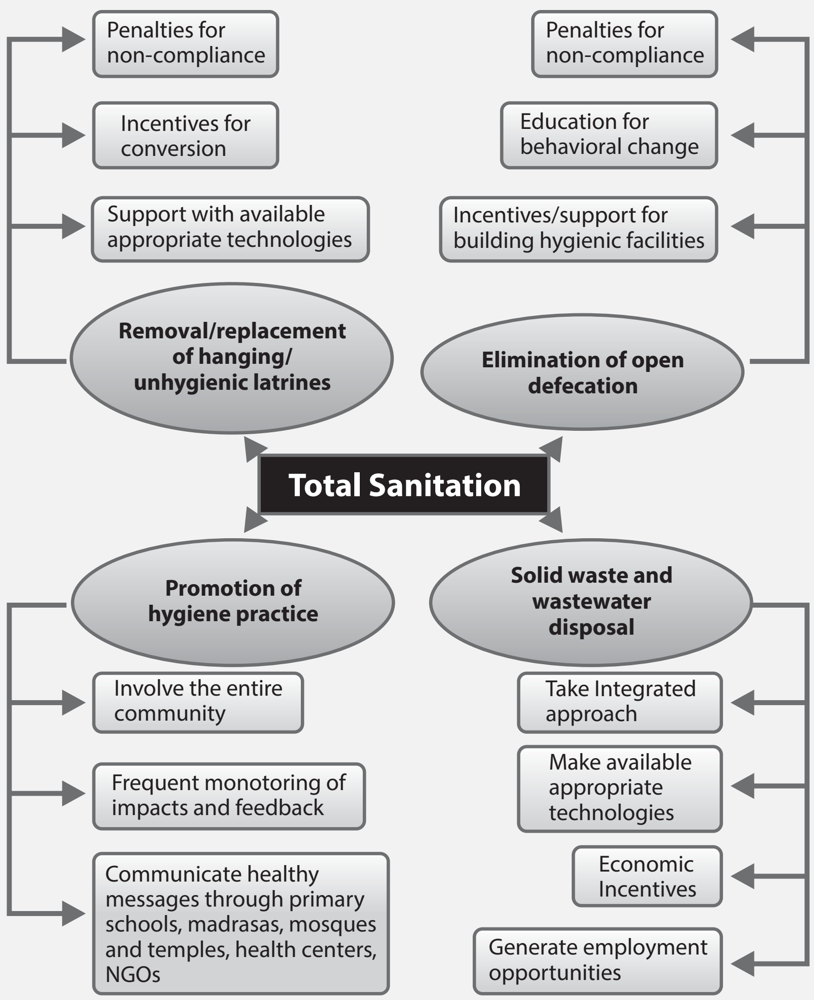


Figure **Objectives of total sanitation campaign.**

The objectives of the total sanitation campaign are mentioned in the Figure above.

Questions

1. Establish a linkage between inadequate sanitation and transmission of epidemic diseases.
2. Write a short note on sanitation challenges.
3. Write short notes on: (a) Reed Odourless Earth Closet; and (b) Ventilated improved pit (VIP) latrines.
4. What are the main advantages of ecological sanitation technologies?
5. Distinguish between cistern flush and pour flush toilets.

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Chapter 3

Sewer Systems

Sewers are underground pipes used to collect and transport the municipal/ industrial wastewater to the disposal point (i.e. natural channels/treatment plants). The network of sewers, that collect the generated wastewater is referred as collection systems. This chapter provides a brief description of on-site and off-site systems, often employed for the collection of wastewater. In addition, a brief description on existing sewerage networks of greater Dhaka has also been included in this chapter, along with future upgrading plans.

3.1 Wastewater Collection Systems

The collection systems of the generated wastewater (from municipal/industrial sources) can generally be classified into three groups as illustrated below.

Sanitary sewers. Sanitary sewers are developed to collect domestic, industrial and commercial wastewaters. These sewer types are useful when rainfall is uneven, for areas with rocky strata, and in areas with steep drainage. Since the sizes of sanitary sewers are usually smaller, they are effective particularly when finance availability is a constraint. However, these sewers often do not promote self cleansing velocities, unless laid at steep gradients. As such, flushing is often mandatory for such systems, which in turn increases operational costs.

Storm sewers. These sewers are employed for the collection of storm runoff from roofs, streets etc., followed by disposal of the collected runoff toward a receiving water channel. Storm sewers are usually larger than sanitary sewers, and are generally operated under gravity flow.

The main advantages of separate sewerage systems (i.e. sanitary and storm sewers) can be enlisted as:

- Smaller waste water treatment works are required.
- Storm water can be pumped when necessary.
- Wastewater and storm sewers may follow optimum route and depth.
- Less variation in terms of wastewater flow and strength.
- Absence of road grit in wastewater sewers.

However there are also some disadvantages of separate sewer systems, such as:

- Additional costs are required for the construction of two separate pipes.
- No flushing of deposited wastewater solids by storm water.
- Lack of storm water treatment.

Combined sewers. In a combined sewer system, domestic, industrial and storm sewage are carried together. The advantages of combined sewer can be illustrated as:

- Lower pipe construction costs.
- Economical in terms of occupying space.
- Allows dilution of sewage, which can reduce input load into the treatment plants.
- Bigger sizes render adequate cleaning provisions.
- Allows some treatment of stormwater.

Despite of the benefits of combined sewers, they also incur some disadvantages as illustrated below:

- Combined sewers allow solid siltation, due to lower flow velocity in dry weather. As such, these systems are not feasible in regions, where annual rainfall distribution is uneven.

- May demand higher pumping cost, if the flow is pumped into treatment plants.
- Can incur greater variation of wastewater flow and strength to treatment plants.
- Grit removal is necessary.

3.2 Classification of Sewers

Despite the types of sewers can vary according to the collection and treatment of wastewater, the major sewer types often found in a network are: building sewer, lateral or branch, main, trunk, and intercepting sewer, as shown in Figure 3.1.

Building sewer. These sewers typically begin outside the building foundation, and are used to convey wastewater from building to lateral or branch sewers.

Lateral/branch sewer. These sewers are usually constructed in streets. They are used to collect wastewater from building sewers, and to convey the collected wastewater to a main sewer.

Main sewer. Main sewers collect wastewater from either a single or multiple lateral sewers, and transfer it to trunk or intercepting sewers.

Trunk sewer. These sewers convey wastewater from main sewers to treatment plants, or other disposal facilities.

Intercepting sewer. These are employed to intercept several main or trunk sewers, and to transfer the wastewater to the treatment plants or other disposal facilities.

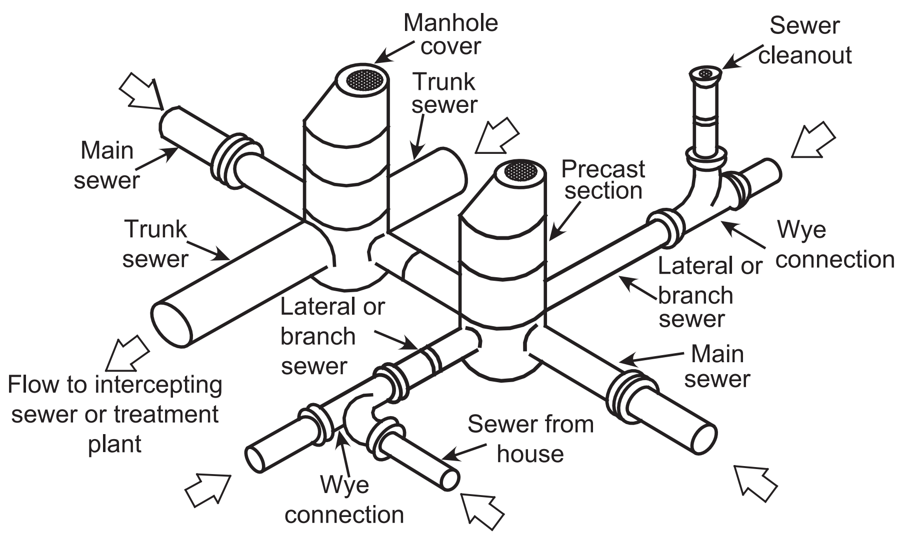

Figure 3.1 Types of sewers in a network.

3.3 Sewage Quantity

The wastewater generally produced from different sectors depends on water usage, life status, economical conditions, climatic changes, public awareness etc. Typical average generation of wastewater in the United States is 265 lpcd. Wastewater generation from commercial facilities ranges between 7.5-28 m^3/ha.d. If the sewer is laid below the water table, ground water infiltration can also contribute to the additional increase of sewage loads

Box 3.1

Sewerage networks of Dhaka (DWASA, 2013)

The sewer system of Dhaka was originally designed as a conventional separate sewerage system, i.e. sanitary sewer system, to transport wastewater from domestic premises and commercial establishments. The sewer systems (of Dhaka) exclude storm/surface water which is managed separately by drainage channels/khals. There are three main recognized trunk sewers in the city:

The Eastern Trunk Sewer: Length of the eastern trunk sewer is 14 km, and diameter ranging from 450 mm to 1360 mm This trunk sewer is routed from Asad Gate to Pagla treatment through the lift stations of Tejgaon, Basaboo and Swamibagh. There are ten pumping stations, such as: Japan Garden City, Asad gate, Bijoy Shawrani, DOHS Banani, DOHS Mohakhali, Tejgaon, Goran, Modertek, Basaboo and Swamibagh. The pumping stations are used for collection and transportation of sewage towards the Pagla sewage treatment plant.

The Western Trunk Sewer: The length of this sewer system is 6 km, and diameter ranging from 600 mm to 900 mm. It is routed from Bashbari and Mohammadpur to Narinda, through Hazaribagh, Nilkhet, Segunbaghicha, Purana Paltan and Motijheel. There are five sewage lift pumping stations associated with this trunk sewer, such as: Hazaribag, New Market, Moghbazar T&T and Zikatola. These lift stations collect wastewater from the related catchments and deliver to the western trunk sewer, which forwards flow by gravity to the Narinda central pumping station.

The South Western Trunk Sewer: The length of the south western trunk sewer is 6 km, and diameter ranging from 400 mm to 1000 mm. It is located in the south-west part of the city, and is routed from Nawabgong to Narinda via Lalbagh, Jailkhan Gate, Abul Hasnat Road, Nawabpur Road and Tipu Sultan. This trunk main was rehabilitated in the year 2003.

into the sewer networks. Such ground water infiltration is dependent on the size of sewer pipes: 3500 to 5000 gal/d per mile of 8 inches sewer, 4500 to 6000 gal/d per mile of 12 inches sewer, and 10000 to 12000 gal/d per mile of 24 inches sewer.

Estimation of storm run-off entering the sewer pipes via manholes is essential for design. ASCE tests on leakage through manhole covers show that 20-70 gal/min of water may enter a manhole cover, submerged by 1 inch of rain water.

The quantity of sewage also fluctuates on hourly and daily basis, depending on the usage pattern. Table 3.1 gives typical ratios of minimum and maximum hourly and daily sewage flow rate, with respect to average flow.

The following equations can be employed to estimate typical variation in daily sewage flows, with respect to Table 3.1.

Table 3.1 Ratio to average and maximum /minimum flows.

Flow description	Ratio to average
Maximum daily	2.5 to 1
Maximum hourly	3.00 to 1
Minimum daily	0.67 to 1
Minimum hourly	0.33 to 1

$$Q_{max(d)} = 2.5 \times Q_{av(d)} \quad 3.1$$

$$Q_{min(d)} = 0.67 \times Q_{av(d)} \quad 3.2$$

$$Q_{av(d)} = Q_{av(sanitary)} + \text{infiltration} + \text{runoff} \quad 3.3$$

$$Q_{av(sanitary)} = 60 - 70\% \text{ water use} \quad 3.4$$

where, Q_{max} = maximum rate of flow of domestic sewage

Q_{av} = average rate of domestic sewage flow

Typical components of sewage and their impact on the environment have been illustrated in Table 3.2.

Table 3.2. Typical sewage components and their environmental impact.

Component	Environmental impact
Pathogen bacteria, virus, worms, and eggs	Impact on human health.
Biodegradable organic	Oxygen depletion in receiving water channels.
Other organics (Detergents, pesticides, fats, oil, grease, solvents, phenol, and cyanide)	Toxic impact, and bioaccumulation.
Nutrients (nitrogen, phosphorus)	Eutrophication, and oxygen depletion of the water bodies.
Metals (Hg, Pb, Cd, Cr, Cu, Ni)	Toxic impact, and bioaccumulation.
Inorganics (acids, bases, and hydrogen sulphides)	Corrosion, and toxic effect.
Odor	Aesthetic inconveniences.

Example 3.1. Determine the maximum hourly, average daily and minimum hourly residential sewage flows from an area occupied by 1000 people. The average per capita sewage flow is 40 gpcd. Consider the sewer length and house connections to be 1.5 miles, and infiltration to be 35,000 gpd.

Solution:

Total average daily flow= $40 \times 1000 = 40000$ gal/d

Infiltration $= 35000 \times 1.5 = 52500$ gal/d

From Table 3.1:

Minimum hourly flow $= 40000 \times 0.33 + 52500 = 65700$ gal/d

Maximum hourly flow $= 40000 \times 3.0 + 52500 = 172500$ gal/d

3.4 Sewer Velocity

Several formulas are available for the hydraulic design of sewers. Among these equations, the formula developed by Robert Manning in 1889, referred as Manning equation is commonly used for accomplishing the hydraulic design of sewers, as illustrated in Equation 3.5.

$$V = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \tag{3.5}$$

The flow rate (Q) can be computed via Equation 3.6:

$$Q = AV = A\frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \tag{3.6}$$

where, V = mean velocity, ft/s
R = hydraulic radius, ft
S = hydraulic gradient
n = manning's roughness coefficient
Q = flow discharge, ft^3/s
A = pipe area, ft^2

The choice of suitable roughness coefficient is the most important factor in sewer design. If the sewer materials consist of concrete, asbestos cement and corrugated steel pipe (with smooth asphalt lining), the typical values of roughness coefficient are: 0.012 for clear water, 0.015 for sewage (0.013 is a common design value). For cast iron pipes, the value ranges between 0.015-0.035.

For partially filled sewer pipes (as shown in Figure 3.2), the Manning equation (Equation 3.5) can be applied. The required parameters angle , area of flow A, wetted perimeter P, and hydraulic radius R can be computed by the following equations.

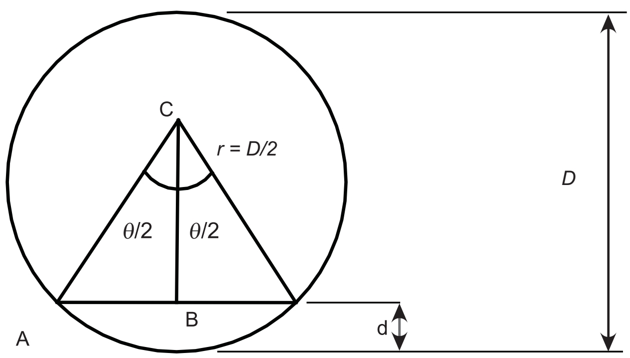

Figure 3.2 Flow in partially filled pipe, circular in shape.

Calculation of θ, $\cos\dfrac{\theta}{2} = \dfrac{r-d}{r} = 1 - \dfrac{d}{r} = 1 - \dfrac{2d}{D}$

$$\theta = 2\cos^{-1}\left(1 - \frac{2d}{D}\right) \qquad 3.7$$

For triangle ABC, the area can be computed as $= \dfrac{1}{2} AB \times BC = \dfrac{1}{2} r \sin\dfrac{\theta}{2}\, r\cos\dfrac{\theta}{2} = \dfrac{r^2}{2}\dfrac{\sin\theta}{2}$

$$= \frac{1}{2}\frac{D^2}{4}\frac{\sin\theta}{2} \qquad 3.8$$

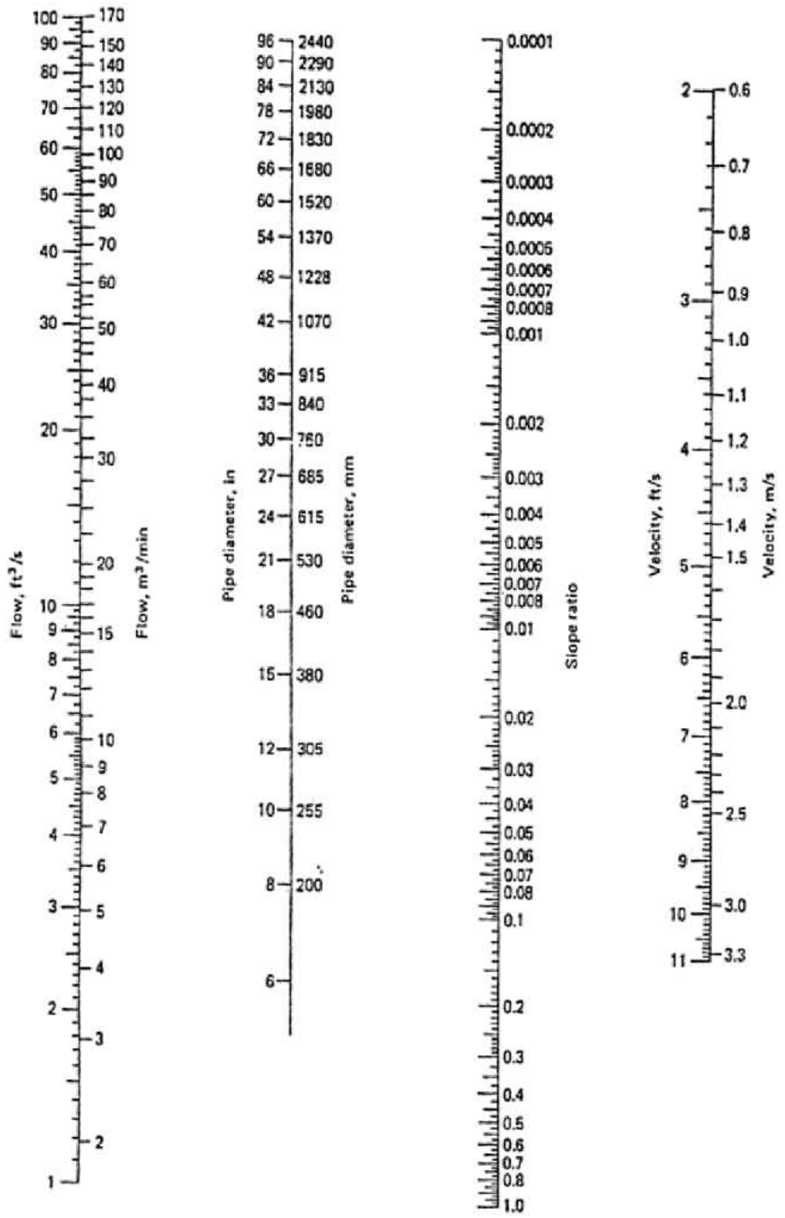

Figure 3.3 **Nomograph for the solution of Manning's equation, for full flowing circular pipes.**

$$\text{Flow area } A = \frac{\pi D^2}{4}\frac{\theta}{360} - 2a = \frac{D^2}{4}\left(\frac{\pi\theta}{360} - \frac{\sin\theta}{2}\right) \qquad 3.9$$

$$\text{Wetted Perimeter, } P = \frac{\pi D\theta}{360} \qquad 3.10$$

$$\text{Hydraulic radius, } R = \frac{A}{P} \qquad 3.11$$

The Manning equation (Equation 3.5) can also be solved by the nomograph, as shown in Figure 3.3. For partially filled circular pipes, with variable Manning roughness coefficient and depth, Figure 3.4 is generally employed.

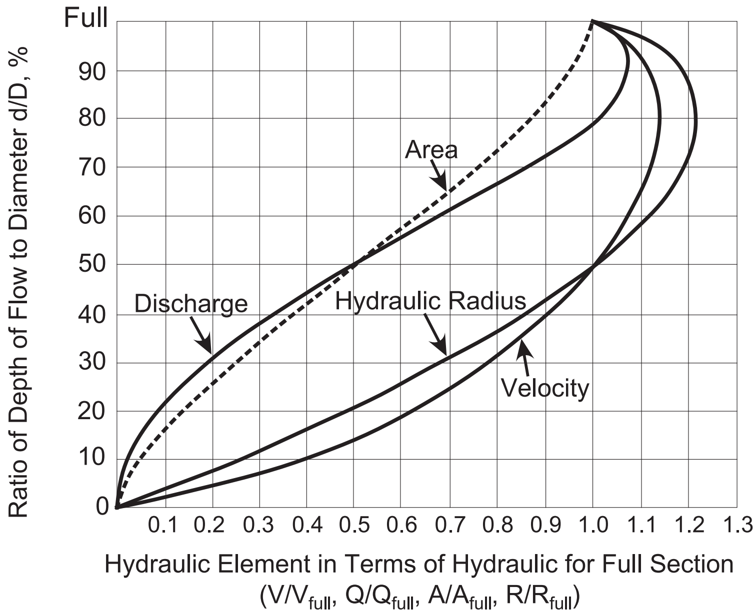

Figure 3.4 **Ratios of sewer hydraulic elements.**

Example 3.2. Flow discharge capacity of sewers. A 20 inches sewer with $n = 0.013$ is laid on a grade of 0.015.

a. What will be the velocity when depth of flow is 5 inches?
b. What will be the capacity when flowing half full?

Solution:

a. Discharge capacity when depth of flow is 5 inches

From Equation (3.7): $\theta = 2\cos^{-1}\left(1 - \frac{2d}{D}\right)$

$$= 2\cos^{-1}\left(1 - \frac{2\times 5}{20}\right) = 120^\circ$$

From Equation (3.9): $A = \frac{D^2}{4}\left(\frac{\pi\theta}{360} - \frac{\sin\theta}{2}\right)$

$$A = \frac{1.66\ \text{ft}^2}{4}\left(\frac{\pi \times 120^\circ}{360} - \frac{\sin 120^\circ}{2}\right) = 0.25\ \text{ft}^2$$

From Equation (3.10): $P = \frac{\pi D\theta}{360} = \frac{\pi \times 1.66\ \text{ft} \times 120}{360} = 1.73\ \text{ft}$

From Equation (3.11): $R = \frac{A}{P} = \frac{0.25\ \text{ft}^2}{1.73\ \text{ft}} = 0.14\ \text{ft}$

From Equation (3.5): $V = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} = \frac{1.486}{0.013} \times (0.14)^{2/3} \times (0.015)^{1/2} = 3.82\ \text{ft/s}$

From Equation (3.6): Discharge capacity, $Q = AV = 0.25\ \text{ft}^2 \times 3.82\ \text{ft/s} = 0.955\ \text{ft}^3/\text{s}$

b. *Capacity when depth of flow is half of the pipe diameter*

The full-flow depth can be determined from Equation (3.6), and can be rearranged as:

$$Q = \frac{D^{8/3} \times \sqrt{S}}{2.16n} \qquad 3.12$$

Flow discharge, at full depth $= \frac{(1.66\ \text{ft})^{8/3} \times (0.015)^{1/2}}{2.16 \times 0.013} = 16.8\ \text{ft}^3/\text{s}$

Discharge capacity when flowing half of the full depth $= \frac{16.8\ \text{ft}^3/\text{s}}{2} = 8.4\ \text{ft}^3/\text{s}$

Example 3.3. Determination of sewer flow velocity by graphical method. A 915 mm (36 inches) sewer is laid in a slope of 0.003; what will be the depth of flow and velocity when the flow is 8.5 m^3/ min?

Solution

From the Figure 3.3, the flow velocity is 1.57 m/s, and flow rate is 62 m^3/min for full-flowing sewer.

The ratio of actual to full flow, $\frac{Q_P}{Q_F} = \frac{8.5}{62} = 0.14$

From Figure 3.4 employing this ratio, the ratio of depth (D_P to D_F) is 0.28;

Using this, from Figure 3.3, $\frac{V_P}{V_F} = 0.75$

So, depth of flow $D_P = 0.28 \times 915 = 256$ mm

Velocity $V_P = 0.75 \times 1.57 = 1.17$ m/s

3.5 Materials of Sewage Pipes

Since the sewer pipes are buried underground, the sewer materials should be able to tolerate stress, resistive to corrosive gases, and should encounter temperature variations. To prevent infiltration of groundwater or exfiltration of sewage, it is essential to select impervious materials. The common materials employed for the construction of sewage pipes include asbestos-cement, ductile iron, reinforced concrete, polyvinyl chloride and virtified clay. The first three materials are highly susceptible to acid corrosion and H_2S attack, whereas the latter two materials are highly resistive to corrosion.

The H_2S corrosion of sewer materials is commonly observed, particularly in sanitary sewers. Organic material often accumulates in sanitary sewers, as a result of deposition at lower flow velocities and grease coagulation. This accumulated material is degraded by the sewage bacteria under anaerobic conditions, thereby producing short chain volatile acids, followed by pH suppression. In addition, biological sulfate reduction also occurs simultaneously. The combination of sulfate reduction and low pH can stimulate the release of H_2S, which can be dissolved in condensed moisture accumulated at the sewer crown. Such dissolved H_2S can be oxidized into H_2SO_4 acid in presence of O_2 (by Thiobacillus bacterium), leading to the sewer failure if the materials are acid soluble.

3.6 Sewer Appurtenances

The major sewer components include inlets, basins, manholes, flushing devices, inverted siphons, vertical drops, energy dissipaters, regulators, outlets and pumping stations. The following sections describe some of the major sewer appurtances.

Inlets. Inlets are the structures, which permit entrance of storm water in the sewers. The location of the inlets is critically dependent on traffic safety. Street inlets are generally placed near the street corner; the distance between the inlets depends on storm water amount, water depth, and the depression to the gutter.

Gutter inlet is more efficient than curb inlet to capture gutter flow. Flow in the street gutter can be calculated by Manning formula (for triangular gutter cross section), as shown in Equation (3.13):

$$Q = K\left(\frac{z}{n}\right)s^{1/2}y^{8/3} \tag{3.13}$$

Where, Q = gutter flow, m^3/s
K = constant = 22.61 $m^3/(\text{min. m})$
z = reciprocal of the cross transverse gutter slope
n = roughness coefficient = 0.015 (for concrete gutters)
s = gutter slope
y = gutter water depth at the curb, m

Manholes. Manholes, generally cylindrical in shape, are used to inter-connect two or more sewers, to provide entry for sewer cleaning and to allow ventilation. For sewers that are 48 inches and smaller, manholes should be located at changes in size, slope and direction (of the

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sewer network). In large sewers, these changes can be made without using a manhole. The spacing of manholes differs within 90-150 m, in straight alignments.

When the elevation difference between higher and lower sewer exceeds 0.6 m, the inflow is usually dropped to the elevation of the outer sewer pipe through drop manhole or drop inlet, as denoted in Figure 3.5. As the sewage falls from higher to lower elevation, the fall is generally interrupted by staggered horizontal plates within the shaft. These devices prevent excessive kinetic energy, thereby hindering the damages of the bottom structure.

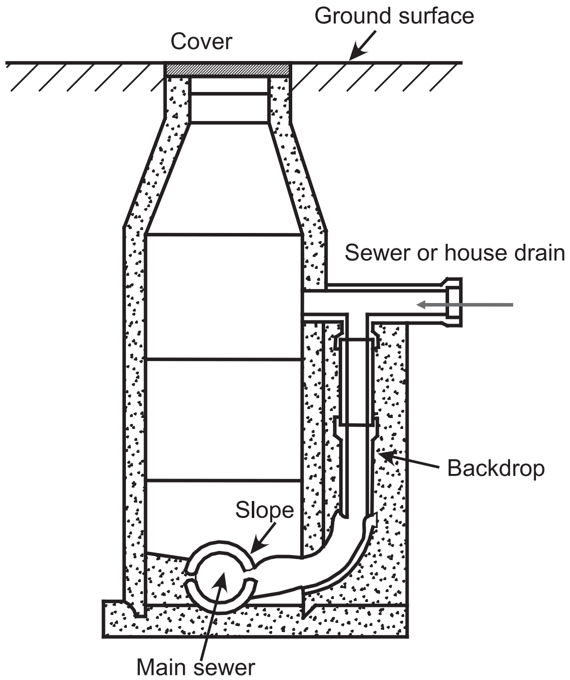

Figure 3.5 **Schematic diagram of a drop manhole.**

Inverted Siphons. An inverted siphon (or depressed sewer) is a section of the sewer, dropped below the hydraulic grade line in order to avoid an obstacle such as a railway, highway cut, subway, and stream. Inverted siphons normally include multiple pipes and an entrance structure designed to divide the flow, so that flow velocity (in those pipes) is sufficient to prevent solid deposition. To maintain such flow characteristics, the velocity in these sections should be kept at least 0.9 m/s for domestic wastewater, and 1.25-1.5 m/s for storm water to prevent soil deposition. The diameter of inverted siphons is generally 150 or 200 mm for sanitary sewers, and 300 mm for storm sewers.

Figure 3.6a shows front view of inverted siphons to avoid an obstacle. Simultaneously, Figure 3.6b shows the design of pipes for inverted siphons to convey minimum flow, difference between minimum and average flow, and the difference between average and maximum flow. The inlet structure has two side flow weirs which direct flow to the central pipes. As the flow increases, the excess flow spills over the lower weir, and enters the second pipe. Further increase in flow diverts it to the third pipe.

Sewer outlets. Sewers discharging wastewater into large water bodies are usually extended beyond the banks into deep water. The outfall lines are constructed of either iron or

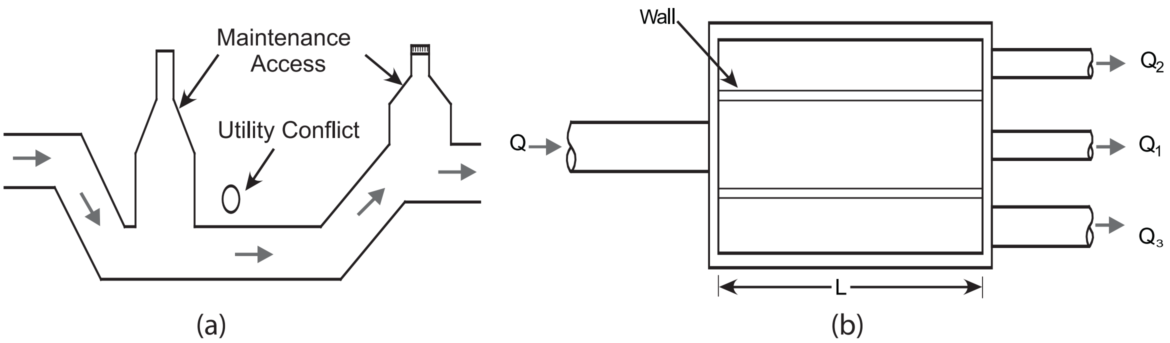

Figure 3.6 (a) front view; and (b) top view of inverted siphons.

reinforced concrete. Iron is generally preferred, having outfalls 610mm diameter or less. For intensified wave action, outfall can be protected, by placing it in a degraded trench.

3.7 Sewers Construction and Maintenance

Excavation techniques. Mechanical experiment should be used for excavation. A chain drive with a bucket can be employed; the bucket cuts the soil, brings it to the surface and discharges it to a conveyor which carries it to the side of the trench. The trench should be excavated 8 inches below the final grade, to allow the placement of bedding material below the pipe. Excavation in rock should be carried below the bottom of the pipe, and to a depth equal to ¼th the pipe diameter or 4 inches (whichever is greater). The space between the pipe and rock is filled with proper bedding material.

Excavation dewatering. The extension of excavators below the ground water promotes passage of fluid. As such, for permeable subsurface the flow can fluidize the soil resulting quick condition and failure of the trenches.

Quick condition can be prevented with the help of well points which can lower the water table. Such wells are situated in parallel positions to the trench which are pumped to depress groundwater table. Well points may be placed along either or both sides of trench, 2m from the centerline, 1m apart, extending well below the trench bottom. These wells are connected with a common pump. For unexpected water problem, bottom trench can be stabilized temporarily with gravel, rock.

If the flow is not substantially higher to cause fluidization, it can be removed by letting it flow along the trench bottom to a sump from which it is being pumped. For large sewer construction, an under drain can be placed in the trench bottom to provide drainage. However after the sewer construction, the drain should be plugged to prevent permanent drainage of soil.

Sheeting and bracing. Unstable trench soil requires sheeting and bracing to prevent collapse of side walls. Sheeting is the support material that is in contact with the excavation wall. Bracing are crosspieces, extending from one side to another. Ranges are members for load transfer from sheet to brace (Figure 3.7).

Vertical sheeting is the strongest method in trench construction, and used in soft soils where groundwater may be faced. Box sheeting consists of horizontal sheeting and vertical rangers;

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these sheeting types are best suited to unconsolidated soils. Poling boards consist of short board pieces against the trenches, supported by rangers and cross bracing (Figure 3.7). Poling boards are used in materials, which can stand without support at a depth of 1.0m-1.5m.

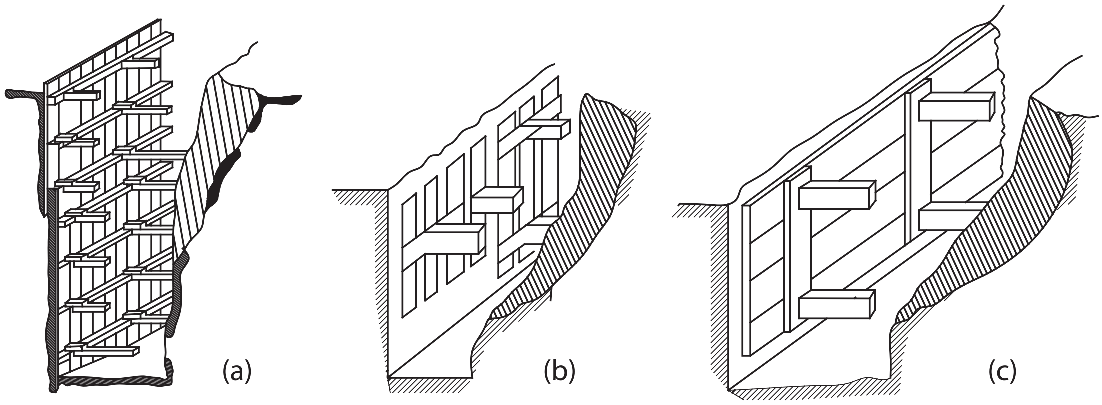

Figure 3.7 **(a) Vertical sheeting; (b) poling boards; and (c) box sheetings for sewer construction.**

Pipe laying. The pipe should be inspected for its soundness and damage. If sections are joined by ring, it should be installed on one end before the assembly is lowered in the trench. The pipe sections are placed on line and grade in the bottom of dewatered trench, and pressed together with a hand lever. The trench should be filled as soon as possible after the pipe installation and inspection. If concrete bedding is needed, backfill is delayed until concrete achieves sufficient strength to support the weight of pipes.

Fill material should be free of bush, debris, rocks. Presence of rock should be inhibited within 900 mm of the top of the pipe, and within 400 mm of the ground surface. Filling materials must be placed in layers; the thickness of each layer should be 150 mm, and tampered under, around and over the pipe to a height of 600 mm above the crown.

Sewer maintenance. Sewer systems can be clogged by root penetration, rain water infiltration causing overflow, deposition of solids (i.e. grease, bones, broken dishware, garbage, concrete, and debris) and defects in manholes/pipes. Root problems can be prevented by eliminating leakage. Troublesome parts can be replaced by iron pipes. The roots (that enter in the sewers) can be removed in sewers (up to 380 mm diameter) by flexible rods with an auger like cutter; the auger can be rotated by hand, or by a machine.

Grease, another critical pollutant, often causes blockage of household sewers. Such sewers can be cleaned with rotating tools, driven by hand or electric motor.

Sand and grit can be removed (from sewers) with buckets pulled by a cable. If deposits are not extensive, they can be removed by devices similar to the turbine cleaner (Figure 3.8); such turbine has a rotating cutter which is positioned in the sewer by a cable, to allow flushing of the deposits.

Flushing of water by fire hydrants can also remove grit deposits, but cannot remove grease and roots. Flushing should be done carefully, as these devices may create backflow into household plumbing fixtures.

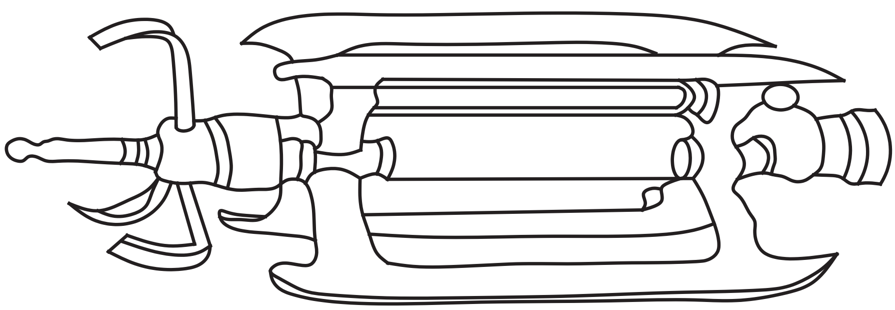

Figure 3.8 Turbine sewer cleaner.

A rubber ball having size slightly less than sewer diameter facilitates in removing grit and grease materials. The ball is being adjusted by the pipe irregularities; water behind the ball escapes around the edges of the ball at higher velocity. Such greater velocity allows the flushing of sewer deposits. The dislodged material (of sewer) should be removed at the next manhole, to prevent downstream blockage. For the maintenance of the broken sewer pipes, the damaged portion may be removed from the sewer line, and the flow can be diverted from manhole to manhole by pumping.

Overall, following steps should be executed by the municipal authorities during maintenance program, to prevent frequent sewer blocking.

- Regular cleaning of sewer lines should be established to remove grease, grit, and other debris, that often block sewer systems.
- Sewer cleaning should be conducted at an established minimum frequency, and more frequently for areas where restaurants are located.
- During routine maintenance the condition of sewers structures should be inspected closely to identify cracks, joint leaks, suspected infiltration/ exfiltration.
- Repairing should be prioritized based on the nature and severity of the problem.
- Previous sewer maintenance records should be reviewed for identifying the areas, that require frequent maintenance.

3.8 Septic System

The term "septic" refers to the anaerobic bacterial environment that develops in the tank and that decomposes or mineralizes the waste discharged into the tank. A septic tank is a key component of the septic system, a small-scale sewage treatment system common in areas with no connection to main sewage pipes provided by local governments or private corporations. Other components, typically mandated and/or restricted by local governments, optionally include pumps, alarms, sand filters, and clarified liquid effluent disposal means such as a septic drain field, ponds, natural stone fiber filter plants or peat moss beds. Septic systems are a type of On-Site Sewage Facility (OSSF). In North America, approximately 25% of the population relies on septic tanks; this can include suburbs and small towns as well as rural areas (Indianapolis is an example of a large city where many of the city's neighborhoods are still on separate septic systems). In Europe, they are in general limited to rural areas only. In Bangladesh, it is a common technology in the suburban areas/towns to treat wastewater as an onsite technology, especially where municipal services are not present.

A septic tank system may include the following:

- sanitary plumbing fixtures connected to drain pipes that enable sewage and sullage wastes to be conveyed from the fixtures to the septic tank
- a septic tank
- a pumping sump
- an effluent disposal system
- a roof, surface and subsurface water disposal system
- single chambered, usually brick built (Figure 3.9) or double chambered, usually brick and concrete built (Figure 3.10).

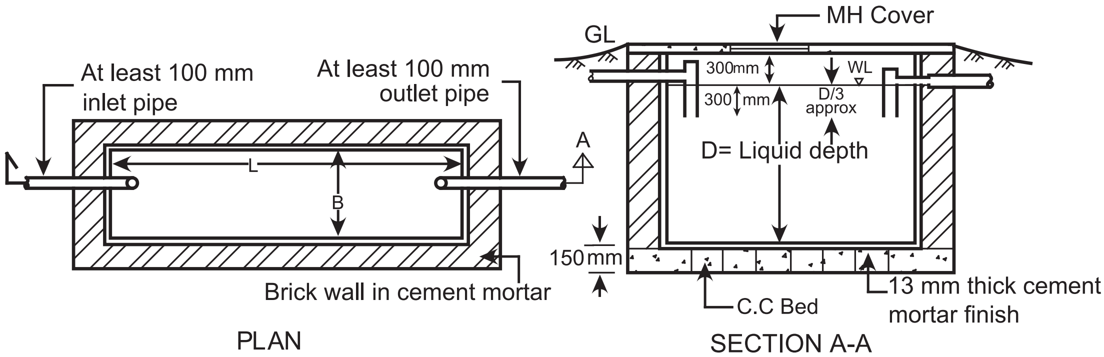

Figure 3.9 **Typical brick built single chambered septic tank.**

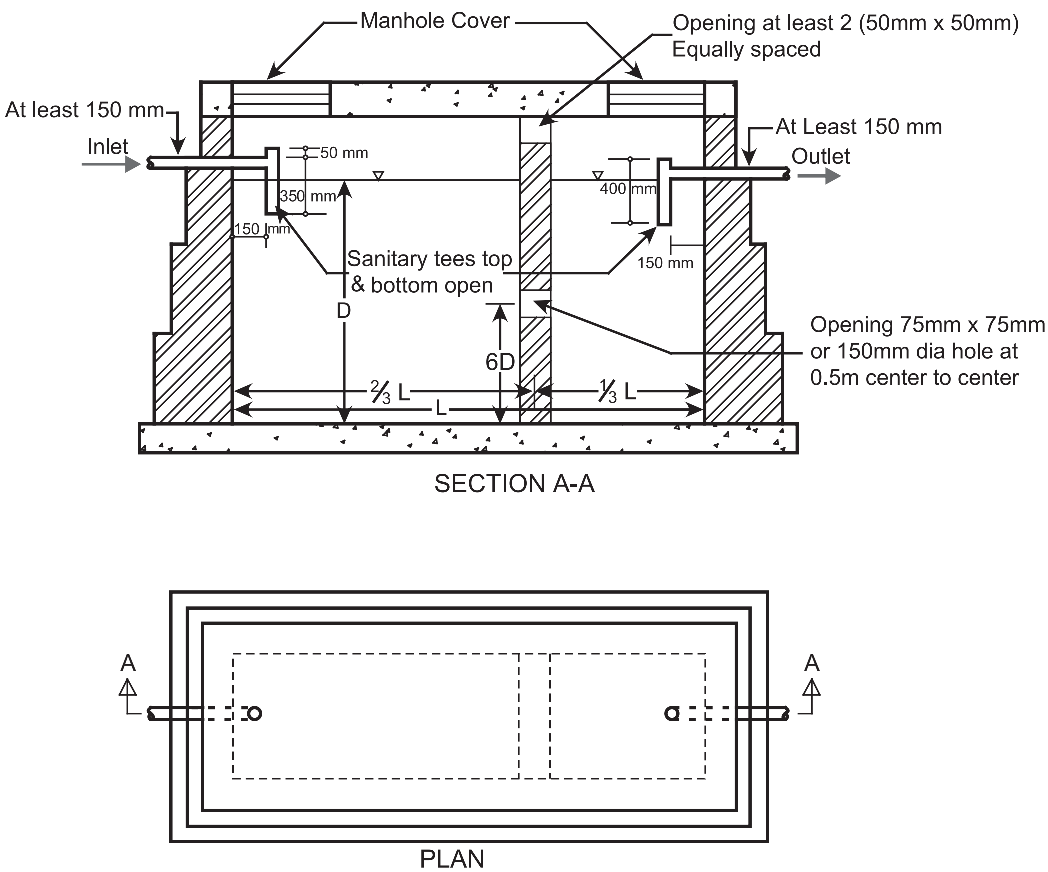

Figure 3.10 **Typical double chambered septic tank (BNBC 2011).**

Where a septic tank effluent disposal scheme is not available, the site must be suitable for subsurface disposal within the allotment boundaries, otherwise an alternative system will be required. This may entail the collection of effluent in an impervious sump for subsequent removal from the property, or the installation of another recognised form of effluent/sullage disposal.

Physical processes of septic system. Septic tanks allow the separation of solids from wastewater as heavier solids settle and fats, greases, and lighter solids float. The solids content of the wastewater is reduced by 60-80% within the tank. The settled solids are called sludge, the floated solids are called scum, and the liquid layer in between is called the clear zone as shown in Figure 3.11. Although the liquid in the clear zone is not highly treated, it is greatly clarified compared to the wastewater entering the tank, the larger particles having migrated to either the sludge or scum layers. Another important function of the tank is storage of these accumulated solids. The tank is sized large enough to hold solids until maintenance (i.e., tank pumping) is performed. The effluent, or wastewater, that leaves the septic tank comes from the clear zone to minimize the solids loading on the downstream components of the system. The baffle, tee, or effluent screen at the outlet is designed to draw from the clear zone retaining floatable or settleable solids in the tank. The settling process requires time to occur, so the tank must be large enough to retain the wastewater in a turbulence-free environment for two to four days. Excessive flow and turbulence can disrupt the settling process as shown in Figure 3.12, so tank volume, size, shape, and inlet baffle configuration are designed to minimize turbulence.

Biological and chemical processes. Septic tank solids include both biodegradable and non-biodegradable materials; although many of the solids will decompose, some solids will accumulate in the tank. Anaerobic and facultative biological processes in the oxygen-deficient environment of the tank provide partial digestion of some of the wastewater components. These processes are slow, incomplete, and odor producing. Gases (hydrogen sulfide, methane, carbon dioxide, and others) result from the anaerobic digestion in the tank and may create safety hazards for improperly equipped service personnel. The gases accumulate in bubbles in the sludge that, as they rise, may re-suspend settled solids. This will elevate the total suspended solids (TSS) concentration in the clear zone and ultimately send more suspended solids to downstream system components. This scenario often results when active digestion occurs during warm temperatures.

Treatment achieved with domestic sewage. The septic tank provides primary anaerobic treatment (dissolved oxygen < 0.5 mg/L) in an onsite sewage treatment system of the raw wastewater. The effluent from the septic tank is typically treated so that it contains 140 - 220 mg/L BOD, 45 - 70 mg/L TSS, and 10-30 mg/L fats, oils and greases (FOGs). If concentrations of biochemical oxygen demands, total suspended solids, and oil and grease from the sewage are expected to be higher than 170 mg/L, 60 mg/L or 25 mg/L respectively, an estimated or measured average concentration must be determined and be acceptable to the local unit of government. System design must account for concentrations of these constituents so as not to cause internal system malfunction, such as, but not limited to, clogging of pipes, orifices, treatment devices, or media.

Factors affecting septic tank performance. The anaerobic digestion processes in tanks are affected by temperature in the tank and by substances that have an adverse impact on biological organisms. Higher temperatures will enhance the rate of biological processes and inhibiting substances will reduce it. Too high of temperatures may liquefy fats, oils and greases (FOGs). Ideal temperatures in the tank allow for FOGs to solidify and bacterial activity to take place. Some factors that affect the way a tank functions include:

- strength (concentration) of the incoming wastewater.
- pH.
- introduction of harsh chemicals, drain cleaners, paint, photo processing chemicals or other inappropriate substances into the waste stream which may affect pH and biological activity.
- introduction of fats, oils and grease (FOG).
- highly variable flow patterns that affect detention time.
- introduction of pharmaceuticals (especially those for chemotherapy and dialysis; long term use of antibiotics, etc.).
- introduction of process discharge, including backwash from a water softener, and; lack of maintenance resulting in excess accumulation of solids, reducing effective volume and reducing detention time.

Advantages of septic tank. Subsurface infiltration systems are ideally suited for decentralized treatment of wastewater because they are buried. The tanks are relatively inexpensive and can be installed in multiple tank installations.

Disadvantages of septic tank. The sludge may pose an odor problem if the sewage remains

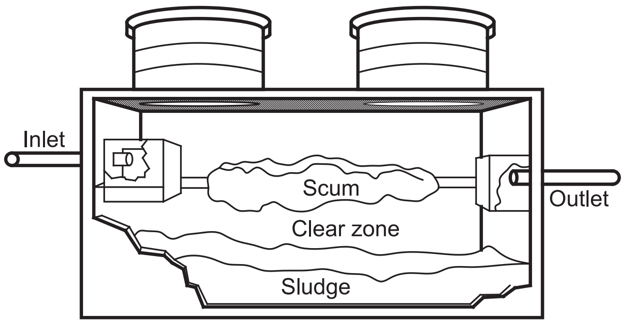

Figure 3.11 Zones of septic tank.

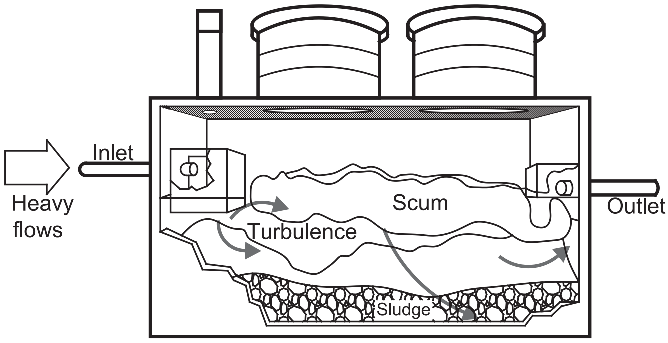

Figure 3.12 Effect of excessive flow and turbulence in septic tank.

untreated for an extended period. Provisions for alarms and pumping are necessary if the downstream treatment units go off-line due to power loss or equipment failure.

Design criteria of septic tank. A holding tank must be the proper size, have a watertight design, and stable structure for proper performance.

- *Tank size:* The size of a tank for a single residence depends upon the number of bedrooms, the number of inhabitants, the home's square footage, and whether or not water-saving fixtures are used. For example, a three-bedroom house with four occupants and no water-saving fixtures would require a 1,000-gallon septic tank. The tank should be designed to hold at least one week of waste flow. Holding tank systems for multiple units should include the above parameters for each residence. Commercial inputs should be evaluated on a case by case basis and may need pretreatment to remove oil, grease, or solids.
- *Tank design:* A key factor in the holding tank's design is the relationship between the liquid surface area, the quantity of sewage it can store, and the rate of wastewater discharged. Each of these factors will impact the tank efficiency and the amount of sludge it retains.
  The greater the liquid's surface area, the more sewage the tank can accommodate. As solids collect in the tank, the water depth decreases, which reduces the time sewage flow is retained in the tank. Less solids will settle in the tank, resulting in increased solids in the tank effluent that may have a negative impact on the final treatment process.
  Placing risers on the tank openings makes it easier to access the tank for inspection and maintenance. If a septic tank is buried more than 12 inches below the soil surface, a riser must be used on the openings to bring the lid to within 6 inches of the soil surface. Generally, the riser can be extended to the ground surface and protected with a lid.
- *Hydraulic loading rate:* The design capacity of the holding tank is related to the hydraulic loading rate of the treatment system. For a ground absorption system, it is determined by soil characteristics, groundwater mounding potential, and applied wastewater quality. Prolonged wastewater loading will clog the infiltrative surface, reduce the capacity of the soil to accept the wastewater, and may back up the wastewater into the holding tank. However, if the loading is controlled, biological activity at the infiltrative surface will maintain waste accumulations in relative equilibrium so that reasonable infiltration rates and pass through in the holding tank can be sustained.

Performance of septic tank. To keep a holding tank system operating efficiently, the tank should be pumped periodically. As the system is used, sludge accumulates in the bottom of the tank. As the sludge level increases, wastewater spends less time in the tank, and solids are more likely to escape into the absorption area. Properly sized tanks can accumulate sludge for at least three years.

The frequency of pumping depends on:

- Tank capacity.
- Amount of wastewater flowing into the tank related to size of household(s).
- Amount of solids in the wastewater. For example, there will be more solids if garbage disposals are used.
- Performance of the final treatment system.

Operation and maintenance of septic tank. A well-designed holding tank requires limited operator attention. Management needs include tracking system status, testing for solids accumulation, evaluating pump performance, and monitoring system controls. Monitoring performance of pretreatment units, mechanical components, and wastewater ponding levels above the filtration surface is essential. If a performance or level change is noted, the operator should inspect the system to determine if additional service is required. Routine servicing of a holding tank is limited to annual or semiannual inspection and cleaning, if necessary.

CHAPTER 3

Septic tank capacity estimation.

a) For residential dwellings:

The following Table 3.3 shows the typical volume needed to construct septic tank for residential dwellings

Table 3.3 Typical volume of septic tank for residential dwellings.

All wastes	Litres
Receiving all water closet pan and household liquid wastes for up to 6 persons	3000
For each additional 2 persons	Add 1000
Sewage only	**Litres**
Receiving sewage from a water closet pan for up to 6 persons	1620
For each additional 2 persons	Add 540

For multiple occupancy residential premises such as flats, units and town houses the capacity of the septic tank is calculated on the basis of total number of bedrooms plus one bedroom and multiply by 2 persons per bedroom.

b) For non-residential dwellings:

Calculation of the septic tank for non-residential installations requires determination of two factors:

- volume for accumulation of sludge/scum.
- volume for daily flow, into the septic tank.

The effective capacity is obtained by calculating: $(S \times P1 \times Y) + (P2 \times DF)$

where, S = Rate of sludge/scum accumulation per person per year
$P1$ = Number of persons using the system
Y = Desludging frequency
$P2$ = Number of persons using the system
DF = Daily inflow in litres per person per day

Table 3.4 provides a range of load factors (S, $P1$, $P2$ and DF) to assist in determining the capacity of the septic tank, and it may be necessary to add a number of individual uses to obtain the sludge/scum and daily inflow total.

Where the system load varies from day to day, an average load figure is used to calculate capacity for sludge/scum accumulation. Calculation of the daily inflow shall always be made using the maximum daily load. The minimum capacity of any non-domestic septic tank shall be 1620 litres or the effective capacity as calculated using Table 3.4, whichever is the greater. The data presented in the Table is provided to assist with determining the capacity of the septic tank based on use conditions. In some cases it may require the addition of a range of uses to obtain the total capacity.

Where the specific use is not listed, it may be necessary to select a similar use to determine the capacity. The rates listed below should be used, except in situations where actual rates have been determined from appropriate monitoring of the sludge scum accumulation rate and water meter readings that exclude non septic tank use, i.e. garden, swimming pool, evaporative cooling etc. The term average or highest daily number over an "x" day period means the highest number in any 12 months period.

CHAPTER 3

Table 3.4 **Typical value of parameters determining capacity of septic tank based on use conditions.**

Parameters	Fixtures	Sludge/scum rate		Daily inflow rate	
		Number of persons	Rate L/person/year	No. of persons	Rate L/persons/day
		P1	S	P2	DF
Hospital & Nursing homes					
Accommodation and resident staff	WC/urinal basin bath/shower laundry kitchen sink dishwasher	Total number or beds plus resident staff	80	Total number of beds plus resident staff	150
Non resident staff	WC/urinal basin kitchen sink (tea service area only)	Number of employees per shift × number of shifts	25	Number of employee's per shift × number of shifts	30
	with shower			As above	10
Hotels/Motels					
Accommodation	WC/urinal basin bath/shower kitchen sink laundry	Total number of beds (single equivalents)	48	Total Number of beds (single equivalents)	100
Non resident staff	WC/urinal basin kitchen sink (tea service area only)	Number of employees per shift × number of shifts	25	Number of employee's per shift × number of shifts	30

Parameters	Fixtures	Sludge/scum rate		Daily inflow rate	
		Number of persons	Rate L/person/year	No. of persons	Rate L/persons/day
		P1	S	P2	DF
Public toilets					
	WC/urinal basin	Average daily number over 7 day period	20	Highest daily number over 7 day period	5
	shower provided	as above	5	as above	10
Restaurants					
	WC/urinal basin kitchen sink dishwasher	Average daily number over 7 day period plus staff	35	Highest daily number over 7 day period plus staff	15
Schools					
with canteen facilities	kitchen sink dishwasher	Total number of students plus staff	10	Total number of students plus staff	5
Staff	WC/urinal basin kitchen sink (tea service area only)	Number of employees per shift × number of shifts	25	Number of employee's per shift × number of shifts	30
Public	WC/urinal basin	average daily number over 7 day period	20	Highest daily number over 7 day period	5

3.9 Small Bore Sewer Systems

Small bore sewer systems (Figure 3.16), which is sometimes referred as SBS system are designed to receive only the liquid portion of household wastewater for off-site treatment and disposal. Grit, grease and other troublesome solids which might cause obstruction in the sewers are separated from the waste flow in interceptor/septic tanks installed upstream of every connection to the sewers; the solids which accumulate in the tanks are removed periodically for safe disposal. However, if the wastewater is channel is not connected to any offsite system/wastewater treatment sites, the system is called septic system or sometimes simply it is termed as conventional sewerage system (Option A of Figure 3.16)

Advantages of using SBS system. Collecting only settled wastewater using SBS sytem has four principal advantages:

- **Reduced water requirements.** Since the sewers are not required to carry solids, large quantities of water are not needed for solids transport. Thus, unlike conventional sewers, small bore sewers can be employed without fear of blockages where domestic water consumption is low, where water-saving plumbing fixtures and appliances are widely used, or where long flat runs with few connections are necessary.

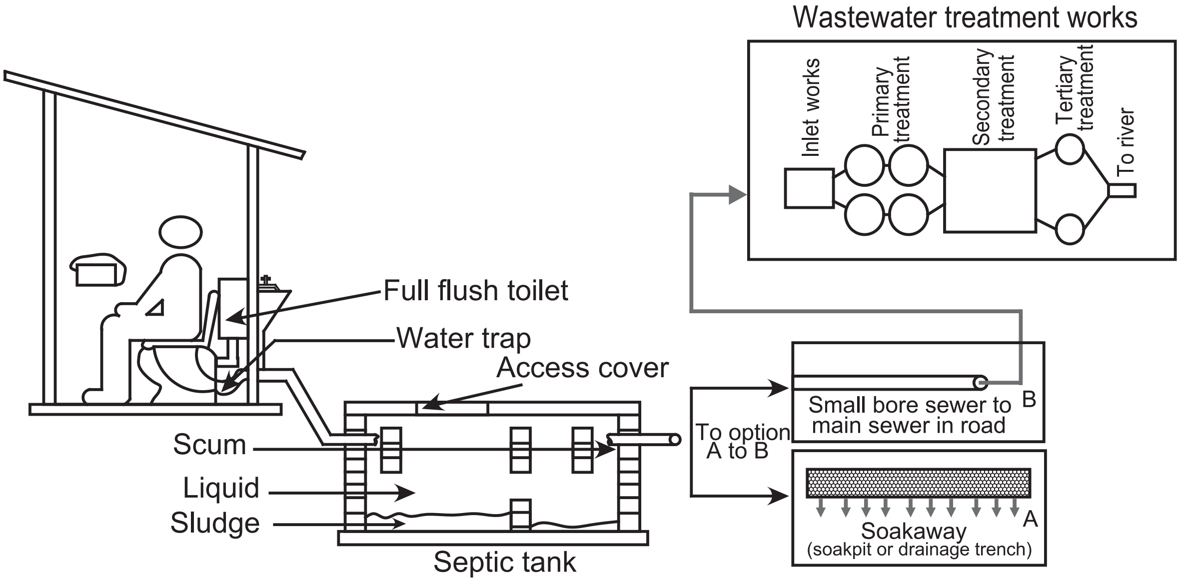

Figure 3.13 SBS (Option B) and conventional (option A) sewerage system.

- **Reduced excavation costs.** With the troublesome solids removed, the sewers do not need to be designed to maintain a minimum flow velocity for self-cleansing. Therefore, rather than being installed on a straight path with a uniform gradient, they may be laid with curvilinear alignment with a variable or inflective gradient. This reduces excavation costs, since the sewer can follow the natural topography more closely than conventional sewers and avoid most obstructions within its path.
- **Reduced materials costs.** Peak flows which the small bore sewers must be designed to handle are lower than those experienced with conventional sewers because the interceptor tanks/septic tanks provide some surge storage which attentuates peak flows. Therefore, the sewer and any pumping equipment can be reduced in size (and pumps handling only liquids are simpler). In addition, expensive manholes can be replaced with much less costly cleanouts or flushing points, since mechanical cleaning equipment is not necessary to maintain the sewers in a free-flowing condition.
- **Reduced treatment requirements.** Screening, grit removal and primary sedimentation or treatment in anaerobic ponds is not needed at the treatment works, since these unit processes are performed in the interceptor tanks.

Thus, small bore sewer systems provide an economical way to upgrade existing sanitation facilities to a level of service comparable to conventional sewers. Because of the lower costs of construction and maintenance and the ability to function with little water, small bore sewers can be used where conventional sewerage would be inappropriate. Small bore sewers therefore offer an opportunity of improving sanitation in areas which otherwise might not be upgraded.

Disadvantage of SBS system. The principal disadvantage of the small bore sewer system is the need for periodic evacuation and disposal of solids from each interceptor tank in the system. Experience with the system is limited and mixed. Consequently, in spite of its obvious advantages it must be used judiciously and adopted only in situations where there is sufficient provision to ensure a strong organization for maintenance. This organization must also be able to exercise effective control over connections to the system. Special precautions should be taken to prevent illegal connections, since it is likely that interceptor tanks would not be installed in such connections, thereby introducing solids into a system which is not designed to handle solids. This could create serious operational problems.

Component parts of SBS system. Small bore sewer systems consist of: (a) house connections; (b) interceptor tanks; (c) the sewers and their appurtenances; and (d) a sewage treatment plant. Occasionally, individual pumping stations may be required to lift the effluent from the interceptor tank into the sewer to overcome adverse elevation differences; additionally, pumping stations may be required in the sewer system itself in very flat areas.

- **House connection.** The house connection is made at the inlet to the interceptor tank. All household wastes, except for garbage and trash which must be removed for disposal elsewhere, enter the system at this point. Storm water must be excluded.
- **Interceptor/septic tank.** The interceptor tank is a buried watertight tank with baffled inlet and outlet. It is designed to detain the liquid flow for 12 to 24 hours and to remove both floating and settleable solids from the liquid stream. Ample volume is also provided for storage of the solids, which are periodically removed through an access port. Typically, a single-chamber septic tank is used as an interceptor tank.
- **Sewers.** The sewers are small bore plastic pipe (minimum diameter of 100 mm) which are trenched into the ground at a depth sufficient to collect the settled wastewater from most connections by gravity. Unlike conventional sewers, small bore sewers are not necessarily laid on a uniform gradient with straight alignment between manholes or cleanouts. The sewer may have an inflective gradient; that is to say, the sewer may have dips so that sections of it remain full under static conditions. Also, the alignment may curve to avoid natural or manmade obstacles. The objective in the design and construction of small bore sewers is to utilize to the maximum extent the energy resulting from the difference in elevation between the upstream and downstream ends.
- **Cleanouts and manholes.** Cleanouts and manholes provide access to the sewers for inspection and maintenance. In most circumstances, cleanouts are preferable to manholes because they cost less and can be more tightly sealed to eliminate most infiltration and grit which commonly enter through the lids and walls of manholes. Also, they can be easily concealed to prevent tampering. They function as flushing points during sewer cleaning operations.
- **Vents.** The sewers must be ventilated to maintain free-flowing conditions. Vents within the household plumbing are sufficient, except where inflective gradient sewers are installed. In such cases, the high points of the sewer should be ventilated either by locating the high points at connections or by installing a cleanout with a ventilated cap.
- **Lift stations.** Lift stations are necessary where elevation differences do not permit gravity flow. Either residential or major lift stations may be used. Residential lift stations are small lift stations pumping wastes from the interceptor tank of one home or of a small

cluster of homes to the sewer, while major lift stations are located in the sewer line and service all connections within a larger drainage basin. Whenever pumping becomes necessary, the difference in total annuitized costs between a small bore sewer system and conventional sewerage may be greatly reduced. Consequently, a close cost comparison would be required between the two before selection between them. Therefore, the most important characteristic of small bore sewers is that they are designed to handle only the liquid portion of domestic wastes. Although the term "small bore sewers" has become commonly accepted, it is not in fact a very accurate description of the system, since the pipes need not be small diameter (the size being determined by hydraulic considerations and not constrained by other conditions), and the pipe system is not designed according to sanitary sewer practice. A more accurate description would be "solids-free sewers", but the best term is probably "effluent drains", as is used in the systems widely employed in Australia; this emphasizes the essential purpose of the sewers to remove liquid effluents (from interceptor/septic tanks) that cannot otherwise be disposed of on site and so forms a natural link to the most likely application of small bore sewers in developing countries: to upgrade on-site disposal systems such as pour-flush latrines when changes in water use, housing densities or other conditions lead to difficulties in on-site effluent disposal. However, for consistency with other recent publications on this subject, in this book, the authors have retained the term "small bore sewers (SBS)".

Analogy between SBS and conventional sewerage system. In new schemes small bore sewerage often appears to have little advantage over conventional sewerage when compared to the latter in present worth terms. Yet the distribution of its costs between capital investment and operation and maintenance is quite different to those of conventional systems and is generally more appropriate to developing country conditions: capital costs with their commonly high foreign exchange requirements are lower; less skill is required in its construction; and its operation and maintenance are quite different to those of conventional systems and are more labor-intensive, and these costs (which are generally lower than those for conventional systems) can be mainly paid for in local currency out of revenue. Small bore sewerage is inherently a much more flexible system than conventional sewerage and its feasibility should always be evaluated during the technology selection stage of a feasibility study since it can offer a viable solution in many situations where conventional systems are technically or economically infeasible.

Hydraulic design of SBS system. Unlike conventional gravity sewers which are designed for open channel flow, small bore sewers may be installed with sections depressed below the hydraulic grade line. Thus, flow within a small bore sewer may alternate between open channel and pressure flow. In making design calculations, separate analysis must be made for each sewer section in which the type of flow does not vary and the slope of the grade line is reasonably uniform. Manning's equation (Equation 3.5) may be used in this analysis.

Box 3.2

Learning from failure of SBS systems at Mirpur, Dhaka (DWASA, 2013).

The area of Mirpur, located approx. 12km north-west from the centre, covers an area of about 6,000ha and entirely lies within MODS Zone 4 of DWASA. In 1991 the Dhaka Urban Infrastructure Improvement Project (DUIIP) was a multi-stakeholder initiative to improve sanitation, inter alia, in Mirpur. A Small Bore Sewer (SBS) System was constructed which is an option for sewage collection in developing countries when financial and water resources are limited. However the small bore system was not effectively commissioned. Reasons which led to failure of the system are mentioned below:

1. The institutional responsibilities were not defined for the various activities required to assure operations and maintenance.
2. Lack of maintenance of the sewer system and de-sludging of the interceptor tanks may have lead to early failure of the system.
3. Only 50-60% of the original appraised pipe connections to interceptor/septic tanks were installed by the DUIIP. Un-served houses were probably never connected to the sewer pipes in order to minimize expenditure.
4. The SBS system was developed for low income areas with single storey houses and one toilet per family. Presently most of the low income areas have developed to middle income areas with fully developed multi-storey buildings. The SBS system was reportedly not designed to serve the increased number of people and it is likely that parts of it have been destroyed during later development. The sizing of the pipes and interceptor tanks cannot meet the current needs in the middle income areas.
5. Community participation is a critical factor to ensure proper O&M of the facilities and ultimate delivery of services. Periodic cleaning of the interceptor tanks is the responsibility of the owners. However there are no septic tank sludge management facilities available in Dhaka City and there is limited incentive for sludge tank emptying. The awareness of periodic interceptor tank de-sludging has not developed, as most of the tanks are not in use and are currently filled with garbage.

Based on the experience in Mirpur, the SBS system is not considered suitable for areas of excessive population growth which includes:

- The DWASA service area except in specific conditions, e.g. in slum areas where development is not expected to grow to high rise buildings.
- Area within the RAJUK area which are adjusted to the DWASA service area (i.e. across the natural river boundaries) where there is a risk of fast development dueto population overspill.
- Pourashavas within the RAJUK area which are subjected to fast growth (e.g.>4% anaual) caused by concentrated industrial and commercial activities.

3.10 Wastewater Production and Sanitation Coverage in Dhaka

Dhaka presently relies heavily on ground water, with approximately 80-90% of demand coming from this source. Groundwater is being extracted primarily from the upper Duptilia aquifer and the lower Duptilia aquifer under Dhaka city. DWASA also operates some surface water treatment plants, such as Saidabad treatment plant Phase 1 (capacity of 225 ML/d), and Phase 2 (capacity of 225 ML/d).

Table 3.5 illustrates drinking water supply and wastewater generation rates in different zones of Dhaka, and Narayongonj (an adjacent district of Dhaka district). Table 3.5 indicates that drinking water supply in Dhaka and Narayongonj is predominantly dependent on groundwater source.

The produced wastewater (in and around Dhaka) as illustrated in Table 3.5 is managed mainly via onsite sanitation and offsite treatment technologies. Within the DWASA service area, it is estimated that only 20% of the population is potentially served by the centralized separate sewerage system. Another 30% of the population is estimated to dispose of sewage by connecting into the drainage networks and open channels. An estimated population of approx. 320,000 in Mirpur area was designed to be served by a small bore water- borne sewerage system, which was never utilized; the generated sewage from these areas is also directed to the drainage system. Improved on-site sanitation is estimated to be the sanitation system utilized by approx. 25% of the population within the DWASA service area, with the remaining 22% served by unhygienic on-site sanitation means, including pit, hanging latrines and open spaces. These figures are shown in the following table 3.6 (DWASA, 2013).

CHAPTER 3

Table 3.5 Water production and wastewater generation rates in different zones of Dhaka (DWASA, 2013).

Zone	Area (km^2)	Pop. (million)	Existing water production (MLD)				Water consumption (60% of production) (MLD)	Wastewater (70% of total) (MLD)
			DTW production (MLD)	WWTP (MLD)	PTW (39.43% of groundwater) (MLD)	Total water production (MLD)		
Zone-1	27.53	1.58	187.15	49.14	121.83	358.12	214.87	150.41
Zone-2	9.16	0.94	164.00	76.60	106.76	347.36	208.42	145.89
Zone-3	24.18	1.09	278.00	12.32	180.97	471.29	282.77	197.94
Zone-4	26.03	1.33	180.00	---	117.18	297.18	178.31	124.82
Zone-5	15.76	0.65	123.00	26.09	80.07	229.17	137.50	96.25
Zone-6	28.21	1.63	194.00	55.79	126.29	376.08	225.65	157.95
Zone-7	29.49	0.77	101.00	16.07	65.75	182.82	109.69	76.78
Zone-8	47.89	0.74	106.00	---	69.00	175.00	105.00	73.50
Zone-9	62.94	0.69	126.00	---	82.02	208.02	124.81	87.37
Zone-10	32.55	0.80	145.00	---	94.39	239.39	143.63	100.54
N.gonj	54.77	1.24	43.26	18.57	28.16	89.99	53.99	37.80
Total	**358**	**11.46**	**1,647**	**254**	**1,072**	**2,974**	**1,785**	**1,250**

Note: DTW = Deep Tubewell, PTW = Private Tubewell.

3.11 On-site Sanitation Technologies of Dhaka: Septic Tanks

In Dhaka a large number of septic tank systems are utilized in areas which are not covered by the conventional sewerage system. The soil properties of Dhaka have low porosity and are not suitable for the drainage of septic tank effluent; as such, the overflow is often discharged

Table 3.6 Total sanitation coverage by population in Dhaka (DWASA, 2013).

	Current population estimated to be covered by the various sanitation systems				
	Separate Sewerage	Combined Sewerage	Small Bore System	Improved On-site Sanitation	Unhygenic Sanitation
2010 population	2,110,000	3,200,000	320,000	2,600,000	2,300,000
% of total population	20%	30%	3%	25%	22%

to a nearby drainage system. The sludge, collected and accumulated in the tanks, should regularly be removed and disposed of in a safe and controlled manner; however there are no septic tank sludge treatment and disposal facilities in and around the city. The sludge collected by private septic tank cleaning services is generally disposed into local low lands, drains and khals. Such collected septic sludge is also disposed into municipal waste collection points, on road sides, in drainage canals and into sewer lines via manholes. This practice is illegal, highly hazardous, unregulated and contributes to the uncontrolled spreading of pollution and pathogenic organisms over large areas.

Based on previous estimates of septic tank usage of 45% in the non-sewered areas, a zone-wise scenario of the population using septic tanks is given in Table 3.7.

Table 3.7 Septic tank usage in DWASA service area (DWASA, 2013).

Zone	Pop. (million)	Sewer coverage (%)	Unsewered (%)	Septic tank coverage (45% of unsewered)	Septic tank use population (million)
Zone-1	1.58	58.47%	41.53%	18.69%	0.295
Zone-2	0.94	54.37%	45.63%	20.53%	0.193
Zone-3	1.16	42.43%	57.57%	25.91%	0.30
Zone-4	1.33	0%	100.00%	45.00%	0.60
Zone-5	0.65	54.41%	45.59%	20.52%	0.133
Zone-6	1.62	26%	74.00%	33.30%	0.539
Zone-7	1.35	0%	100.00%	45.00%	0.61
Zone-8	0.78	0%	100.00%	45.00%	0.35
Zone-9	0.55	0%	100.00%	45.00%	0.25
Zone-10	0.20	0%	100.00%	45.00%	0.09
Naryanganj	1.38	0%	100.00%	45.00%	0.62
Kamrangir char	0.19	0%	100.00%	45.00%	0.09
DND (part)	0.72	0%	100.00%	45.00%	0.32
Total	**12.44**	**21.74%**	**78.26%**	**35.22%**	**4.39**

Table 3.7 indicates that a significant number of population of Dhaka utilize septic tanks, that contribute to substantial dry solids production. The following table estimates the annual septic tank sludge production in each zone in the DWASA service area, assuming an estimated sludge production of 0.25L/P/day.

Table 3.8 Septic tank sludge production in DWASA service area (DWASA, 2013).

Zone	Septic tank use Pop. (million)	Liquid volume (m^3/d)	Dry solid (t/d)	Dry solid (t/year)
Zone-1	0.295	69	8.85	3231
Zone-2	0.193	45	5.79	2114
Zone-3	0.30	70	9	3285
Zone-4	0.60	141	18	6,570
Zone-5	0.133	31	3.99	1457
Zone-6	0.539	126	16.17	5902
Zone-7	0.61	143	18.3	6,680
Zone-8	0.35	82	10.5	3,833
Zone-9	0.25	59	7.5	2,738
Zone-10	0.09	21	2.7	986
Naryanganj	0.62	145	18.6	6,789
Kamrangir char	0.09	21	2.7	986
DND (part)	0.32	75	9.6	3,504
Total	**4.39**	**1028**	**131.7**	**48075**

Since there are no formal collection, transportation, treatment and disposal systems of septic tank sludge in DWASA service area, a proposal has been placed by DWASA for septic tank sludge management, as illustrated in Box 3.3. A proposal for the treatment of such transported septic tank sludge has also been proposed by DWASA, as indicated in Box 3.4.

Box 3.3

Proposal for Septic Tank Sludge Management (DWASA, 2013).

Options for septic sludge management

Options to ensure proper systematic control of the septic sludge management may include: i) DWASA takes the lead role in septic tank sludge management (either via own sludge collection vehicles or through service agreements with service providers), or ii) allowing private sector service providers to deal directly with the households, and DWASA takes a monitoring function. In any case, a department/division (sludge management division) should be created within DWASA, similar to the Water Supply and Sewerage Divisions of DWASA, or the Solid Waste Division of DCC. All households with septic tanks should pay a septic tank cleaning fee in order to recover costs for septic sludge management.

Engagement of sludge contractors and other measures

In the short term, DWASA would engage one or more contractors for the collection and transportation of septic tank sludge to a special facility located at a wastewater treatment plant, operated by DWASA. The possibility of utilizing an NGO as a contractor for these operations can be considered.

Applying the Service Fee
DWASA may consider adding sludge collection charges to the water bill and outsource the collection and transport of sludge. In order to encourage households, septic tank owners shall be required to pay a fee whether they wish to empty their septic tanks or not. There will be a provision that DWASA may do the inspection of each septic tank of the house owners.

Public Awareness
An intensive publicity drive should be implemented, by means of advertising in printed and electronic media along with rallies and workshops. The purpose should be the creation of public awareness about the benefits of the paid service, in contrast with the danger of sludge disposal into drains, canals and land, and its hazardous impacts to water, air and the environment in general.

Box 3.4

Septic Tank Sludge Treatment (DWASA, 2013).

In order to provide septic tank sludge reception, treatment and disposal facilities, it is proposed to construct septic tank sludge treatment systems at the centralized wastewater treatment plants, which will include a screening facility (nominally 10 mm gap width), anaerobic sludge treatment in ponds, drying and disposal. As an urgent priority, it is recommended to pilot scale this type of system at the Pagla Sewage Treatment Plant (PSTP), in order to gain experience in construction and operation procedures for future projects. Initially, such pilot scale testing might include DWASA in developing, and implementing a service contract with a septic tank service provider for a specified region adjacent to Pagla STP. However this should be broadened to allow private contractors to approach households directly throughout the city, once septic tank sludge facilities are constructed at the various treatment plant sites. Consequently it will be essential for the pilot scale test to determine the market interest and associated costs for private sector participation. Concerning the septic tank sludge treatment facilities, in the case of Pagla, there are two options: a) introduction into the existing sludge lagoons; and b) introduction into the treatment plant via a dedicated process such as Anaerobic Primary Settlement Ponds (APSP). Option b is more advantageous, but would require an upgrading of Pagla treatment plant in order to be effective.

3.12 Sewerage System of Dhaka

Dhaka central sewer consists of networks of relatively small diameter sewers, that are connected via branch lines to the main transmission trunk sewers. Within the city, there are 24 sewage lift stations and one central pumping station at Narinda that are designed to raise the hydraulic level of the sewage, so that it can flow by gravity via the trunk sewers to the main treatment plant at Pagla. Manholes have been provided on the route of the mains but most are now inaccessible and in a very poor state of repair, with many being used as receptacles for household waste. Currently only 30% of the city area is served by the sewer system of which only 20% of the population have connections. The total length of the sewer network is 882 km (DWASA, 2010) of varying pipe materials, and sizes from 100 mm to 1350 mm.

Unplanned sewage discharges occur on a regular and prolonged basis throughout the DWASA sewerage system due to its inadequacy, and lack of maintenance. Such phenomena become critical in the rainy season due to infiltration.

Table 3.9 indicates the overall scenario of sewer networks in different zones of Dhaka. Subsequently, Figure 3.14 depicts the areas covered by sewer networks within DWASA service area.

Table 3.9 Sewerage system statistics of Dhaka (DWASA, 2010).

Zone	Area (km^2)	Population 2010 (million)	Existing sewer (km)		Total sewer (km)
			<450mm	>450mm	
Zone-1	27.53	1.58	143	29	172
Zone-2	9.16	0.94	146	6	152
Zone-3	24.18	1.09	127	1	128
Zone-4	26.03	1.33	95	0	95
Zone-5	15.76	0.65	167	0	167
Zone-6	28.21	1.63	168	0	168
Zone-7	29.49	0.77	0	0	
Zone-8	47.89	0.74	0	0	
Zone-9	62.94	0.69	0	0	
Zone-10	32.55	0.8	0	0	
N.Gonj W/S Div.	54.77	1.24	0	0	
Total	**358.51**	**11.46**	**846**	**36**	**882**

3.13 Pagla Sewerage Treatment Plant

DWASA operates a sewage treatment plant at Pagla located on an area of 110.5 ha, approximately 8 km away from the city centre in the south-east of Dhaka City, and approximately 1 km north of the Buriganga River. The Pagla treatment plant was originally constructed in 1978. It provides treatment of the wastewater collected by the central sewerage system, and is currently the only treatment facility in the city. However, construction of another treatment plant has been proposed at Dasherkandi for Hartijheel scheme.

The existing wastewater treatment plant at Pagla treats an average wastewater flow rate of 96,000 m^3/day and diurnal peak of 120,000 m^3/day. Due to damage of the trunk mains and sewerage system, the actual flow rate entering the Pagla treatment plant is approximately 30,000-40,000 m^3/day. Pagla treatment plant utilizes primary sedimentation, facultative ponds and disinfection, as treatment units. The mechanism of such units is described in the following chapters of this book. However, the existing treatment plant is subjected to some

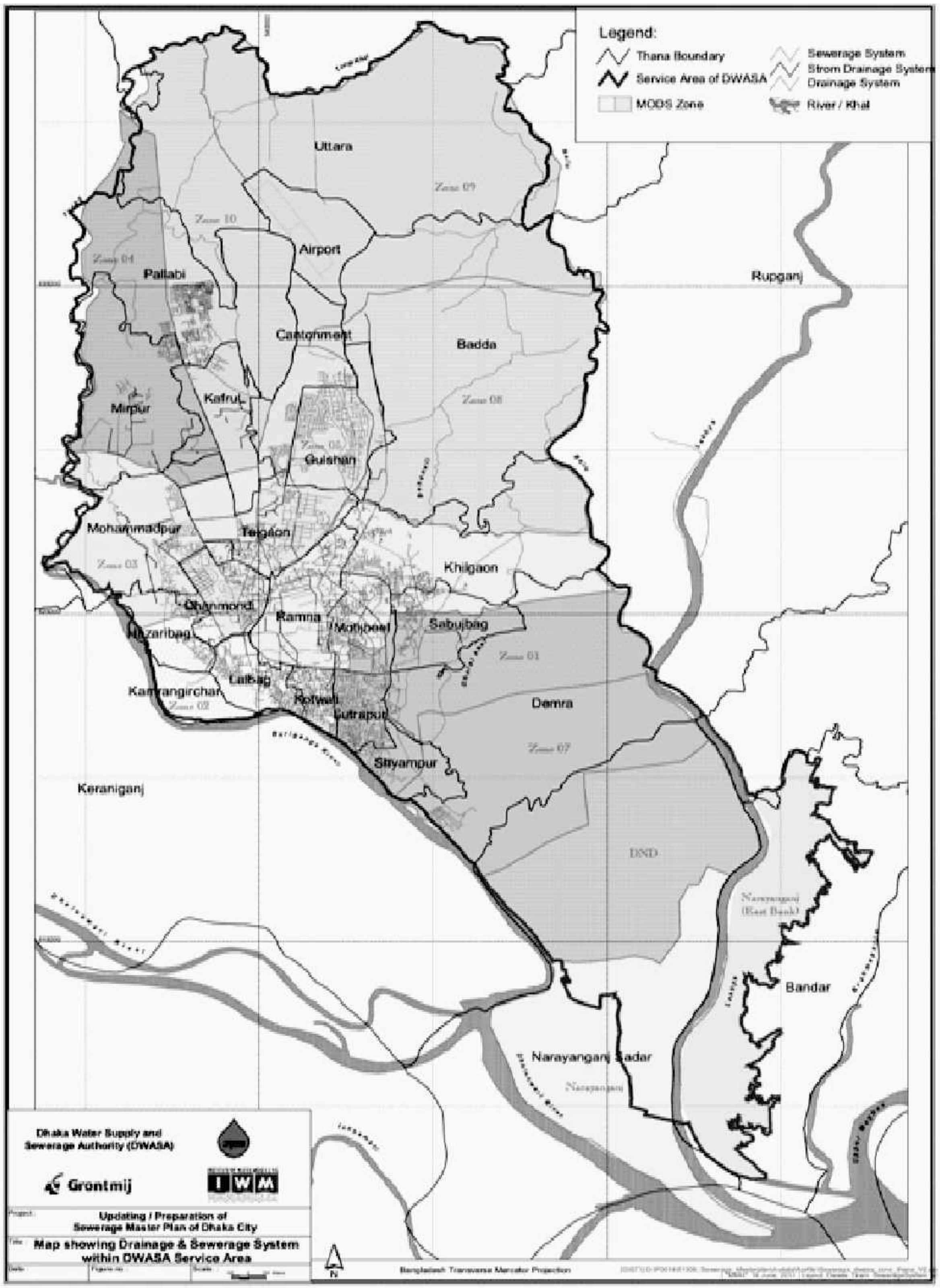

Figure 3.14 **Sewerage system coverage in DWASA service area (adapted from DWASA, 2013).**

operational and maintenance problems such as: (a) primary sedimentation tank scrapers do not efficiently cover the tank floor, hence sludge is not collected and removed; (b) the facultative ponds have accumulated sludge which should be emptied; and (c) the disinfection system is not operational. These operational problems are decreasing the quality of the effluent from the treatment plant.

As such, upgrading of the treatment plant is necessary to improve effluent qualities. Due to the land intensive nature of the treatment process, insufficient land is available for augmenting the capacity of the treatment plant on the existing site, illustrating the necessity of alternative process. Based on a comparison of various options, the trickling filter process has been proposed to be installed in phased modules.

Table 3.10 illustrates typical organics and suspended solids removal performances of Pagla treatment plant.

Table 3.10 **Organics and solid removal performances of Pagla treatment plant (DWASA, 2013).**

	Inlet (mg/L)		Primary sedimentation tank effluent (mg/L)		Facultative lagoon effluent (mg/L)		Overall removal (%)	
	BOD	**SS**	**BOD**	**SS**	**BOD**	**SS**	**BOD**	**SS**
Average	340	351	127	119	49	58	85	83

3.14 Upgrading Sewerage Systems of Dhaka

According to the master plan of DWASA, sewage transportation and treatment facilities will be upgraded in three phases, as summarized below.

The Phase 1 priority investment program (nominally 2010-2015) focuses on the Dhaka South (Pagla) and Dhaka East (Dasherkandi) catchments and includes: a) the first phase of Pagla treatment plant trickling filter system (and existing facultative ponds which has a combined capacity of 200,000 m^3/d); b) replacement of the eastern trunk sewer from Madhubag (near Tejgaon) to Pagla; c) and associated pump stations at Bashaboo and Golapbagh (near Swamibagh); d) re-directing wastewater via a new trunk sewer from Tejgaon and Gulshan to the new Dasherkandi treatment system; e) and associated pump stations at Tejgaon and Gulshan; f) new sewerage systems in Gulshan/Banani; g) re-connection of the existing lateral mains to the new eastern trunk sewer so that existing sewerage systems can be re-connected; h) construct new sewerage lift stations in the Khilgaon thana area, nominally Khilgaon and Rampura.

The Phase 2 investment program (nominally 2015-2025) focuses on the Pagla and Dasherkandi catchments and includes: a) the second phase of Pagla treatment plant trickling filter system (and existing facultative ponds which has a combined capacity of 250,000 m^3/d); b) rehabilitation of the trunk sewer to Tejgaon pump station and associated sewage collection system; c) construct new collection system and trunk mains servicing the Baridhara, Baridhara DOHS, West Badda and Niketan areas; d) a subsequent third phase of the Pagla treatment plant (decommissioning the facultative ponds hence providing a treatment capacity of 300,000 m^3/d) will then be commissioned; e) a program of rehabilitation and replacement of the remaining existing sewerage systems in Pagla catchment will be undertaken; f) extensions to new sewerage systems to be constructed within the Pagla catchment; g) refurbish Nerinda old pump station in order to increase capacity. The second component of the Phase 2 investments will include: a) construction of a parallel trunk sewer from Golapbagh (near Swamibagh) to Pagla treatment plant; b) construction of the sewerage systems, transmission mains and wastewater treatment plants for each of the remaining urban centre, such as: Rayerbazar, Mirpur, Uttara and Narayanganj, in the current DWASA service area.

The Phase 3 investment program (nominally 2025-2035) includes the 4th and 5th phases of Pagla trickling filter system (which has a cumulative capacity of 500,000 m^3/d), and will service the Pagla catchment. The Phase 3 investment program will also construct the sewerage systems, transmission mains and wastewater treatment plants for each of the remaining urban centre, such as: Savar, Tongi/Gazipur, Purchabal and Keraniganj, in the greater Dhaka region.

Questions

1. Write short notes on: (a) sanitary; (b) storm; and (c) combined sewers.
2. Why sewer networks are often subjected to H_2S corrosion?
3. Illustrate the main theory of inverted siphons in sewer network.
4. Determine the maximum hourly, average daily and minimum hourly residential sewage flows from an area occupied by 3000 people. The average per capita sewage flow is 50 gpcd. Consider the sewer length and house connections to be 4 miles, and infiltration to be 25,000 gpd.
5. A 24 inches sewer with n=0.013 is laid on a grade of 0.016. Compute the flow velocity when depth of flow is 6 inches.
6. A 25 inches sewer with n=0.011 is laid on a grade of 0.020.What will be the velocity when depth of flow is 3 inches?
7. State the comparison between small bore sewerage and conventional sewerage system.

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Chapter 4

Preliminary Treatment of Municipal Wastewater

Large floating solids, grit, and grease are the common components of raw municipal wastewater (collected through sewerage networks-Chapter 3), which can cause corrosion, blockage of the pumps, and equipments of wastewater treatment plants. In addition, these pollutants can also reduce the efficacy of the biological units of treatment plants. The proper functioning of the treatment plants is critically dependent on efficient removal of such pollutants, which is usually achieved through preliminary treatment process.

This chapter provides a brief description of the common preliminary treatment processes, employed for the treatment of raw wastewater. Figure 4.1 illustrates a flow diagram of the required operations, associated with preliminary treatment process; subsequently, sections 4.1-4.6 provide a brief description of each stage, along with necessary design criteria.

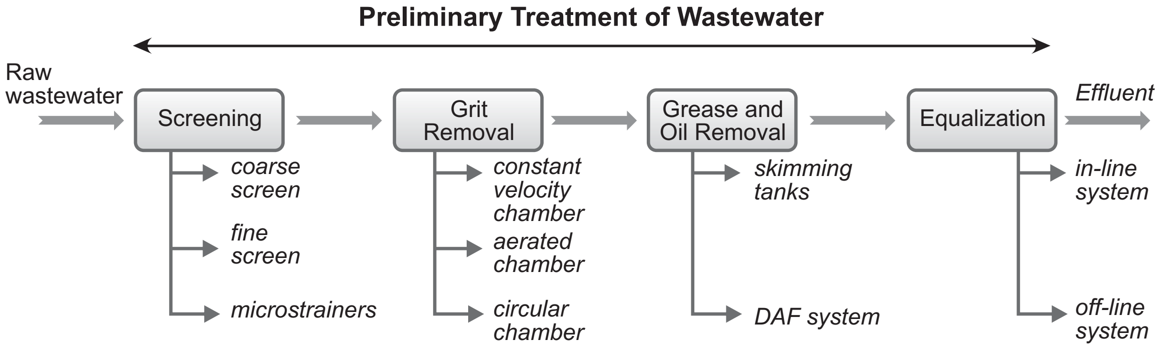

Figure 4.1 Flow diagram of preliminary treatment process.

4.1 Screening

Screening devices are usually employed as the first units, for the treatment of incoming wastewater. Screens can be defined as devices, with generally uniform openings for capturing the solids of influent wastewater. Depending on the operating mechanisms, screens can be classified into: (a) coarse screens; (b) fine screens; and (c) microstrainers.

Coarse screens. Coarse screens usually consist of equally spaced inclined vertical bars, predominantly made of steel. Coarse screens can further be classified into bar racks or bar screens, according to the bar spacing. Bar racks have clear spacing ranging between 5.08-10.16 cm (Figure 4.2), while bar screens have clear spacing of 0.64-5.08 cm.

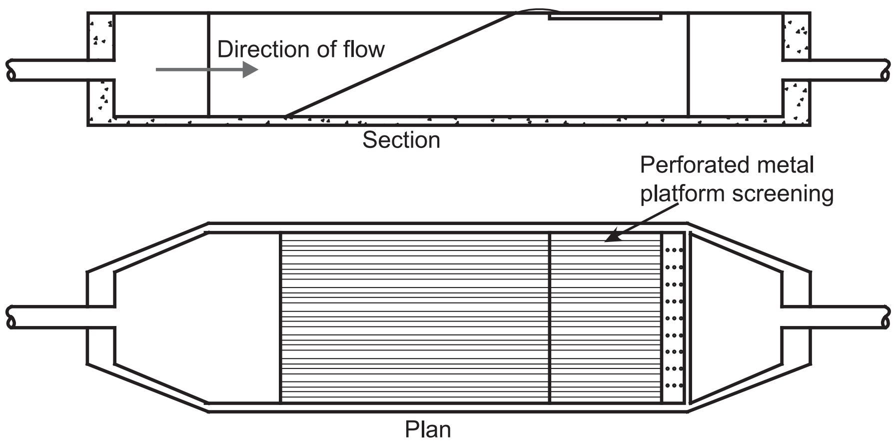

Figure 4.2 Bar racks employed for wastewater treatment.

Screens can be cleaned manually, or mechanically raked and cleaned. Manually cleaned screens are only used in small treatment plants, typically servicing a population equivalent (PE) of lower than 5000. Mechanically raked screens are used for plants servicing a PE greater than 20000.

Figure 4.3 shows a schematic of a manually cleaned screen that is manually raked on to a perforated plate. Cleaning of the screen must be carried out frequently to avoid clogging. Infrequent cleaning may result in significant upstream flow problems that are caused by the buildup of solids. A schematic of a mechanically raked bar screen is shown in Figure 4.4. Typically, the maximum clear space between the bars is 25 mm, although American practice permits spaces up to 38 mm. A space of 18 mm is considered typical, sufficient for the protection of downstream equipment.

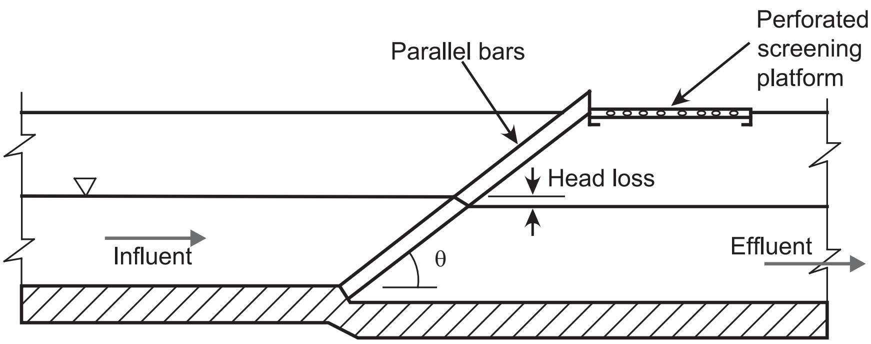

Figure 4.3 **Schematic of a manually cleaned screen.**

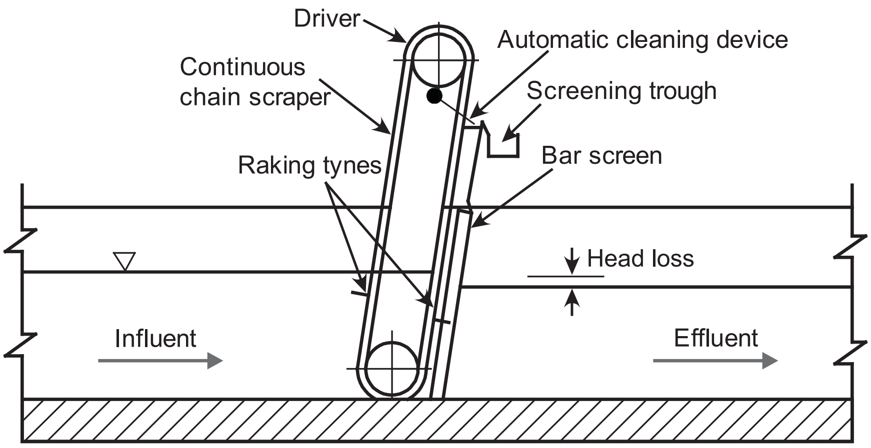

Figure 4.4 **Schematic of mechanically raked bar screen.**

Mechanically raked screens are normally set at an angle between 0 and 45° from the vertical. The use of such screens reduces labor cost and improves flow condition. They are available commercially, and manufacturers normally provide design charts to facilitate the selection of correct screen size for a particular service.

Figure 4.5 shows a special type of mechanically cleaned screen – a drum screen; materials being screened out naturally fall into a hopper as the screen rotates. A water spray assists in cleaning the screen.

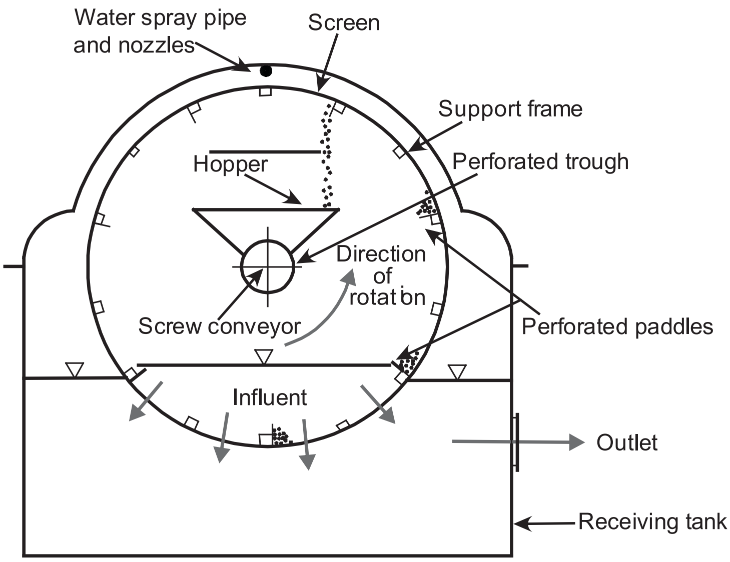

Figure 4.5 **Schematic diagram of a drum screen.**

Typical design values for manually and mechanically cleaned bar racks have been provided in Table 4.1.

Table 4.1 **Typical design criteria for manually and mechanically cleaned bar racks.**[a]

Parameter	Unit	Method of Cleaning	
		Manual	Mechanical
Bar size			
Width	mm	5-15	5-15
Depth	mm	25-38	25-38
Bar clearing space	mm	25-50	15-75
Approach velocity			
Maximum	m/s	0.3-0.6	0.6-1.0
Minimum	m/s		0.3-0.5
Allowable headloss	mm	150	150-600

[a]*after Metcalf and Eddy, (2003).*

The accumulation of solids across the bar racks can create hydraulic loss, which is a function of upstream and downstream velocity through the bars (Figure 4.6), along with a discharge coefficient. Bernoulli's equation can be used to compute such headloss through bar racks, as illustrated in Equation (4.1):

$$h_1 + \frac{v^2}{2g} = h_2 + \frac{V^2}{2g} + \Delta h \qquad 4.1$$

and

$$h = h_1 - h_2 = \frac{V^2 - v^2}{2gC^2} \qquad 4.2$$

where h_1 = upstream depth of water, m

h_2 = downstream depth of water, m

h = headloss, m
V = flow velocity through bar rack, m/s
v = approach velocity in upstream channel, m/s
g = gravity acceleration, 9.81 m/s^2
C = discharge coefficient, typically 0.84 (C^2 = 0.7)

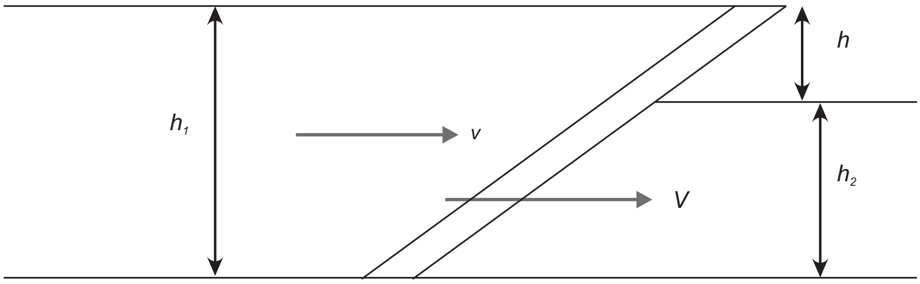

Figure 4.6 Water flow profile across a bar screen.

Equation (4.2) can be rearranged into Equation (4.3), after considering the value of C.

$$h = \frac{1}{0.7}\left(\frac{V^2 - v^2}{2g}\right) \qquad 4.3$$

Kirschmer (1926) proposed the following equation for computing headloss through racks:

$$h = B\left(\frac{w}{b}\right)^{4/3} \frac{v^2}{2g} \sin\theta \qquad 4.4$$

where h = headloss, m
w = maximum width of the bar with flow facing, m
b = minimum bar clear spacing, m
v = approach velocity towards the rack, m/s
g = gravity acceleration, 9.81 m/s^2
θ = horizontal angle of the rack
B = bar shape factor

The value of bar shape factor (B) can be obtained from Table 4.2.

Table 4.2 Typical bar shape factors.[a]

Bar Type	B
Sharp edged rectangular	2.42
Rectangular with semicircular face	1.83
Circular	1.79

[a] *after* Lin (2007).

Example 4.1 Headloss across coarse screens. Calculate the velocity through a rack, when approach velocity is 0.60 m/s, flow open area through clean bar rack is 0.15 m^2 and headloss across the rack is 30 mm. Also estimate the headloss, when 50% area of the flow area is blocked off due to coarse solids accumulation.

Solution

1. Calculation of velocity through bar rack

The headloss across the rack can be computed employing Equation (4.3):

$$h = \frac{1}{0.7}\left(\frac{V^2 - v^2}{2g}\right)$$

$$0.030\ \text{m} = \frac{V^2 - (0.6\ \text{m/s})^2}{0.7(2 \times 9.8\ \text{m/s}^2)}$$

$$V = 0.87\ \text{m/s}$$

2. Calculation of headloss through the clogged bar rack

Reduction of screen area by 50% is associated with doubling of the velocity; as such, the velocity through the clogged bar screen (V) is:

$$V = 0.87 \times 2 = 1.74\ \text{m/s}$$

Assuming the flow coefficient for the clogged bar screen to be 0.6, the estimated headloss is:

$$h = \frac{1}{0.6}\left(\frac{(1.74\ \text{m/s})^2 - (0.6\ \text{m/s})^2}{2(9.81\ \text{m/s}^2)}\right) = 0.22\ \text{m}$$

Fine screens. Fine screens, which usually consist of wire, perforated plate, or closely spaced bars (with openings 1.5-6.4 mm), are usually employed after coarse screens to retain finer materials from the incoming wastewater (such as: agro and food processing industries). Stainless-steel mesh or special wedge-shaped bars are commonly used as screening medium.

The clean water headloss through fine screens can be obtained by common orifice equation, as illustrated in Equation (4.5):

$$h = \frac{1}{2g}\left(\frac{v}{C}\right)^2 = \frac{1}{2}\left(\frac{Q}{CA}\right)^2 \qquad 4.5$$

where h = headloss, m
v = approach velocity, m/s
C = discharge coefficient of screen (i.e. 0.60 for a clean screen)
g = gravitational acceleration, m/s^2
Q = discharge through screen, m^3/s
A = effective area of submerged screen, m^2

Although the headloss of clean water through a clean screen is less, it depends on the cleaning method and frequency, size and quantity of suspended solids in the wastewater, and screen opening size.

Microstrainers. Microstrainers are sometimes employed after fine screen, to achieve: (a) removal of algae; and (b) further reduction of suspended solids from incoming wastewater. Microstrainers are made of very fine fabric, or screen wound around a drum. The drum which is usually 75% submerged continues to rotate, with water flowing from inside to the outer portion of the drum. The solids retained by the strainers are removed by water jets directed towards drum surface, and collected in a channel beneath the drum. Figure 4.7 illustrates the components of a microstrainer, to facilitate wastewater treatment.

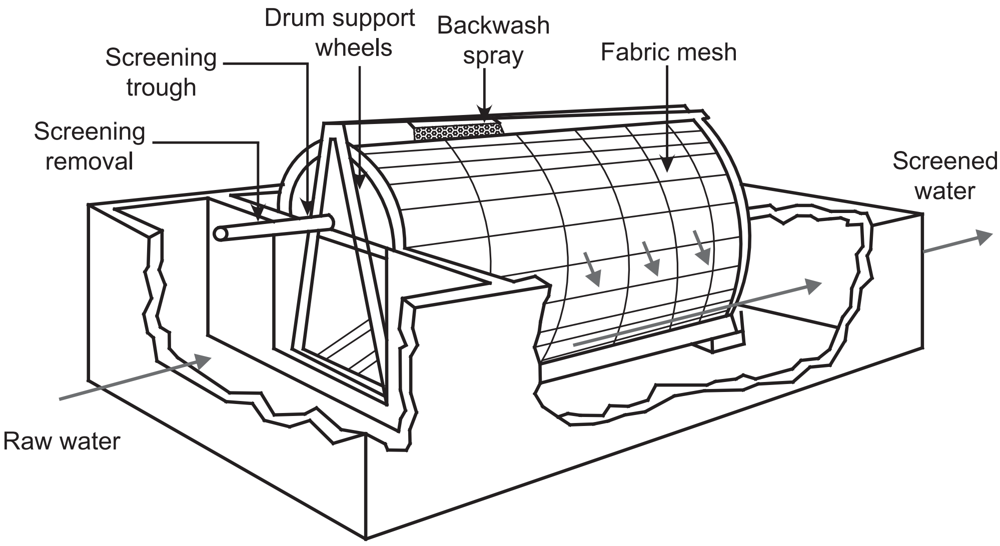

Figure 4.7 Schematic diagram of a microstrainer for wastewater treatment.

Continuous operation of microstrainers promote slime growth (on the surface), which can be mitigated by washing the fabric with Cl_2 solution. Frequent slime build up can be corrected by installing ultra violet radiation. Table 4.3 indicates typical design values of microstrainers.

Table 4.3 Microstrainer design parameters.[a]

Item	Typical Value
Screen mesh	20-25 μm
Submergence	75% of height
Hydraulic loading	12-24 $m^3/m^2/h$ of submerged drum surface area
Peripheral drum speed	4.5 m/min at 7.5 cm headloss
Typical drum diameter	3m

[a]*after* USEPA (1975).

4.2 Comminuting

Comminutors, an alternative to racks or screens (Figure 4.8), cut coarse solids (from raw wastewater) to 6-10 mm, so that they do not interfere with other systems. Chopped solids are removed from the flow during sedimentation process.

The basic parts of a comminutor include a slotted drum rotating in the vertical plane, and a cutting tooth. Stationary teeth shred the material intercepted by the screen; the shredded materials pass through the drum slots. Barminutor is the most widely implemented comminuting device, which uses a vertical screen with cutting head travelling up and down the rack of bars, thereby cutting the intercepted materials.

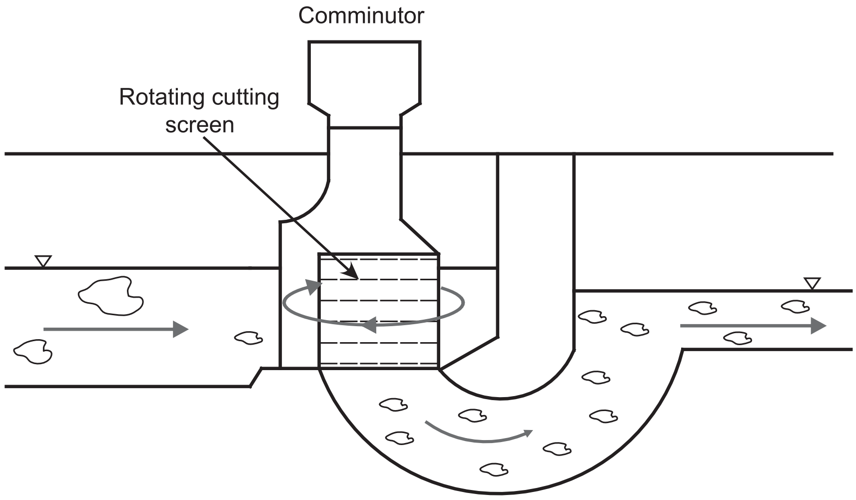

Figure 4.8 Comminutors for wastewater treatment.

Shredding devices should be located in parallel with bar screens, ahead of pumping facilities at the treatment plant. Grit removal prior to shredder saves wear on the cutting head of the comminutor.

4.3 Grit Removal

Grit materials are common components of domestic and municipal wastewater, stormwater runoff, and industrial sewage. These materials include inorganic solids such as pebbles, sand, silt, cinders, cigarette filters, metal fragments etc; in addition, eggshells, bone chips, coffee grounds, and seeds are an inherent proportion of grit materials. The composition of grit varies with moisture and volatile contents. The moisture content of grit ranges from 13-63%; the volatile content ranges from 1-5%. The specific gravity of clean grit can be as high as 2.7 with inert material, and as low as 1.3 when the inert material contains higher organic matter.

Grit materials can promote substantial wearing of mechanical equipments, pumps and clogging of piping networks. As such, grit chambers are extremely important units of wastewater treatment plants, which eventually protect the mechanical equipments from abrasion, reduce the deposition in pipelines and digester cleaning frequency.

The settling phenomenon in a grit chamber is based on free settling of particles (i.e. discrete settling of particles), which can be described by Stokes formula (Equation 4.6) applicable for spherical shaped particles:

$$V_s = \frac{g\,(\rho_s - \rho_L)\,d^2}{18\mu} \qquad 4.6$$

where V_s = settling velocity, m/s
g = gravity acceleration, m/s^2
ρ_s = solid density, kg/m^3
ρ_L = liquid density, kg/m^3
μ = liquid viscosity, kg/s/m
d = particle diameter, m

Grit chambers employed for the treatment of wastewater can be classified into three categories, such as: (a) constant velocity grit chamber; (b) aerated grit chamber; and (c) circular grit chamber.

The constant velocity grit chamber is usually equipped with two components (Figure 4.9); the two components must be matched to have constant forward velocity (i.e. 0.3 m/s). The retention time usually varies between 1-2 min for grit settling, with low yield capacity. This system is applicable for small treatment plants.

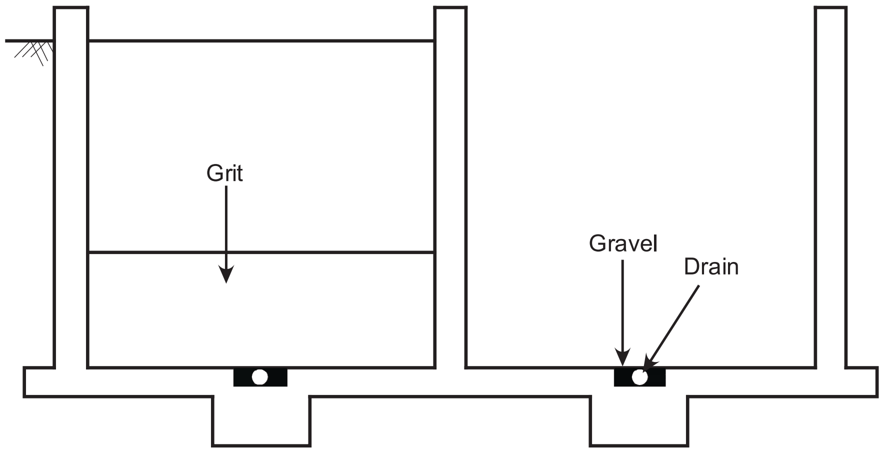

Figure 4.9 **Constant velocity grit chamber.**

In larger treatment plant, the trend is toward aerated grit chambers (Figure 4.10). In such systems, turbulence is created by the injection of compressed air to provide a constant rate cross-current allowing settlement of heavier grit, whereas lighter organic particles remain suspended and pass through the tank. Proper control of the turbulence is essential for efficient functioning of aerated grit chambers. Extremely high turbulence washes away grit from the chambers; on the other hand, lower turbulence promotes the removal of organic matter, along with grit. Typically 0.0005-0.00236 m^3/s of air per foot of chamber length is required with 3-5 min retention time, to maintain the efficiency of aerated grit chambers. Typical design values for aerated grit chambers are illustrated in Table 4.4.

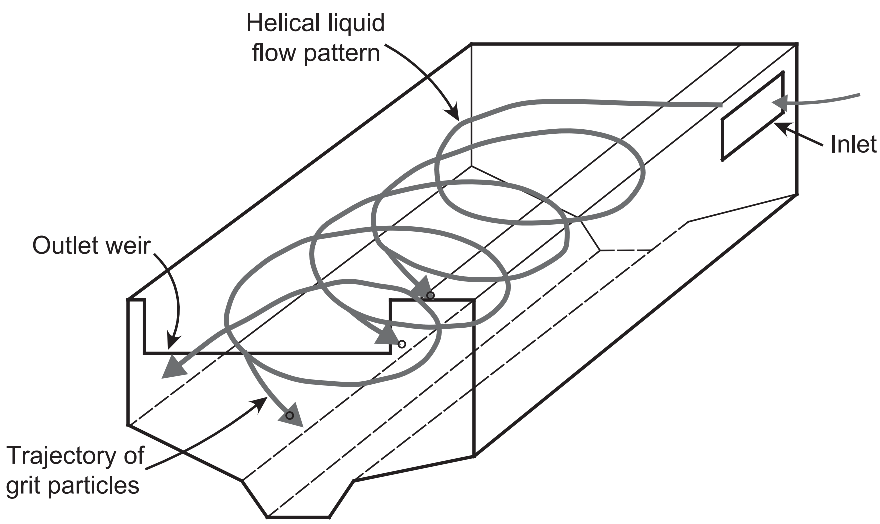

Figure 4.10 **Aerated grit chamber.**

CHAPTER 4

Table 4.4 Aerated grit chamber design parameters.[a]

Item	Unit	Typical Design Values
Dimensions		
Depth	m	2-5
Length	m	7.5-20
Width	m	2.5-7
Width-depth ratio		1:1.5-1:2
Detention time at peak flow	min	2-5
Grit quantities	$m^3/10\ m^3$	0.004-0.2

[a] *after* Peavy et al., 1985

An additional advantage of air supply into aerated grit chambers can be attributed to the freshening of wastewater, leading to notable odor reduction. If desired, the chamber can also be used for chemical addition, mixing and/or flocculation ahead of primary treatment. If correctly designed, an aerated grit chamber with a minimum hydraulic detention time of 3 minutes (operating at peak flow) can capture approximately 95% of the grits, that have sizes larger than 0.2 mm.

In circular grit chambers, velocity is controlled through the paddles on a vertical shaft. The retention time in such chambers varies between 2-3 min.

4.4 Grease and Oil Removal

Effluent from kitchens, restaurants, oil refineries, and slaughter houses contains substantial amount of grease and oil; such components, if not removed (from wastewater) can cause deposits in air diffusers of biological units, and adversely effect bacteria and protozoa life in biological reactors. As such, grease and oil removal are extremely important for proper functioning of the treatment plants, and are usually achieved through skimming tanks or floatation methods.

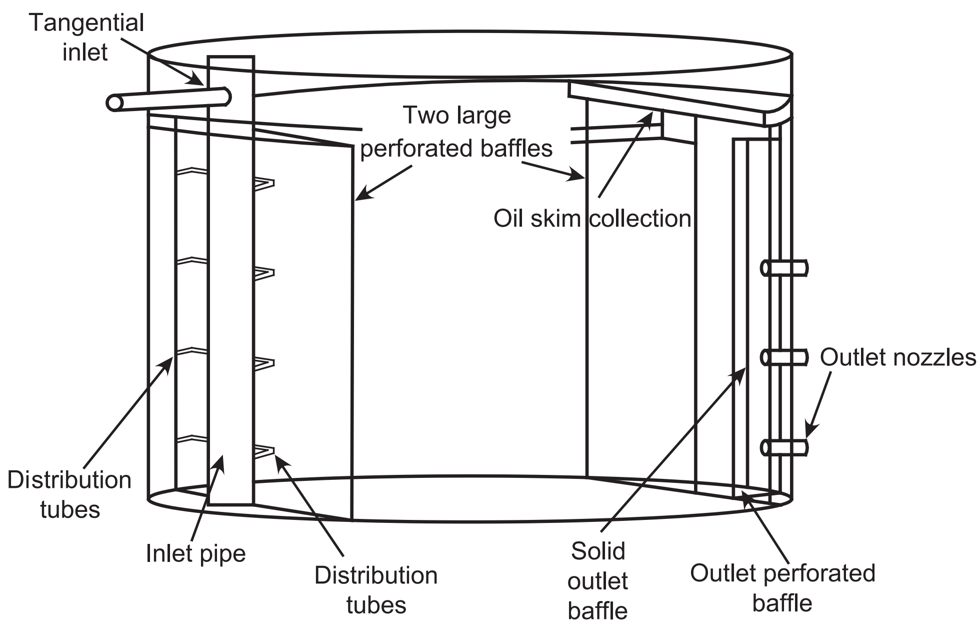

Figure 4.11 Components of a skimming tank.

Skimming tanks. These units have baffled subsurface entrance and exit structures (Figure 4.11), which permit the floating material to be retained. Skimming tanks are usually designed with retention time 15 min or less. Horizontal velocity of water is kept between 50-250 mm/s, to prevent deposition of organic particles on the bottom.

Floatation. Floatation is required to separate solid or liquid materials from the liquid phase, which is achieved by introducing fine gas bubbles into the bulk liquid. The bubbles are attached into particulate materials, and the buoyant force of combined particles and gas bubbles is adequate to cause particles to rise to the water surface. Particles, thus having higher density than liquid can be forced to rise.

Dissolved air floatation systems (DAF) are often implemented as floatation chambers. The basic components of a DAF system comprises of pressurizing pumps, air injection facilities, retention tank, pressure reducing valve and floatation tank. The operation mode of a DAF system can be classified into: (a) full flow mode; and (b) recycle flow mode.

When a DAF system is operated under full flow mode (Figure 4.12), air is dissolved in wastewater under a pressure of several atmospheres, followed by the release of pressure to the atmospheric level. For small pressure systems, the total flow can be pressurized up to 275-350 kPa by a pump, with compressed air added at the pump suction. The entire flow is held in a retention tank under pressure for several minutes, to allow air dissolving. The flow is then admitted to the floatation tank through a pressure reducing valve, where air comes out of solution in very fine bubbles attaching the particulate matter, and is removed by the skimmer from the surface water. The main advantages of a full mode flow DAF system can be enlisted as: (a) the entire flow is exposed to air pressure; (b) formation of bubbles; and (c) moderate pressure requirement.

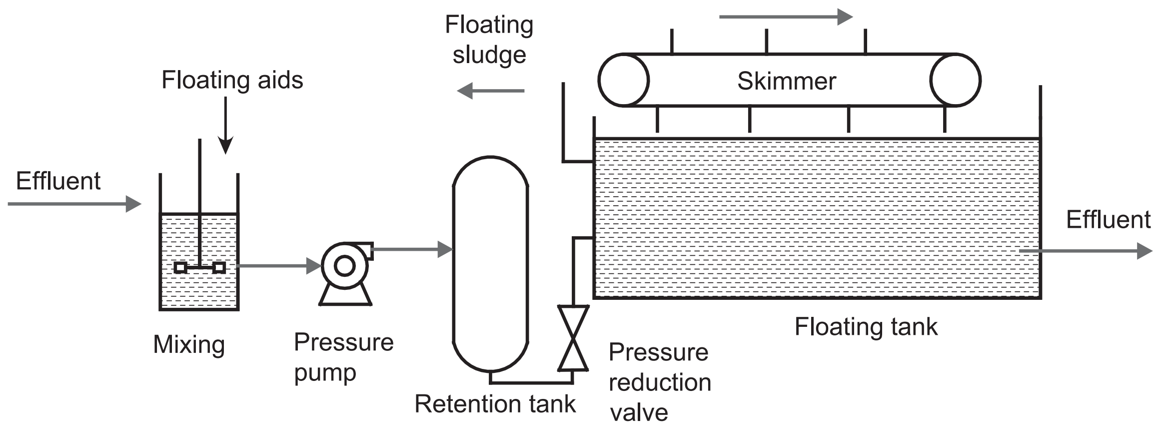

Figure 4.12 DAF system with full-flow mode.

In a recycle flow mode DAF system (Figure 4.13), a portion of the DAF effluent (15-120%) is recycled, pressurized and semi saturated with air. The recycled flow is mixed with unpressurized main stream, prior to the entrance point of the floatation tank. Air comes out of the solution associated with particulate matter, at the entrance zone. Since relatively cleaner DAF effluent is utilized in recycling mode system, the blockage of diffusers is generally reduced. However, the size of the floatation unit must be higher, for accommodating the combination of incoming wastewater flow and recycled water.

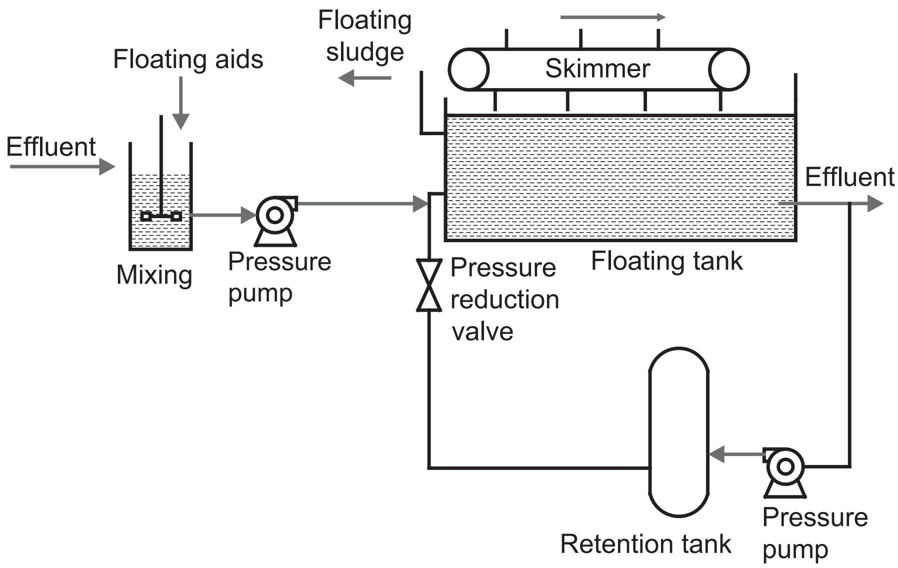

Figure 4.13 **DAF system with recycle-flow mode.**

The performance of a DAF system depends on the ratio of air volume (A) to the mass of solids (S) i.e. A/S. The relationship between A/S ratio, air solubility, pressure, and solids concentration for a system in which the flow is pressurized is illustrated in Equation (4.7).

$$\frac{A}{S} = \frac{1.3 s_a (fP - 1)}{S_a} \qquad 4.7$$

where A/S = air to solids ratio, mL air/mg solids
s_a = air solubility, mL/L
f = air fraction dissolved at P pressure
P = pressure, atm
S_a = influent suspended solids, mg/L

Equation (4.8) is employed for DAF system with only pressurized recycle.

$$\frac{A}{S} = \frac{1.3 s_a (fP - 1) R}{S_a Q} \qquad 4.8$$

where R = pressurized recycle, m^3/d
Q = mixed liquor flow, m^3/d

4.5 Equalization Basin

The purpose of equalization basins is to achieve mixing of wastewater of different volumes, in order to obtain a uniform flow rate. Equalization basins eliminate or reduce shock loadings, dilute inhibiting substances, and stabilize pH, thereby enhancing smooth operation of the following biological reactors. Such basins also enhance effluent water quality and thickening performances of secondary sedimentation tanks, through consistent loading of solids.

The arrangement of equalization basins in a treatment plant can be classified as in-line (Figure 4.14a) and off-line (Figure 4.14b) arrangement. In the in-line system, the entire flow is passed through an equalization tank, allowing substantial flow damping and constituent concentration. In the off-line system, a portion of the incoming wastewater is diverted into the equalization basin; despite such operation reduces pumping cost, the flow rate damping is significantly reduced.

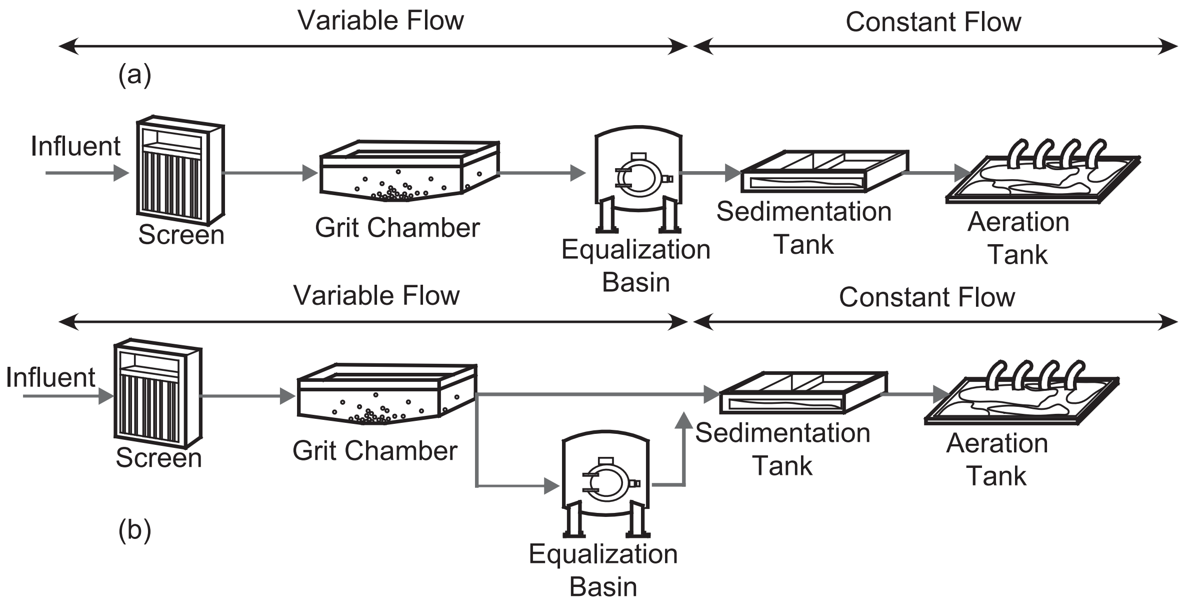

Figure 4.14 **Arrangement of equalization tank: (a) in-line system; and (b) off-line system.**

4.6 Design Examples

This section describes detailed design procedures of the important units of preliminary treatment process (i.e. coarse screen, grit chambers, and equalization tank). Firstly, the design of coarse screens and grit chambers has been illustrated in example 4.2; secondly, example 4.3 denotes the procedure of computing the volume of an equalization tank.

Example 4.2 **Design of coarse screens and aerated grit chambers.** Design a coarse screen and aerated grit chambers for wastewater treatment, employing the following information:

- Peak flow through the rack, $Q_p = 0.9\ m^3/s$
- Velocity through rack during peak flow, $v = 0.8\ m/s$
- Angle of the rack to the horizontal, $\theta = 60°$
- Upstream depth of wastewater, $d = 1.5\ m$
- Retention time in aerated grit chamber during peak flow = 3 min
- Air supply = $0.00236\ m^3/s$ per ft
- Volume of grit to be produced during peak flow= $80\ mL/m^3$

Solution

a. Design of coarse screen.

Step 1. Determine total clean area (A) through the rack

$$A = \frac{Q_P}{v}$$

$$= \frac{0.9\ \text{m}^3/\text{s}}{0.8\ \text{m/s}} = 1.12\ \text{m}^2$$

Step 2. Determine total width (w) of the opening at the rack

$$w = \frac{A}{d}$$

$$= \frac{1.12}{1.5} = 0.75\ \text{m}$$

Step 3. Calculate number of opening

Choose a 25 mm clear opening

$$\text{Number of opening, } n = \frac{w}{\text{opening}} = \frac{0.75\ \text{m}}{0.025\ \text{m}} = 30$$

Use 29 bars with 10 mm (0.01 m) width, and 50 mm thickness.

Step 4. Calculate width (W) of the chamber

$$W = 0.75 + 0.01 \times 29 = 1.04\ \text{m}$$

Step 5. Calculate height of the rack

$$\text{height} = \frac{1.5\ \text{m}}{\sin 60°} = 1.73\ \text{m}$$

Allowing at least 0.6 m of freeboard, the total height 2.5 m is selected.

b. Design of aerated grit chambers.

Step 1. Calculate the volume of the grit chambers

Provide two chambers; as such, the volume of each unit can be computed as:

$$\text{Volume} = (0.9\ \text{m}^3/\text{s} \times 3\ \text{min}) / 2 = 81\ \text{m}^3$$

Step 2. Calculate the size of a rectangular grit chamber

Assuming the width of the tank to be 2m, and employing a depth-to-width ratio of 1.5:1 (Table 4.4):

$$\text{Depth} = 2\ \text{m} \times 1.5 = 3\ \text{m}$$

$$\text{Length} = \frac{\text{volume}}{(\text{depth} \times \text{width})} = \frac{81\ \text{m}^3}{3\ \text{m} \times 2\ \text{m}} = 13.5\ \text{m}$$

The dimension of each chamber is 13.5m×2m×3m.

Step 3. Calculate the required air supply

Given 0.00236m^3/s per ft (0.3m) length.

$$\text{Air required} = \frac{0.00236\ \text{m}^3/\text{s}}{0.3\ \text{m}} \times 13.5\ \text{m} = 0.10\ \text{m}^3/\text{s}$$

Step 4. Estimation of the production of grit volume during peak flow

Given the volume of grit to be produced (during peak flow) is 80 mL/m^3.

$$\text{Grit volume} = 80\ \text{mL/m}^3 \times 0.9\ \text{m}^3/\text{s} = 6220800\ \text{mL/d} = 6.22\ \text{m}^3/\text{d}$$

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Example 4.3. Volume of equalization tanks. Calculate the volume of the equalization tank of a treatment plant, subjected to variable inflow rate (with time) as illustrated below.

Time Interval	Flow (m^3/h)
6.00 A.M-10.00 A.M	330
10.00 A.M -14.00P.M	150
14.00 P.M -18.00 P.M	400
18.00 P.M -22.00 P.M	80
22.00 P.M -2.00A.M	90
2.00A.M-6.00 A.M	150

Solution

Assuming the wastewater inflow is $Q(t)$, constant pumping rate is Q_p, and the process cycle time is T, the volume of wastewater collected during the process cycle can be illustrated in Equation (4.9).

$$\int Q(t)\,dt = \Sigma Q(t)\,\Delta t \qquad 4.9$$

If the volume of pumping rate is $\Sigma Q_p t$, then the volume of the equalization tank can be computed as the maximum difference between influent and pumped wastewater; such relation can be illustrated as $= \Sigma Q(t)\,\Delta t - \Sigma Q_p t$. Employing the above relations the volume of the equalization tank can be calculated, as illustrated below.

Time, Δt	Flow	$\Sigma Q(t)\Delta t$	$\Sigma Q_p t$	$\Sigma Q(t)\Delta t - \Sigma Q_p t$
4	330	1320	800	520
4	150	1920	1600	320
4	400	3520	2400	1120
4	80	3840	3200	640
4	90	4200	4000	200
4	150	4800	4800	0

$$Q_p = \frac{4800}{24} = 200\ \text{m}^3/\text{h}$$

$$V = 1120\ \text{m}^3$$

Questions

1. Briefly describe the importance of preliminary treatment process, to facilitate the performance of wastewater treatment plants.
2. Write a short note on aerated grit chamber.
3. Enlist the basic differences between full flow mode and recycle flow mode DAF systems in wastewater treatment plants.
4. Calculate the velocity through a rack, when approach velocity is 0.40 m/s, flow open area through clean bar rack is 0.15 m^2 and headloss across the rack is 35 mm. Also estimate the headloss, when 40% area of the flow area is blocked off due to coarse solids accumulation.
5. Design aerated grit chambers (assuming two compartments) for the treatment of wastewater, when influent peak flow is 0.7 m^3/s, and the design retention time in aerated grit chamber during peak flow is 3.5 min.

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Chapter 5

Primary Treatment of Municipal Wastewater

Primary treatment of wastewater can be achieved by employing septic tanks (Chpater 3), or sedimentation basins. The former technologies are preferred in those areas, that do not have sewer networks for transporting wastewater into treatment plants. The latter systems are usually included as integrated units of treatment plants, and are heavily dependent on sedimentation mechanism.

Sedimentation depends on physical separation of the suspended materials (from wastewater) by gravity settling. Sedimentation process removes solids (heavier than water), particulate matter, and grit from preliminary treated effluent (Figure 5.1) through sedimentation basin; such basin is also denoted as primary sedimentation tank, clarifier, settling basin or settling tank. It is the most widely employed treatment unit, to provide treatment of raw sewage. Properly designed sedimentation tanks often remove 50-70% suspended solids, and 25-40% BOD from wastewater.

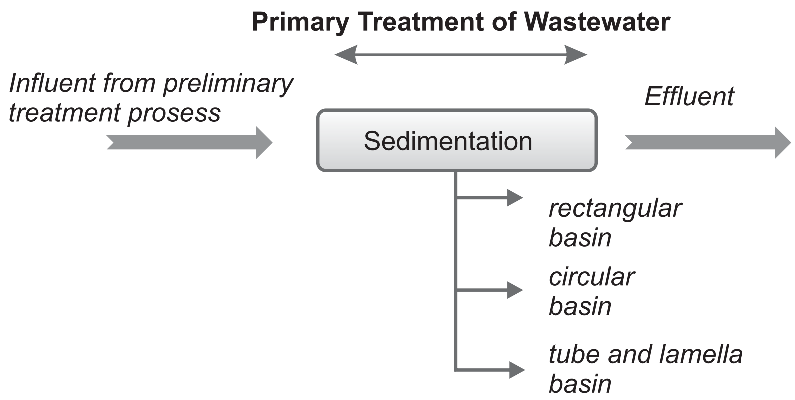

Figure 5.1 **A schematic diagram of wastewater flow profile across a sedimentation tank.**

Depending on solids concentration, four types of settling process contribute to solids removal in sedimentation tanks such as: (a) discrete; (b) flocculant; (c) hindered; and (d) compression settlings. A brief description of these settling phenomena has been provided in Table 5.1.

Table 5.1 **Typical settling types of particles in sedimentation tanks.**

Settling Phenomena	Description
Discrete settling (Type-1)	Discrete settling is often observed in wastewater with low solids concentration. Discrete settling involves individual settling of particles which do not change size, shape and mass during settling, and do not influence each other by being too close.
Flocculant settling (Type-2)	Flocculant settling is observed when wastewater comprises of coalesced, or flocculate particles during settling process. Such settling results particles coalescing, and formation of bigger particles which settle at a faster rate.
Hindered settling (Type-3)	Observed in wastewater consisting closely packed particles. The settling velocity of these particles is obstructed, causing upward water displacement (by particles), which in turn reduces the settling velocity.
Compression settling (Type-4)	Observed in wastewater where particle concentration is such that a structure is formed, and further settling can occur only by the compression of the formed structure.

This chapter focuses on the detailed analyses of the settling mechanisms usually observed in sedimentation tanks. In addition, this chapter also provides a brief description of the important design factors applicable for the sedimentation tanks, along with a design example.

5.1 Discrete Particle Settling

Analyses of discrete settling (Type-1). This type of settling is easier to analyze. If a particle is suspended in water, two forces initially act upon it such as: (a) gravity force, f_g (Equation 5.1); and (b) buoyant force, f_b (Equation 5.2).

$$f_g = \rho_p g V_p \qquad 5.1$$

$$\text{and } f_b = \rho_w g V_p \qquad 5.2$$

where ρ_p = particle density
ρ_w = water density
g = gravity constant
V_p = particle volume

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Since these forces act in opposite directions, there will be no net force on the suspended particle when $\rho_\rho=\rho_\omega$. However, if the density of particle and water differs a net force will be exerted, resulting in acceleration (of the particle) in the direction of force, as expressed in Equation (5.3):

$$f_{net} = (\rho_p - \rho_w)\, g V_p \qquad 5.3$$

When the particle is in motion a third force is created due to viscous friction defined as drag force (f_d), expressed through Equation (5.4):

$$f_d = C_D A_p \rho_w \frac{v^2}{2} \qquad 5.4$$

where C_D = drag co-efficient
A_p = particle cross section area, perpendicular to the movement direction
v = particle velocity

Drag force (Equation 5.4) acts in the opposite direction to the driving force (Equation 5.3), and increases to the square of velocity; acceleration occurs at a decreasing rate until a steady velocity is reached at a point where the drag force equals the driving force:

$$(\rho_p - \rho_w)\, g V_p = C_D A_p \rho_w \frac{v^2}{2} \qquad 5.5$$

For spherical particles, the ratio of volume (V_p) and particle cross-sectional area (A_p) can be expressed through Equation (5.6):

$$\frac{V_p}{A_p} = \frac{{}^4\!/_3\, \pi ({}^d\!/_2)^3}{\pi ({}^d\!/_2)^2} = \frac{2}{3} d \qquad 5.6$$

Substituting the ratio of volume and particle cross-sectional area in Equation (5.5) yields the following equation:

$$v_t^2 = \frac{4}{3} g \frac{(\rho_\rho - \rho_\omega) d}{C_D \rho_\omega} \qquad 5.7$$

The value of C_D changes with different flow regimes. As such, the expression of C_D values for laminar, transitional, and turbulent flow regimes are defined in Equations (5.8)-(5.10).

$$C_D = \frac{24}{R_e} \text{ (laminar flow)} \qquad 5.8$$

$$= \frac{24}{R_e} + \frac{3}{R_e^{1/2}} + 0.34 \text{ (transitional flow)} \qquad 5.9$$

$$= 0.4 \text{ (turbulent flow)} \qquad 5.10$$

where R_e is the Reynolds number, and can be expressed by Equation (5.11).

$$R_e = \frac{\phi v_t \rho_\omega d}{\mu} \qquad 5.11$$

where d = particle diameter
μ = kinematic viscosity of water

Reynolds number (R_e) values less than 1.0 indicate laminar flow, while values greater than 10^4 indicate turbulent flow; intermediate values denote transitional flow. The shape factor is added to correct lack of particle spherosity. For perfect spheres, the value of is assumed to be 1.0.

In case of laminar flow, substituting Equation (5.8) into Equation (5.7) yields Equation (5.12).

$$v_t = \frac{g (\rho_\rho - \rho_\omega) d^2}{18\mu} \qquad 5.12$$

Equation (5.12) is defined as Stokes equation, often employed for determining the settling velocity of discrete particles.

Sedimentation tank configuration. Rectangular, square or circular shaped sedimentation tanks are usually designed for wastewater treatment. An ideal continuous horizontal flow sedimentation tank (Figure 5.2a and b) comprises of four zones such as: (a) inlet zone; (b) settling zone; (c) sludge zone; and (d) outlet zone. The inlet zone provides uniform distribution of influent wastewater over the cross-sectional area of the tank in such a way that, the flow follows horizontal path to prevent short-circuiting. In the settling zone, uniform concentration of particles settle at terminal settling velocity to the sludge zone (beneath the settling zone). In the outlet zone, the clarified effluent is collected through an outlet weir, and is discharged to the following treatment units.

For the design of sedimentation tanks, a particle settling velocity V_s (Figure 5.2b) is considered as design overflow settling velocity, and can be defined as the settling velocity of a particle that settles through the total effective depth H in theoretical detention time. The particles, having a terminal velocity (V_a) equal or greater than V_s are also removed in the sedimentation tanks (Figure 5.2b). Equation (5.13) expresses the design overflow settling velocity (V_s) in terms of flow, area, width and length of the sedimentation tank.

$$V_s = \frac{Q}{A} = \frac{Q}{WL} \quad 5.13$$

where Q = flow, m^3/d

A = surface area of the settling zone, m^2

V_s = overflow rate, $m^3/m^2.d$

W, L = width and length of the tank, m

Equation (5.13) can also be defined as the SOR, that represents the settling velocity of the slowest settling particle that is 100% removed. As such, particles settling at velocities, equal or greater than the SOR will be entirely removed.

Flow through a sedimentation tank is independent of the clarifier depth; such depth can be expressed as a product of the particle settling velocity and detention time (t), as expressed through Equation (5.14):

$$H = V_s t \quad 5.14$$

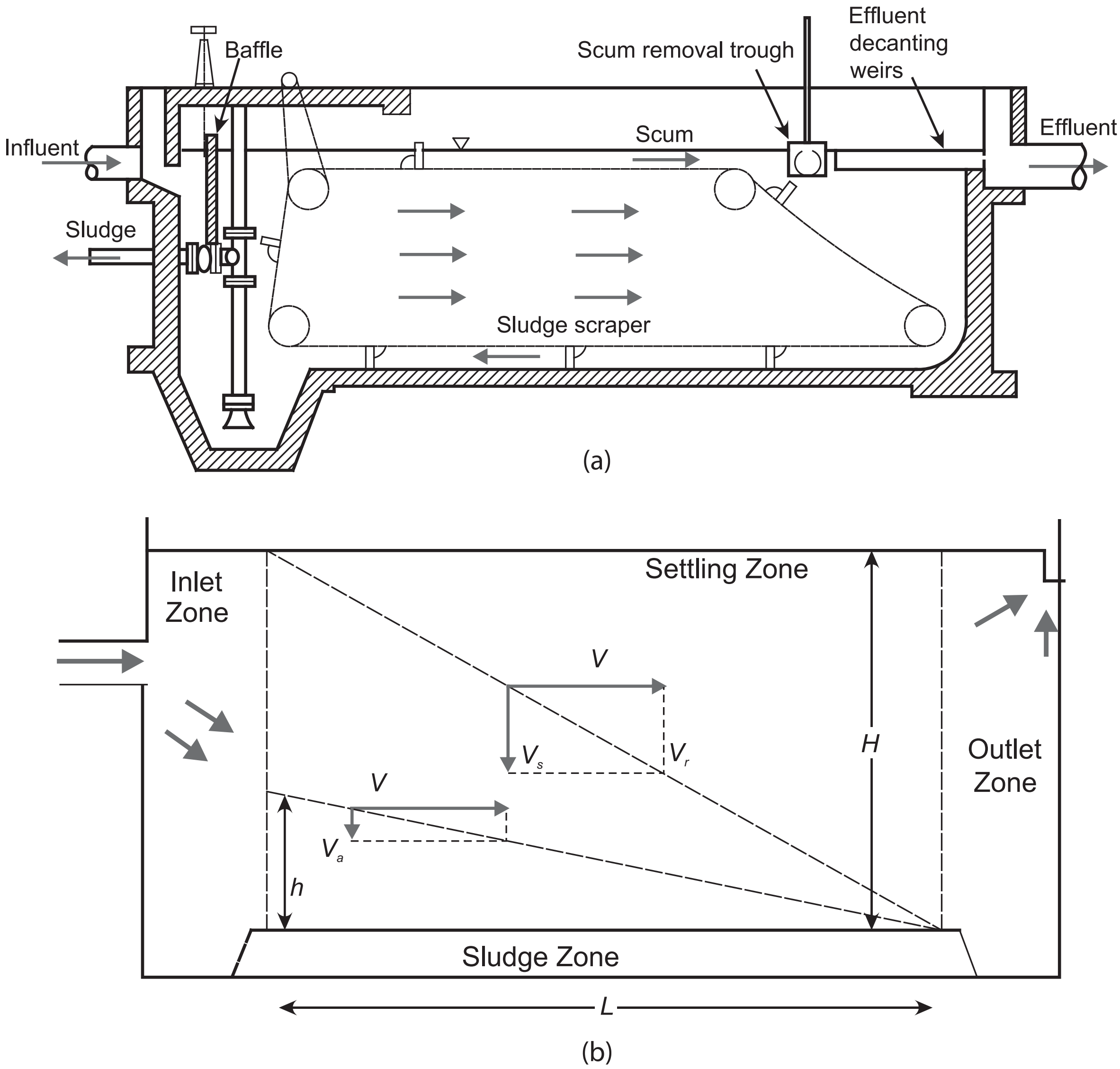

Figure 5.2 **(a) Schematic diagram of a rectangular sedimentation tank; and (b) discrete settling in rectangular sedimentation basin.**

Considering the relationship as illustrated in Equation (5.14), the removal ratio (r) of particles with settling velocity equal to V_a, can be expressed by Equation (5.15):

$$r = \frac{h}{H} = \frac{V_a t}{V_s t} = \frac{V_a}{V_s} \qquad 5.15$$

Equation (5.15) indicates that particles with settling velocity V_a (less than V_s) will also be removed, provided that they enter the settling zone at a depth (h), less than H (Figure 5.2b).

For actual particle suspension with size variety and density, prediction of a sedimentation basin performance requires either particle size distribution or column analysis. From either technique, a cumulative settling velocity frequency distribution curve may be obtained, according to Figure 5.3.

For a given flow rate Q, particles having settling velocity $\geq V_s$ will be completely removed. If y_0 represents particles portion with a settling velocity $< V_s$, then the percentage totally removed (in the sedimentation tank) can be expressed as $1-y_0$. Also, for each particle size with $V_a<V_s$, the removal proportion can be expressed according to Equation (5.15). For various particle sizes in this group, the percentage of removal is:

$$\int_0^{y_0} \frac{V_a}{V_s}\, dy$$

The overall fraction of particles removed would be:

$$F = (1-y_0) + \frac{1}{V_s}\int_0^{y_0} V_a\, dy \qquad 5.16$$

Equation (5.16) can be rearranged as:

$$F = (1-y_0) + \frac{1}{V_s}\sum V_a \Delta y \qquad 5.17$$

Example 5.1. Analyses of type-1 settling. A clarifier is designed to have a surface overflow rate of 30.0 m^3/m^2.d. Compute the overall removal efficiencies with settling analyses data, illustrated in Table 5.2. The water temperature is 15°C, and particle specific gravity is 1.20.

Table 5.2 Results of settling test analyses.

Particle size mm	Weight fraction <size, %
0.10	10
0.08	20
0.07	38
0.06	70
0.05	82
0.04	92
0.02	99
0.01	100

Solution

1. *Determining settling velocities employing Stokes law*

At 15°C the kinematic viscosity of water, μ is 0.00113kg/(s.m) and specific gravity of water is 0.9990.

The settling velocities of the fraction of particles (V_a- Equation 5.16) to be removed by the clarifier can be computed, by applying Stokes law (Equation 5.12):

$$V_a = \frac{g\,(\rho_s - \rho)\,d^2}{18\mu}$$

$$= \frac{9.81\ \text{m/s}^2 (1200 - 999)\ \text{kg/m}^3 \times d^2}{18 \times 0.00113\ \text{kg}/(sm)} = 96{,}942 d^2\ \text{m/s}$$

2. *Calculate V_a for each particle size from Table 5.2*

Particle size, d = 0.1 mm = 0.0001 m

Therefore, $V_a = 96942\ (0.0001)^2 = 0.969$ mm/s

For other particle sizes, the calculation of settling velocities is similar, as recorded below.

Particle size mm	Weight fraction <size %	Settling velocity V mm/s
0.10	10	0.969
0.08	20	0.620
0.07	38	0.475
0.06	70	0.349
0.05	82	0.242
0.04	92	0.155
0.02	99	0.039
0.01	100	0.010

3. *Construction of settling velocities vs. cumulative distribution curve*

The graphical representation of settling velocities (V) vs. fraction of particles with less than stated velocity is illustrated in Figure 5.3.

4. *Calculation of SOR (V_s)*

V_s = 30 m/d = 0.34 mm/s

Particles with settling velocities greater than 0.34 mm/s will be completely removed.

5. *Find the fraction (1-y_o)*

For V_s = 0.34 mm/*s*, y_0 = 0.297 (from Figure 5.3)

and 1-y_0 = 1-0.297 = 0.703

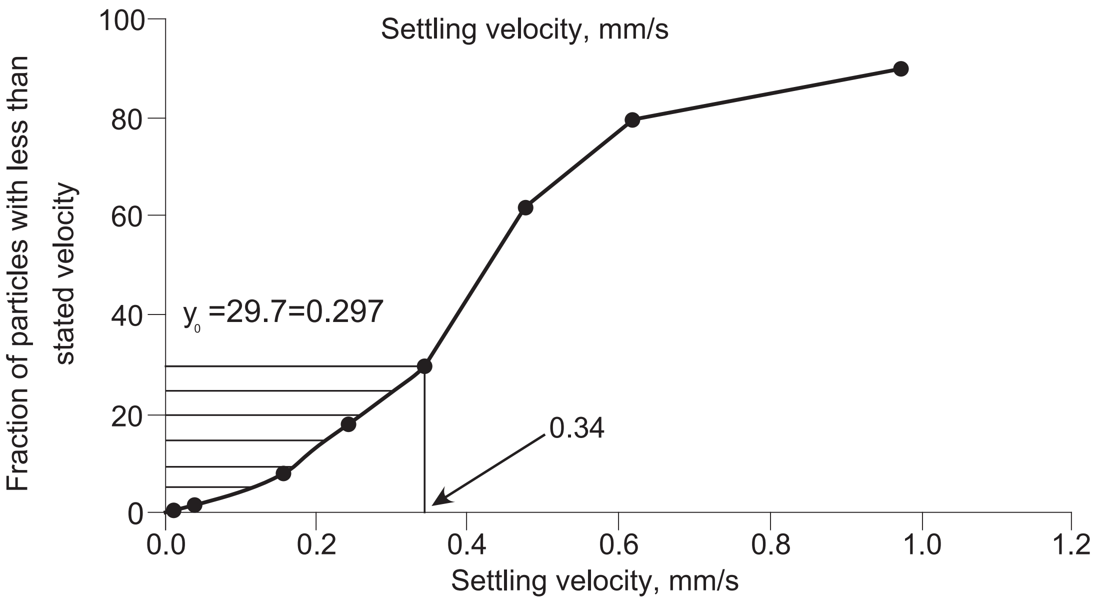

Figure 5.3 **Cumulative percentage removal vs. settling velocities graph.**

6. Graphical determination of $\sum V\Delta y$ *from Figure 5.3*

Δy	0.05	0.05	0.05	0.05	0.05	0.047
V	0.08	0.12	0.17	0.22	0.27	0.31
$V\Delta y$	0.004	0.006	0.008	0.01	0.02	0.02
$\sum V\Delta y$	0.068					

7. Determine overall removal F

The overall removal efficacy can be computed, employing Equation (5.17).

$$F = (1 - y_0) + \frac{1}{V_s} \sum V\Delta y$$

$$= 0.703 + \frac{0.068}{0.34} = 90\%$$

Settling column analysis. Direct application of the procedures, as described in the previous sections is limited in wastewater treatment due to uncertainty of particles sizes, and lack of spherical shapes. As such, an indirect method of measuring settling velocities of discrete particles in dilute suspension was proposed by Camp (1943), commonly referred as settling column analysis. According to the process, a settling column is employed as demonstrated in Figure 5.4; the suspension to be tested is placed in a column, and is being mixed completely to ensure uniform particle distribution. The suspension is then allowed to settle.

If a particle (of the solution) is just at the surface at time equal to zero (Figure 5.4), and its settling velocity is such that it arrives at a later time i.e. $t = t_0$, then the average settling velocity of the particle is:

$$v_0 = \frac{\text{distance traveled}}{\text{time of travel}} = \frac{Z_0}{t_0}$$

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If another particle is suspended at a distance Z_P above the sampling port (Figure 5.4), and its terminal settling velocity is such that (less than v_0) it arrives at the port at the same time as previous particle. So, its settling velocity can be computed as,

$$v_p = \frac{\text{distance traveled}}{\text{time of travel}} = \frac{Z_p}{t_0}$$

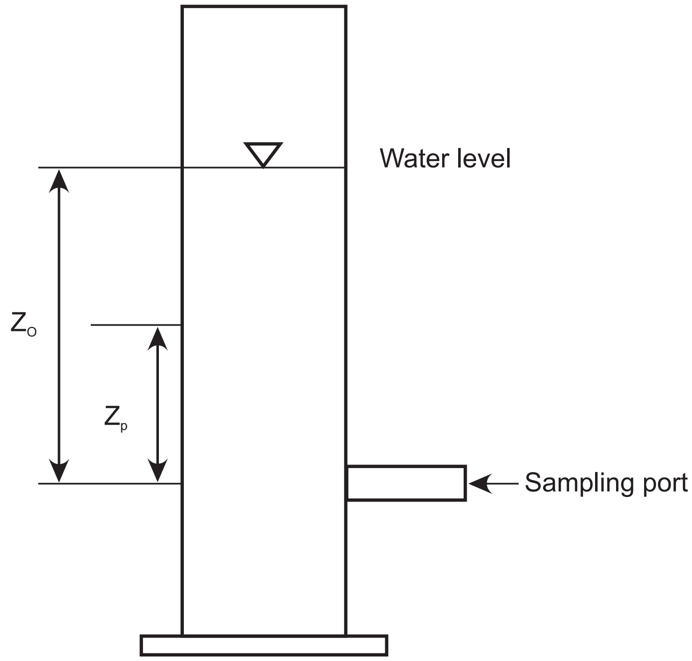

Figure 5.4 **Settling column for type-1 suspension analyses.**

Since, the time of travel is equal for the two particles, it can be shown:

$$t_0 = \frac{Z_0}{v_0} = \frac{Z_p}{v_p}$$

$$\text{and } \frac{Z_p}{Z_0} = \frac{v_p}{v_0}$$

Some generalized statements can be made on the above equation:

- Particles with diameter equal or greater than d_0, will have settling velocities equal or greater than v_0; these particles will arrive or pass sampling port in time t_0.
- Particle with diameter $d_p < d_0$ will have a terminal settling velocity $v_p < v_0$; such particles will arrive at or pass sampling port at time t_0, provided its original position was at or below a point Z_p.
- If the suspension is mixed uniformly (all particle size are randomly distributed from top to bottom), then the fraction of particles (of size d_p and settling velocity v_p) arriving at the sampling port in time t_0 can be expressed as $Z_p/Z_0 = v_p/v_0$. So the removal efficiency of any particle size from suspension, is the ratio of settling velocity of that particle to v_0, defined by Z_0/t_0.

In column settling process, an apparatus (Figure 5.4) is filled with suspension, maintaining a water depth of 2m. The suspension is mixed to ensure an initially uniform particle distribution, and an initial concentration C_0 is measured. After the suspension is allowed to settle for a time period t_1, a second sample is drawn off, and another concentration C_1 is measured. Particles having concentration C_1, must have settling velocities less than Z_0/t_1; So, the mass fraction of particles with $v_1 < Z_0/t_1$ is:

$$x = \frac{C_i}{C_0}$$

The process is repeated several times with x_i being the mass fraction of particles with $v_i < Z_0/t_i$; the values are plotted on a graph, as shown in Figure 5.5, and the particles fraction with any settling velocities can be computed.

For a given detention time t_0, an overall percent removal can be obtained. All particles, with settling velocities greater than $v_0 = Z_0/t_0$ will be 100% removed. So the fraction $(1\text{-}x_o)$ will be removed completely in time t_0. The remaining particles will be removed according to the ratio v_i/v_0. If the equation relating to v and x is known, the area can be computed by integration, as expressed in Equation (5.18).

$$X = 1 - x_0 + \int_0^{x_0} \frac{v_i}{v_0}\, dx = 1 - x_0 + \frac{\sum \Delta x v_t}{v_0} \qquad 5.18$$

Example 5.2. Settling column analysis of type-1 suspension. A settling analysis is run on a type 1 suspension. The column is 2m deep, and the dataset are given below. Compute the removal efficiency in a settling basin with a load of 25 m/d.

Time, min	0	60	80	100	130	200	240	420
Conc., mg/L	400	200	175	150	140	110	70	10

Solution

1. Calculate mass fraction remaining and corresponding settling rate

The mass fraction at 60 min can be computed as 200/400 = 0.5; the procedure is similar to compute mass fractions, for the following time periods. The settling rates at corresponding time period can be obtained from the ratio of travelled distance (i.e. column height) and corresponding time.

Time, min	60	80	100	130	200	240	420
Mass fraction remaining	0.5	0.43	0.38	0.35	0.28	0.18	0.02
$v_s \times 10^2$ m/min	3.3	2.5	2.0	1.5	1.0	0.83	0.48

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2. Plot mass fraction remaining vs. settling velocity

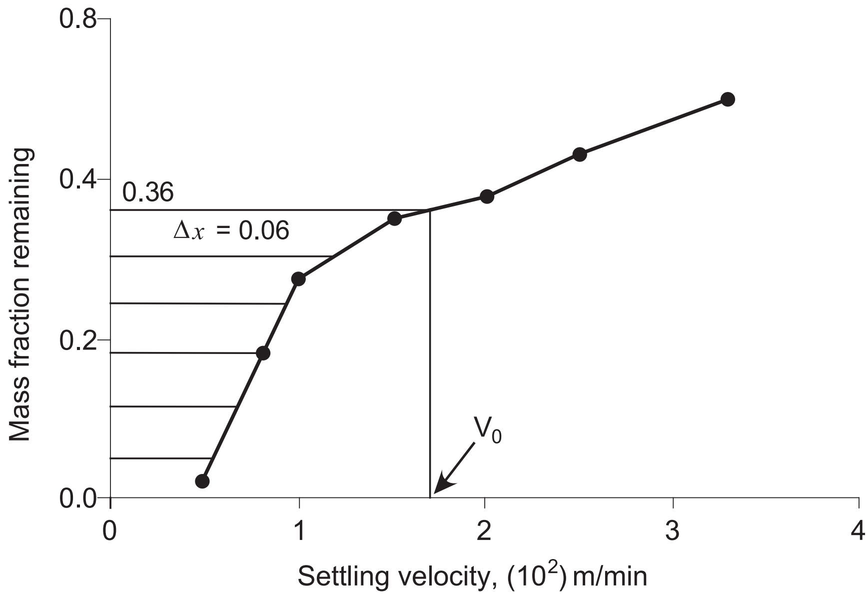

Figure 5.5 **Plot of mass fraction remaining vs. settling velocity.**

3. Determine the value of v_0

$$v_0 = 25\ \mathrm{m^3/m^2}d = 1.74 \times 10^{-2}\ \mathrm{m/min}$$

4. Determine the value of x_0

From Figure 5.5, x_0 can be determined as, $x_0 = 36\%$

5. Determine $\Delta x v_t$ by graphical integration

Δx	v_t	$\Delta x v_t$
0.06	0.49	0.03
0.06	0.60	0.04
0.06	0.73	0.04
0.06	0.90	0.05
0.06	0.98	0.06
0.06	1.21	0.07
		$\sum \Delta x v_t = 0.29$

6. Determine overall removal efficiency

$$\text{From Equation (5.18): } X = 1 - x_0 + \frac{\sum \Delta x v_t}{v_0}$$

$$= 1 - 0.36 + \frac{0.29}{1.74} = 80\%$$

5.2 Flocculant Settling

Particles in sedimentation tanks often do not follow discrete settling due to their extremely light size. In fact, they come into contact with each other resulting in agglomeration of particle size, allowing faster settling.

To analyze such flocculant settling of particles, settling column analyses is generally employed. The samples are drawn off at several time intervals and depth of the column. These concentrations are used to calculate mass fraction removed (instead of mass fraction remaining), at each depth for each time interval, as illustrated in Equation (5.19):

$$X_{ij} = \left(1 - \frac{C_{ij}}{C_0}\right) \times 100 \qquad 5.19$$

where X_{ij} = mass fraction (in percentage) removed at i^{th} depth at j^{th} time interval.

The values are plotted in graphical forms (Figure 5.6), and isoremoval lines are drawn. The slope at any point on any line is the velocity of the fraction of particles represented by that line.

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Example 5.3. Flocculant settling analyses. A column analysis of a flocculating suspension is being performed. The initial solids concentration is 300 mg/L, with the resulting matrix as illustrated below. What will be the overall efficiency of the removal of a settling basin, having 3 m depth with 90 min detention time?

Depth, m	Sampling time, min					
	30	60	90	120	150	180
0.5	120	70	45	33	25	20
1	125	115	91	60	50	40
1.5	210	140	110	80	60	50
2	215	160	130	105	90	60
2.5	230	190	155	120	100	70
3	240	195	165	130	115	90

Solution

1. Determine the removal rate at each depth and associated time interval

The mass fraction removed, at each depth for each time can be computed employing Equation (5.19).

$$X_{ij} = \left(1 - \frac{C_{ij}}{C_0}\right) \times 100$$

The velocity becomes greater (e.g. slope of isothermal lines become steeper) at greater depth. These common characteristics of flocculating suspensions indicate agglomeration of particle size and settling velocity, due to continuous collision and aggregation with other particles.

Depth, m	Sampling time, min					
	30	60	90	120	150	180
0.5	60	76	85	89	92	93
1	58	62	70	80	83	87
1.5	30	53	63	73	80	83
2	28	47	56	65	70	80
2.5	23	37	48	60	66	76
3	20	35	45	56	62	70

2. Plot concentration lines as in the graph, and construction of vertical line $t_0 = 90min$

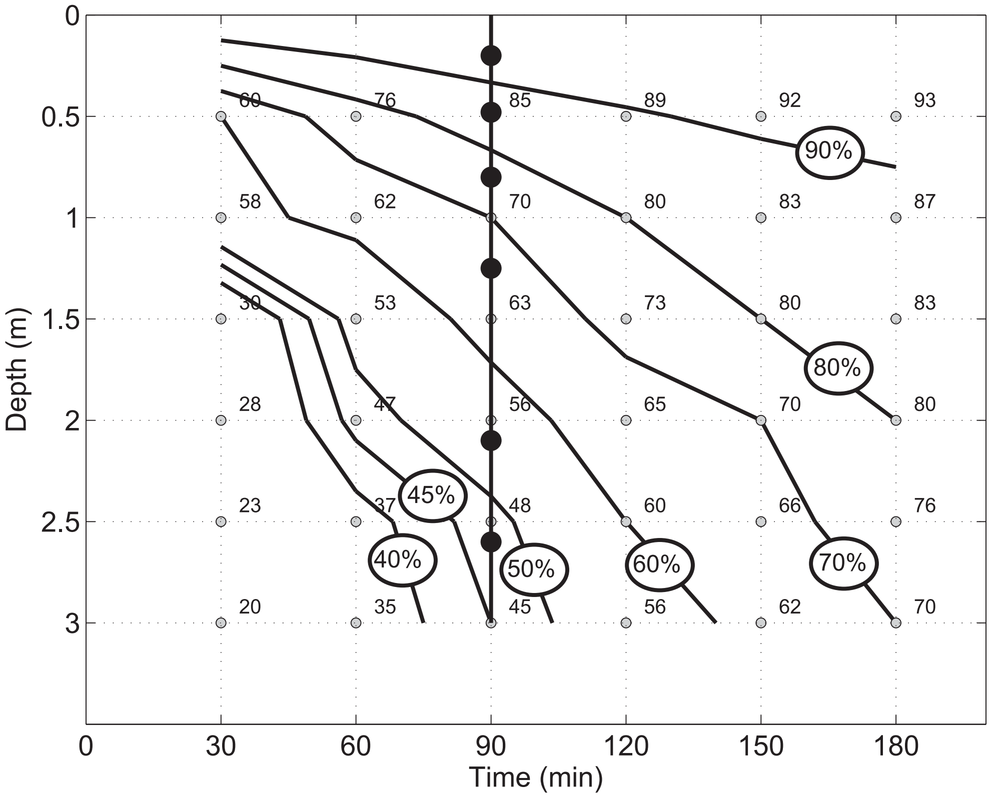

Figure 5.6 **Plot of concentration lines.**

3. Calculation of removal efficacy

The removal efficiency can be calculated employing the following formula:

$$\text{Removal efficiency, } R = r_0 + \sum \frac{\Delta r z_i}{Z_0}$$

$$= 0.45 + \frac{0.61}{3} = 65\%$$

where r_0 = fraction removed on curve with t

Δr = i^{th} increment of removal

z_i = depth to midpoint of increment

Δr	z_i	Δrz_i
0.05	2.60	0.13
0.1	2.10	0.21
0.1	1.25	0.12
0.1	0.80	0.08
0.1	0.48	0.05
0.1	0.20	0.02
		$\Sigma \Delta rz_i = 0.61$

5.3 Tube and Lamella Clarifiers

In a typical sedimentation tank, the upflow velocity of the incoming fluid must be less than the velocity of the slowest settling particle to be removed. Such criteria can also be achieved by inserting a number of subfloors into horizontal sedimentation basins; tube and lamella clarifiers are the examples of such alternative concepts. Tube settlers employ plastic modules, inclined and spaced uniformly. Lamella settlers comprise of uniformly spaced inclined panels, constructed with plastic, rawhide materials (Figure 5.7). The resultant velocity (on a particle), due to upward water flow and vertically downward settling velocity directs the particle to the bottom wall of a tube, or toward the lamella. The particle slides down the surface towards the bottom of the tank.

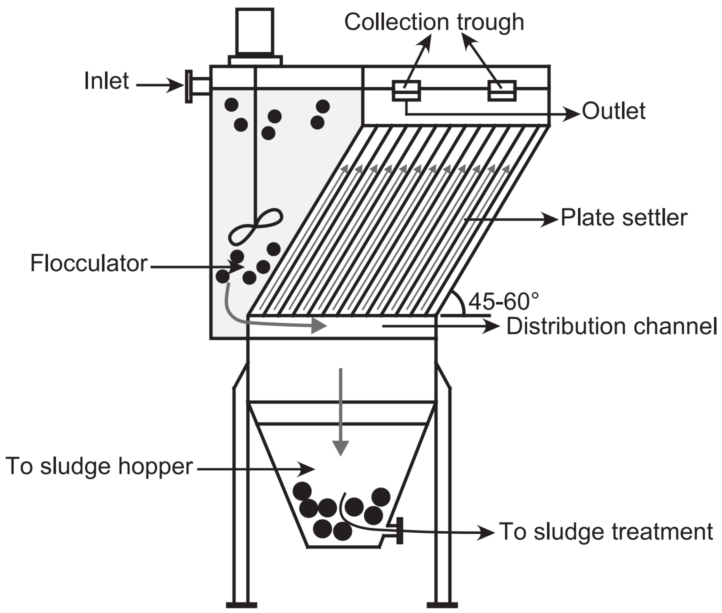

Figure 5.7 **Configuration of lamella clarifier.**

The plates or tube settlers are set at an angle between 45° and 60° above the horizontal (Figure 5.8), to promote self-cleaning. If the angle is increased above 60°, solid removal efficiency is decreased; on the other hand, the solids will accumulate on the plates or tubes, when inclined at angles less than 45°. The plates are usually spaced 50 mm apart, having an inclined length of 1-2 m. The accumulated solids (on plates or tube settlers) must be washed out periodically, to prevent biological growth.

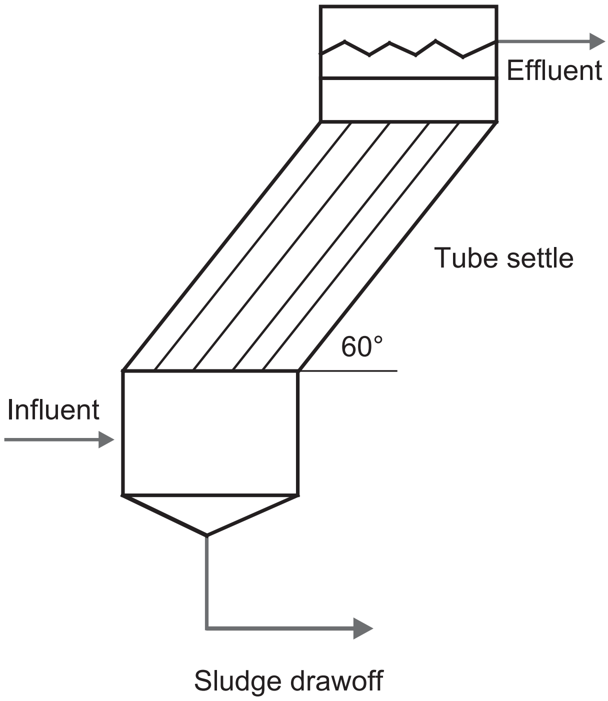

Figure 5.8 Configuration of tube settlers or plates at 60° angle.

5.4 Guidelines for the Design of Sedimentation Tanks

This section provides a brief discussion of the important factors, necessary for the design of rectangular and circular sedimentation tanks. Subsequently, the design procedures of these sedimentation tanks have also been illustrated in Example 5.4.

a. Rectangular sedimentation tank design criteria

- Overflow rate: The overflow rate in rectangular sedimentation tank usually ranges between 24.5-49 $m^3/m^2.d$, depending on application.
- Side depth: The side depth of rectangular clarifiers should be shallow, but should not be less than 3 m.
- Detention time: Primary sedimentation tanks are designed to provide detention time ranging between 1.5-2.5 h. Tanks providing shorter detention periods (0.5h-1h), which generally exhibits less removal of suspended solids, are sometimes used prior to biological treatment units.
- Weir loading rate: Large weir overflow rates are associated with excessive velocities at the outlet. These velocities extend backward into settling zone causing rise of particles and flocks, thereby degrading the effluent water quality. Weir loading must not exceed 250 m^3/d linear meter (based on design peak hourly flow), for plants with an average of 3785 m^3/d inflow; for plants with an average inflow greater than 3785 m^3/d, weir loading must not exceed 373 $m^3/m.d$ linear meter (based on design peak hourly flow).
- Scour velocity: To avoid resuspension (scouring) of the settled particles, horizontal velocities through the tank should be kept low. The calculation of scouring velocities is performed employing Equation (5.20).

$$V_H = \left[\frac{8k\,(s-1)\,gd}{f} \right]^{1/2} \quad 5.20$$

where V_H = horizontal velocity to produce scour, m/s
k = constant depending on material type being scoured
s = specific gravity of particles
g = acceleration due to gravity, 9.81m/s^2
d = particle diameter, m
f = Darcy-Weisbach friction factor (unit-less), 0.02-0.03

- BOD and TSS removal: The estimation of BOD and total suspended solid (TSS) removal efficacy in sedimentation tanks is defined through the following formula:

$$R = \frac{t}{a+bt} \quad 5.21$$

where R = expected removal efficiency, %
t = nominal detention time
a, b = empirical constants, as illustrated in Table 5.3

Table 5.3 **Typical empirical values for sedimentation tank design[a].**

Typical empirical values		
Pollutant	*a*	*b*
BOD	0.018	0.020
TSS	0.0075	0.014

[a]*Metcalf and Eddy (2003)*

The calculation of BOD and TSS removal efficacy (from wastewater) in sedimentation tanks can also be computed graphically from overflow rate, as illustrated in Figure 5.9.

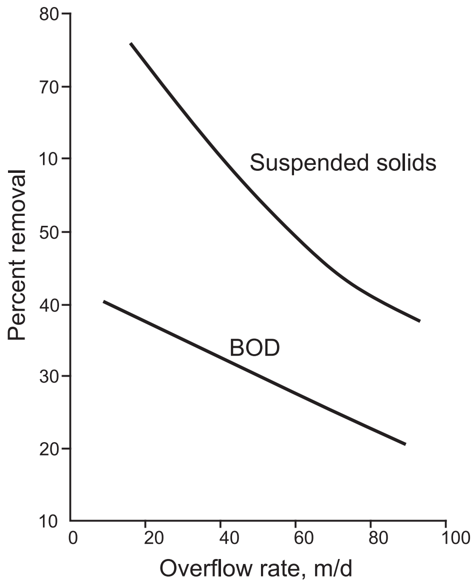

Figure 5.9 **Graphical relation of BOD and TSS removal efficacy vs. overflow rate in sedimentation tanks (Steel and McGhee, 1979).**

CHAPTER 5

b. Circular basin design

Despite the functional zones of circular basins are similar to rectangular ones, the flow regime is different; the flow is directed upward through the inlet baffles in circular basins. These inlet baffles are designed to be 10-20% of the basin diameter, extending 0.9-1.8 m below the wastewater surface (McGhee, 1991). Circular basins are usually designed with a sidewall depth of 3 m; floor slope is typically 300 mm (horizontal) to 25 mm (vertical) (Aqua-Aerobic Systems, 1976).

Outlet weirs are extended around the basin periphery. Excessive weir overflow should never be a problem in these basins, due to the usage of the entire circumference. To prevent extremely thin sheet outflow, the effective overflow area is reduced by providing V-notched plates on overflow weirs. The schematic diagram of circular basins has been provided in Figure 5.10.

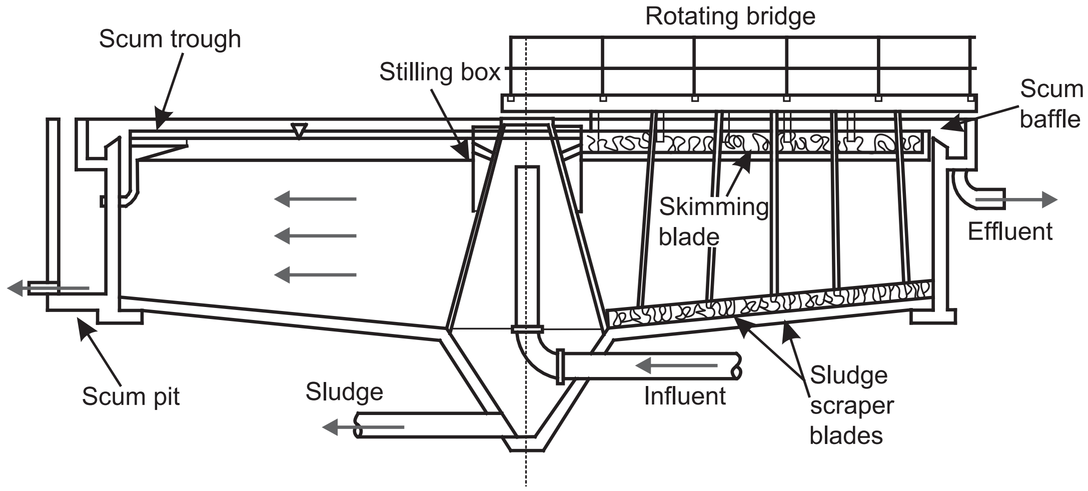

Figure 5.10 Diagram of circular basin for wastewater treatment.

Table 5.4 and 5.5 indicate typical design guidelines, and dimensional data of sedimentation tanks.

Table 5.4 Typical design information of primary sedimentation tanks.

Treatment type	Surface settling rate $m^3/m^2.d$		Depth m	Detention time h	Source
	Average	Peak			
Primary settling followed by secondary treatment	33-50	81-122	3-3.7	-	US EPA, 1975
	30-50	80-120	-	2.0	Metcalf and Eddy. 2003
Primary settling with waste activated sludge return	24-33	49-61	3.7-4.6	-	US EPA, 1975
	24-32	48-70	-	2	Metcalf and Eddy. 2003

Table 5.5 **Typical dimensional values of sedimentation tanks.**[a, b]

Parameter	Unit	Value	
		Range	Typical
Detention time	h	1.5-2.5	2
Overflow rate	$m^3/m^2.d$	20-70	-
Average flow		32-48	-
Peak flow		80-120	-
Weir loading	$m^3/m.d$	125-500	250
Dimensions			
Rectangular			
Depth	m	3-5	3.6
Length	m	15-90	25-40
Width	m	3-24	6-10
Circular			
Depth	m	3-5	4.5
Diameter	m	3.6-60	12-45
Bottom slope	mm/m	60-160	80

[a]*Value after Droste, (1997),* [b]*Metcalf and Eddy, (2003).*

c. Inlets of the sedimentation tanks

Inlets of the sedimentation tanks need to be designed to distribute the flow of wastewater as uniformly as possible, so that the best possible flow pattern is maintained.

For rectangular sedimentation tanks, various baffled inlet arrangements have been used which are effective for flow distribution. Typical arrangements are shown schematically in Figure 5.11.

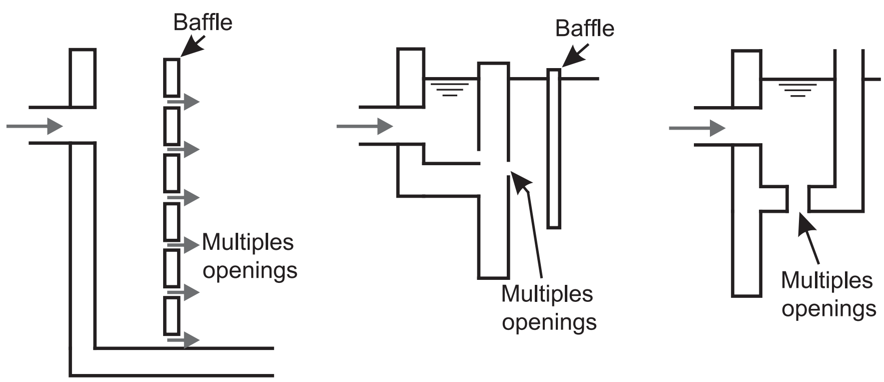

Figure 5.11 **Schematic diagram of typical rectangular sedimentation tank inlets.**

With circular tanks, the radial flow pattern of the wastewater from the inlet is inherently less stable than the horizontal flow in a rectangular tank. Careful design is needed to achieve a stable radial flow pattern. Typical arrangements are shown in Figure 5.12 for: (a) side feed; (b) vertical pipe feed; and (c) slotted vertical pipe feed. In all these cases, the primary design principles are that the energy of flow must be dissipated and flow velocity distribution must be uniform.

CHAPTER 5

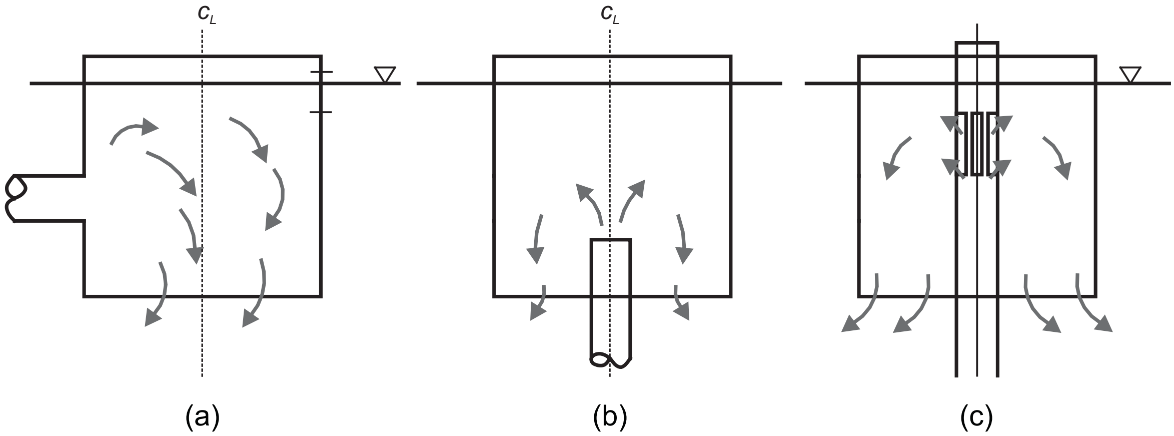

Figure 5.12 **Standard arrangements of centre-feed inlets for circular clarifiers: (a) side feed, (b) vertical pipe feed, (c) slotted vertical pipe feed.**

Example 5.4. Design problem. Design (a) rectangular sedimentation tank, and (b) circular tank employing the following dataset:

- Average flow rate, $Q_{av} = 10{,}000\ m^3/d$.
- Peak hourly flow rate, $Q_p = 30{,}000\ m^3/d$.
- Specific gravity of the particles to be removed, $s = 1.25$.
- Diameter of the particles, $d = 100\ \mu m$.
- Darcy-Weisbach fraction factor, $f = 0.025$.
- Scouring material constant, $k = 0.05$.

Solution

a. Design of rectangular tank

Step 1. Assumption of the design values

Assume the surface settling rate, $v = 30 m^3/(m^2.d)$, minimum depth, $d = 4m$, and maximum weir loading rate= $120 m^3/d.m$.

Step 2. Determine surface area

Use two settling tanks; each of the tanks to be designed for an average flow =10000/2 = 5000 m3/d.

Surface area $A = \dfrac{Q}{v} = \dfrac{5000\ m^3/d}{30\ m^3/(m^2d)} = 167\ m^2$

Step 3. Determine length (l) and width (w) of the tank

Assume a length-width ratio (l/w) to be 4:1.

Surface area $w \times 4w = 167\ m^2$

Width $w = 6.5\ m$

Length $l = 26\ m$

The required surface area, $A = 6.5\ m \times 26\ m = 169\ m^2$

Step 4. Determine volume of the tank

Volume of the tank $V = 169\ \text{m}^2 \times 4\ \text{m} = 676\ \text{m}^3$

Step 5. Check detention time

At average design flow, $t_{av} = \dfrac{V}{Q} = \dfrac{676\ \text{m}^3}{5000\ \text{m}^3/d} = 3.2\ \text{h}$

At peak design flow, $t_p = \dfrac{V}{Q} = \dfrac{676\ \text{m}^3}{15000\ \text{m}^3/\text{d}} = 1.08\ \text{h}$

Step 6. Check overflow rate (v) at peak flow

$$v = 30\ \text{m}^3/\text{m}^2\text{d} \times \left(\frac{30000}{10000}\right) = 90\ \text{m}^3/(\text{m}^2\text{d})$$

Step 7. Compute length of outlet weir

Length=flow/weir load rate

$$= \frac{5000\ \text{m}^3/\text{d}}{120\ \text{m}^3/(\text{dm})} = 42\ \text{m}$$

Step 8. Compute scour velocity (V_H)

From Equation (5.20): $V_H = \left[\dfrac{8k\,(s-1)\,gd}{f}\right]^{1/2}$

$$= \left[\frac{8 \times 0.05 \times 0.25 \times 9.81 \times 100 \times 10^{-6}}{0.025}\right]^{1/2} = 0.063\ \text{m/s}$$

Step 9. Compare scour to peak flow horizontal velocity

Peak flow horizontal settling velocity through the tank:

$$V = \frac{Q}{A_x}$$

$$= \frac{15{,}000\ \text{m}^3/\text{d}}{(6.5\ \text{m} \times 4\ \text{m})} \times \frac{1}{24\ \text{h/d} \times 3600\ \text{s/h}} = 0.00667\ \text{m/s}$$

Horizontal settling velocity at peak flow is less than scour velocity (V_H-step 8); therefore, the settled matter should not be resuspended.

Step 10. BOD and TSS removal at peak flow

Removal percentages of BOD and TSS removal can be calculated employing Equation (5.21):

$$\text{BOD removal} = \frac{t}{a+bt}$$

$$= \frac{1.08}{0.018+(0.020)(1.08)} = 27.2\%$$

$$\text{TSS removal} = \frac{t}{a+bt}$$

$$= \frac{1.08}{0.0075+(0.014)(1.08)} = 47.7\%$$

b. Circular tank design

Step 1. Calculating surface area

Use two circular clarifiers; the average incoming flow will be equally divided between two tanks.

$$\text{Surface area } A = \frac{5000 \text{ m}^3/d}{30 \text{ m}^3/\text{m}^2 d} = 167 \text{ m}^2$$

Therefore $\pi r^2 = 167 \text{m}^2$

Radius $r = 7.3$ m

Diameter $d = 14.5$ m

The required surface area for each tank, $A = \pi r^2 = 167.3 \text{ m}^2$

Step 2. Calculate the overflow rate

$$\text{Overflow rate} = \frac{5000 \text{ m}^3/\text{d}}{167.3 \text{ m}^2} = 29.8 \text{ m}^3/\text{m}^2\text{d}$$

Step 3. Compute the detention time (t)

Provide a sidewall depth of 3m.

$$t = \frac{167.3 \text{ m}^2 \times 3 \text{ m}}{5000 \text{ m}^3/\text{d}} = 2.4 \text{ h}$$

Step 4. Determine wire loading rate

Provide an inboard weir of 12.2 m.

$$\text{Periphery length} = \pi \times 12.2 \text{ m} = 38.3 \text{ m}$$

$$\text{Weir loading} = \frac{5000 \text{ m}^3/\text{d}}{38.3 \text{ m}} = 130.4 \text{ m}^3/\text{md}$$

Step 5. Engineering diagram

The engineering diagram of the designed circular tank has been provided in Figure 5.13.

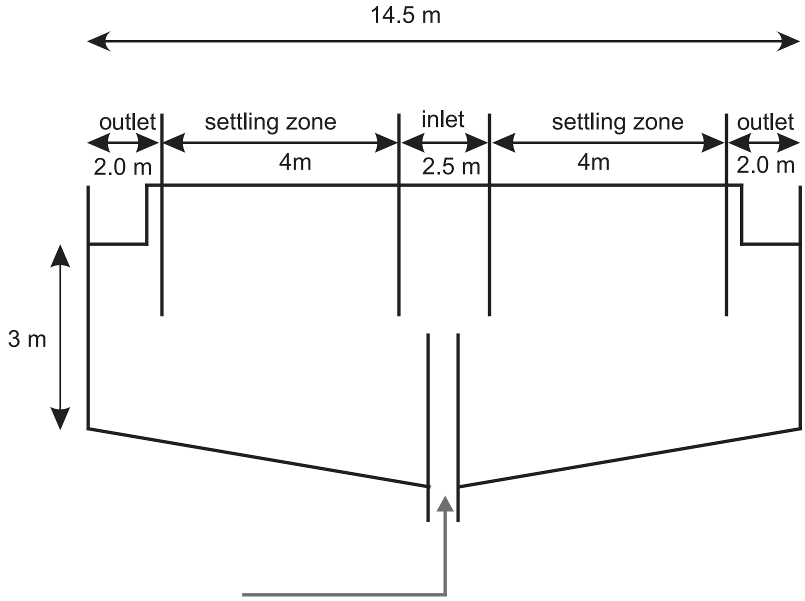

Figure 5.13 Engineering diagram of a circular sedimentation tank.

Questions

1. With necessary conditions derive the Stokes law equation, for the analyses of discrete particle settling in sedimentation tanks.
2. Enlist the important factors for the design of rectangular and circular sedimentation tanks.
3. A settling analysis is run on a type 1 suspension. The column is 2m deep, and the dataset are given below. Compute the removal efficiency in a settling basin with a load of 30 m/d; use settling column analyses method.

Time, min	0	50	1000	120	150	220	280	450
Conc., mg/L	450	300	225	200	120	80	50	15

4. Design rectangular sedimentation tanks for wastewater treatment, employing the following dataset. Assume necessary values, indicated in Example 5.4.
   - Average flow rate, $Q_{av} = 15{,}000\ m^3/d$.
   - Peak hourly flow rate, $Q_p = 45000\ m^3/d$.
   - Specific gravity of the particles to be removed, $s = 1.25$.
   - Diameter of the particles, $d = 120\ \mu m$.
   - Darcy-Weisbach fraction factor, $f = 0.025$.
   - Scouring material constant, $k = 0.05$

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Chapter 6

Secondary Treatment of Municipal Wastewater

Wastewater after receiving primary treatment (Chapter 5), generally contains 40-50% of the original suspended solids, along with substantial amount of initial dissolved organics and inorganics. As such, the effluent quality of primary treatment process does not often meet the discharge criteria in most countries. To polish such effluent (to discharge criteria), secondary treatment mechanism, commonly referred as biological treatment mechanism, usually follows primary treatment process. A typical secondary treatment process includes an aeration basin, followed by a secondary clarifier (Figure 6.1); the effluent quality (from secondary treatment) is enhanced due to: (a) removal of organics (i.e. soluble and colloidal form) through biodegradation; and (b) capturing of solids into biological films.

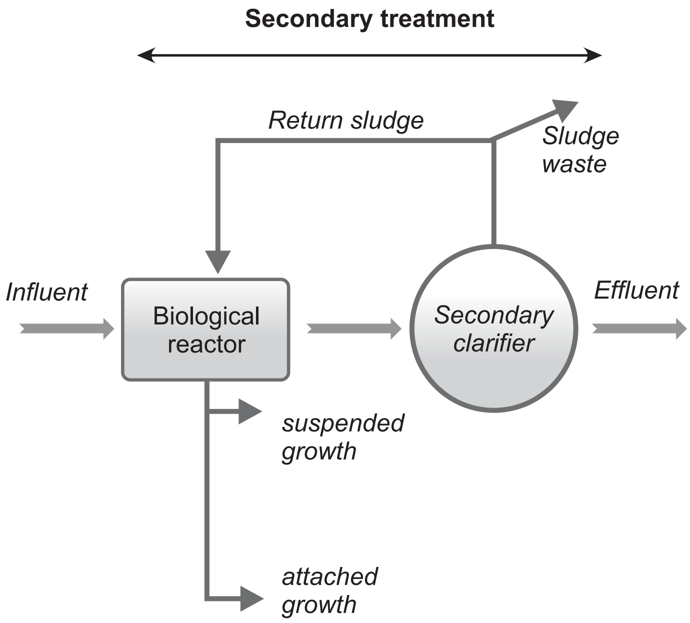

Figure 6.1 **Flow diagram of secondary treatment process.**

The volatile organic compounds such as: benzene, toluene and the dichlorobenzenes may be lost by volatilisation process during wastewater and sludge treatment at thickening, particularly if the sludge is aerated or agitated and by dewatering. The majority of volatilisation occurs through air stripping in aerated vessels. However, some studies (Melcer et al., 1992) reported that the stripping of volatiles may not be significant; biodegradation during secondary biological wastewater treatment may be the main mechanism of loss of the potentially volatile compounds. For example, Melcer et al. (1992) reported that biodegradation processes removed ≥ 90 % of the dichoromethane, 1,1,1-trichoromethane, trichloroethylene, toluene and xylene from a municipal wastewater. Volatilisation was only a significant mechanism of removal for 1,4-dichlorobenzene (20 %) and tetrachloroethylene (60 %).

This chapter provides a comprehensive description of organics removal kinetics, and biodegradation routes in secondary treatment plants. Sections 6.1-6.5 provide information on the common microorganisms observed in biological reactors, and associated kinetic equations that accelerate the removal of organics from wastewater. Sections 6.6-6.13 emphasize on the common variations of the treatment plants employed for the removal of organic materials from wastewater. Subsequently, section 6.14 demonstrates a step-wise procedure required for the design of secondary treatment units.

6.1 Common Organisms of Biological Process

Biological treatment plants are generally designed to facilitate substantial growth of different types of microorganisms, for wastewater treatment. The most common organisms that grow in biological reactors are bacteria, fungi, algae, protozoa and rotifiers.

Bacteria. Bacteria are the most common microorganisms in biological reactors; they are unicellular prokaryotic organisms (Figure 6.2), and distinguishable according to their shape (coccoid, spherical, rod-like, spiral, and filamentous). Bacteria are capable of stabilizing colloidal and solid organic matter outside the cell by means of extra cellular enzymes, thereby contributing to organics removal (from wastewater). Each type of bacteria lives and multiplies (through binary fission) under specific environmental conditions, such as: light, air, water, food, temperature, pH, and dissolved oxygen (DO).

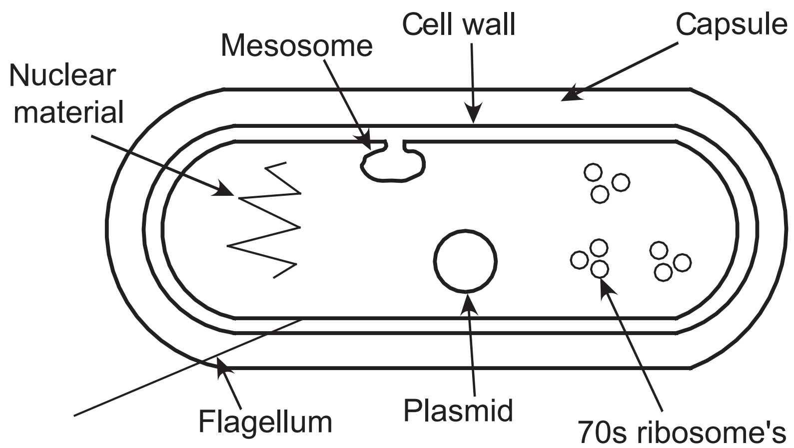

Figure 6.2 Single cell bacteria.

According to the metabolic requirements, bacteria can be classified as autotrophic and heterotrophic bacteria. Autotrophic bacteria use carbon from carbon dioxide; heterotrophic bacteria utilize organic carbon for the formation of cell tissue. Organisms that use light as their energy source are known as phototrophic bacteria, that can be either heterotrophic (such as sulphur bacteria), or autotrophic (such as photosynthetic bacteria). Organisms deriving energy from chemical reactions are known as chemotrophs, and can be either heterotrophic (most bacteria), or autotrophic (such as *Nitrosonomas* and *Nitrobacter*). Table 6.1 provides a general classification of bacteria based on metabolism activities.

Table 6.1 General classification of microorganisms depending on metabolism.

Classification	Energy source	Carbon source
Autotrophic		
Photo-autotrophic	Light	CO_2
Chemo-autotrophic	Inorganic oxidation and reduction	CO_2
Heterotrophic		
Photo-heterotrophic	Light	Organic Carbon
Chemo- heterotrophic	Organic oxidation and reduction	Organic Carbon

Table 6.2 Classification of bacteria with temperature variation.

Type	Temperature °C	
	Range	Optimum
Psychrophilic	-2-30	12-18
Mesophilic	20-45	25-40
Thermophilic	45-75	55-65

Bacteria can also be classified into the following groups (Table 6.2), depending on the survival temperature range.

Fungi. Fungi are multicellular, non-photosynthetic plants. They are strict aerobes, and have the ability to grow under low moisture conditions. The optimum pH for fungi is 5.6; the survival range is 2-9. Fungi compete with bacteria for food, where bacteria are mostly favored. Fungi also have lower nitrogen requirement. The ability of fungi to survive under low pH and nitrogen limiting condition makes them very important, in terms of providing biological treatment to some industrial wastewaters. However, because of large filamentous shape, fungi tend to settle poorly, thereby limiting sedimentation.

Algae. Algae are photosynthetic microorganisms. They produce bad taste and odors, so they are undesirable in water supplies. In oxidation ponds, algae are valuable because they have the ability to produce O_2 by photosynthesis. At night, when light is no longer available for photosynthesis, they use up O_2 for respiration.

Protozoa. Protozoa are single-celled animals. They may grow under aerobic, anaerobic or facultative environment. They are larger than bacteria and often consume bacteria. As a result, they perform as polishers of the biological treatment effluents.

Rotifers. Rotifers are simple multi cellular animals. They feed on bacteria, small protozoa, thus stabilizing the waste. Since rotifiers require higher dissolved oxygen content, their presence is a good indicator of the treated wastewater stability.

6.2 Cell Growth of Bacteria

When small amount of bacteria is inoculated in a fixed volume with culture medium, they generally require time to acclimatize to the new environmental conditions. This period is referred as *lag phase*; during such phase the density of bacteria remains constant. When excess food is applied, bacteria cells divide at a rate, determined by their generation time and ability to process food. This phase is known as *log phase*; such phase results rapid increase of bacterial density, associated with maximum substrate removal rate. The log phase exhausts some nutrients and accumulation of toxic substances which decreases cell growth followed by growth inhibition. At this point, the growth of cell remains stationary which is referred as *stationary phase*. As the food depletes and bacterial density increases, cell growth further declines. This phase is referred as *log death phase* or *endogenous phase*. Starvation and decrease of bacterial cell metabolism is a common feature in this phase. Figure 6.3 illustrates different phases usually observed in bacterial growth curve.

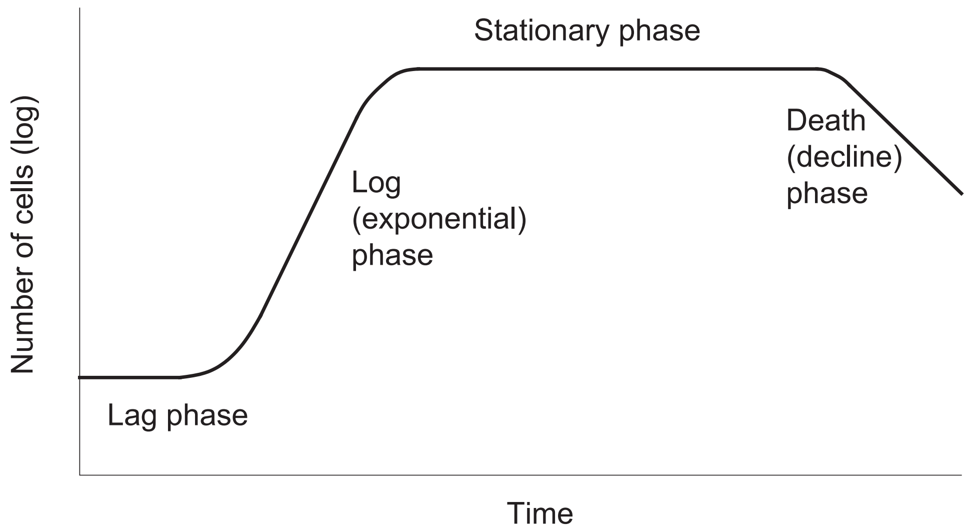

Figure 6.3 Bacterial growth curve.

6.3 Kinetic Transformations in Biological Reactors

A wide variety of biological conversions can occur in biological treatment plants (for pollutant removal) such as: (a) biological growth; (b) hydrolysis; and (c) decay, illustrated in Figure 6.4.

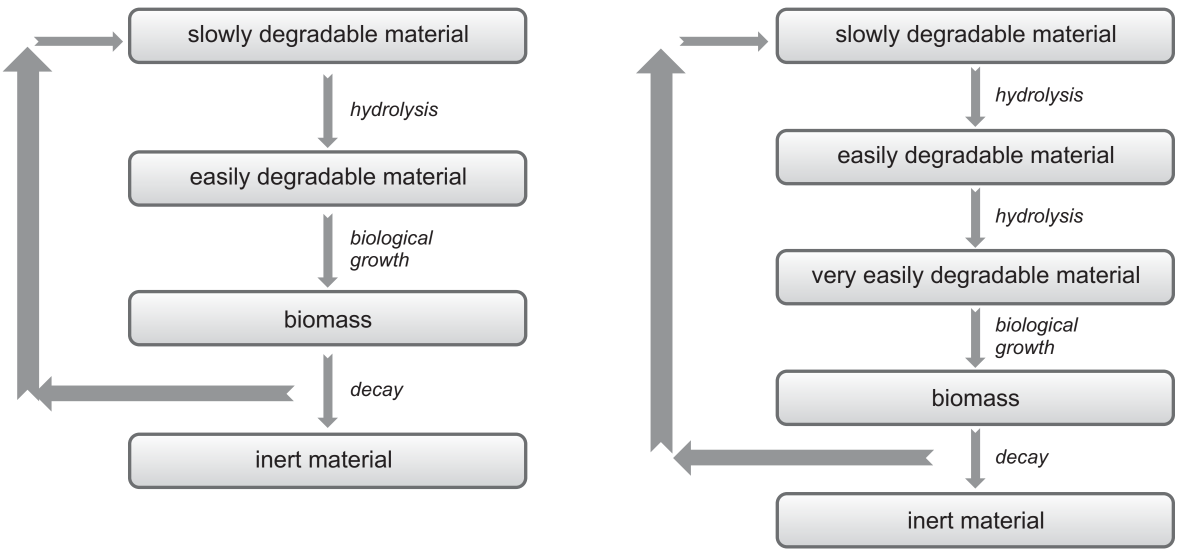

Figure 6.4 Biological conversion in treatment plants.

Biological growth. Bacteria in wastewater treatment process can only utilize very simple and small compounds for growth, demonstrated in Equation (6.1).

$$\frac{dX}{dt} = r_{g,X_B} = \mu X_B \qquad 6.1$$

where r_g, X_B = biological growth rate, mg/L.d
μ = specific growth rate, d^{-1}
X_B = biomass concentration, mg/L

At limiting substrate concentration, the specific growth rate of bacteria can be expressed through Monod kinetics:

$$\mu = \frac{\mu_{max} S}{S + K_S} \qquad 6.2$$

where S = limiting substrate concentration in solution, mg/L

K_S = half saturation constant at one half the maximum growth rate (Figure 6.5), mg/L; K_S represents the affinity of the microorganism for the substrate

μ = specific growth rate, d^{-1}

μ_{max} = maximum specific growth rate, d^{-1}

Combining Equation (6.2) and (6.1) yields:

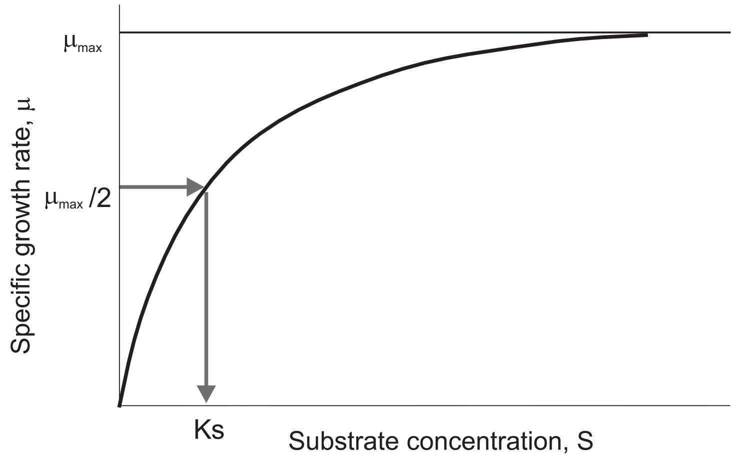

Figure 6.5 **Monod half saturation constant.**

$$r_{g,X_B} = \frac{\mu_{max} S}{S + K_S} X_B \qquad 6.3$$

Several observations can be made from Equation (6.3); where there is an excess of the limiting food (i.e. $S >> K_S$), the growth rate constant is almost equal to maximum specific growth rate μ_{max}, and the system becomes biomass limited (i.e. the specific growth rate is controlled by μ_{max}). Such correlation has been demonstrated in Equation (6.4):

$$r_{g,X_B} = \frac{\mu_{max} S}{S\left(1 + \frac{K_S}{S}\right)}$$

$$r_{g,X_B} = \mu_{max} \qquad 6.4$$

Equation (6.4) is a first order reaction kinetics, denoting that the growth rate is proportional to the first power of biomass present. When $S << K_S$ (Equation 6.3), the system becomes food limited and the specific growth rate is controlled by K_S (Equation 6.5).

$$r_{g,X_B} = \frac{\mu_{max} S}{K_S\left(1 + \frac{S}{K_S}\right)}$$

CHAPTER 6

$$r_{g,X_B} = \frac{\mu_{max} S}{K_S} \qquad 6.5$$

Equation (6.5) is a zero order reaction, illustrating that the growth rate is independent of biomass presence. When $S = K_S$ (Equation 6.3), the growth rate constant is one-half of the half saturation constant (K_S).

If all food is converted to biomass by bacterial population, then food utilization rate should theoretically be equal to the rate of biomass production. However, a portion of the food is converted into waste products; as such, food utilization rate by bacteria is always greater than the biomass production rate. The interrelation between food utilization rate and biomass production is demonstrated by Equation (6.6).

$$r_{g,X_B} = -Y\frac{dS}{dt} = -Yr_s \qquad 6.6$$

$$\text{and } r_s = -\frac{r_{g,X_B}}{Y} = -\frac{\mu_{max}\, S}{Y(S+K_S)} X_B \qquad 6.7$$

where Y = decimal fraction of food mass converted to biomass

$$= \frac{\text{mg/L biomass}}{\text{mg/L food utilized}}$$

$$r_s = \frac{dS}{dt} = \text{rate of food utilization, mg/L.d}$$

Hydrolysis. It is a process which converts larger particulate, dissolved organics and solid molecules into small degradable products, which can be metabolized by bacteria. Since the reaction rate of this process is slower than biological growth process, hydrolysis often becomes the rate limiting step in biological reactors. Hydrolysis process follows first order kinetics, as illustrated in Equations (6.8)-(6.9).

$$r_{g,X_S} = k_h X_S \qquad 6.8$$

where X_S = suspended solid concentration, mg/L

$$r_{g,S} = k_h S \qquad 6.9$$

where S = dissolved organic matter concentration, mg/L
k_h = hydrolysis constant

It should be noted that, the hydrolysis constant (k_h) of Equation (6.8) and (6.9) is not identical.

Decay. Decay process is usually associated with the reduction of the number, and mass of microorganisms. In biological reactors, decay is the degradation of biomass due to oxidation of the biomass organic portion. This process does not change substance amounts; in fact, through decay metabolism slowly degradable organic matter is added to the system which is hydrolyzed further, thereby causing new growth. Decay is described as a first order kinetics, as illustrated in Equation (6.10):

$$r_{g,X_B} = -k_d X_B \qquad 6.10$$

where k_d is the constant for decay, d^{-1}.

Combining Equation (6.3) and (6.10) yields the Monod equation:

$$r_{g,X_B} = \frac{\mu_{max} S}{S + K_S} X_B - k_d X_B \qquad 6.11$$

6.4 Organic Matter Measurement

Biodegradable organics (BOD). These materials consist of organics which can be utilized as food by microorganisms within a specified time period. The degradation of organics in a biological reactor can proceed via two routes: (a) aerobic; and (b) anaerobic degradation. In aerobic degradation, microbiological decomposition occurs in presence of oxygen, and the end products of such decomposition are stable and acceptable. In anaerobic system, degradation proceeds in the absence of oxygen; the end products of anaerobic degradation are unstable and objectionable. This chapter describes aerobic process in details; the mechanisms of anaerobic process have been illustrated in Chapter 9 (sludge management).

The amount of oxygen consumed during organics degradation (by microorganisms) is known as biochemical oxygen demand (BOD). BOD indicates the biodegradable organic content of wastewater. It is often referred as BOD_5, which is measured at the end of 5 days on incubation at 20°C.

The rate at which biodegradable organics are utilized by microorganisms is assumed to be a first order reaction; as such, the rate at which organic materials are utilized is proportional to the amount available (in wastewater). This can be expressed mathematically through Equation (6.12).

$$\frac{dL_t}{dt} = -kL_t \qquad 6.12$$

where, L_t = oxygen equivalent of organics at time t, mg/L
k = reaction constant, d^{-1}

Equation (6.12) can be arranged as shown in Equation (6.13).

$$\frac{dL_t}{L_t} = -kdt \qquad 6.13$$

The integration of Equation (6.13) yields Equation (6.14).

$$\int_{L_0}^{L} \frac{dL_t}{L_t} = -k\int_{0}^{t} dt$$

$$L_t = L_0 e^{-kt} \qquad 6.14$$

where L_0 = total oxygen equivalent of the organics at time zero

L_t = oxygen amount remaining at time t

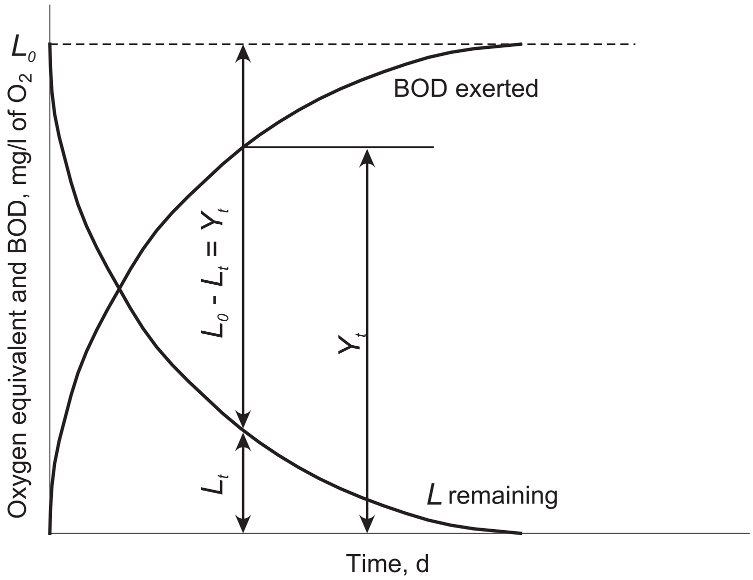

Figure 6.6 **BOD exertion and remaining relationships.**

If L_0 is the oxygen equivalent of the total mass of organics, then the difference between L_0 and L_t is the oxygen equivalent consumed, or BOD exerted (Figure 6.6), defined mathematically through Equation (6.15).

$$y_t = L_0 - L_t \qquad 6.15$$

$$\text{and } y_t = L_0(1 - e^{-kt}) \qquad 6.16$$

where y_t represents BOD_t of the water.

The value of k determines the reaction speed of BOD reaction. The reaction kinetics of easily degradable organics exhibits higher k values, whereas complex organics degradation kinetics demonstrates lower k values. For example, if k_1, k_2 and k_3 are the degradation reaction kinetics of complex, degradable and easily degradable organics, then the interrelation between these three constants can be expressed as $k_3 > k_2 > k_1$ (Figure 6.7). The values of k usually range from 0-1-0.5 d^{-1}, and can be determined by Arrhenius model (Equation 6.17), which is critically dependent on temperature.

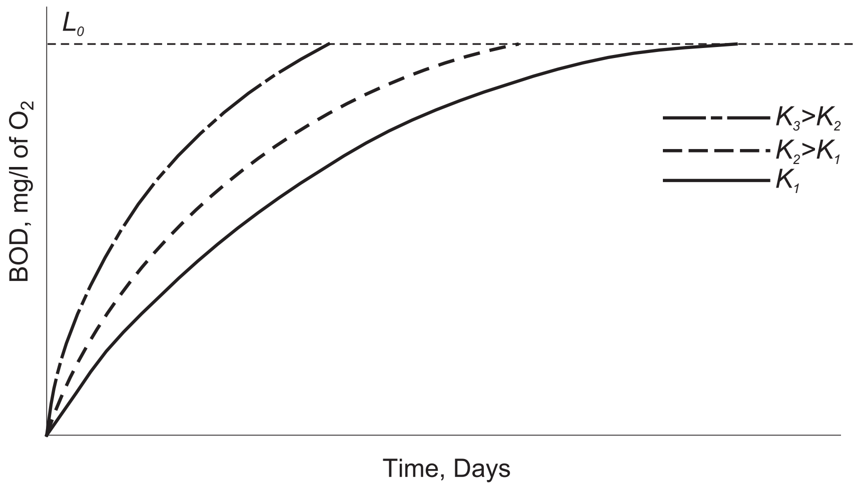

Figure 6.7 **Exertion of BOD as a function of *k*.**

$$k_T = k_{20}\theta^{T-20°} \qquad 6.17$$

where, k_T, k_{20} are BOD rate constants at T °C and 20 °C respectively

$$\theta = 1.047$$

Example 6.1. Determination of BOD in wastewater. The BOD_5 of a wastewater is 200 mg/L at 20°C. k at 20°C is 0.20 per day. Determine the BOD_7 of the sample at 10°C.

Solution

1. Determination of ultimate BOD (L_0)

From Equation (6.16): $L_0 = \dfrac{y_5}{1-e^{-kt}}$

$$= \frac{200}{1-e^{-0.20\times5}} = 316 \text{ mg/L}$$

The ultimate BOD (L_0) is 316 mg/L.

2. Determination of k at 10°C

$$k_T = k_{20}\theta^{T-20°}$$

$$k_{10} = 0.20\,(1.047)^{-10} = 0.12 \text{ d}^{-1}$$

3. Determination of BOD_7 at 10°C

$$y_7 = L_0(1-e^{-kt}) = 316(1-e^{-0.12\times7}) = 180.1 \text{ mg/L}$$

Chemical oxygen demand (COD). Chemical oxygen demand (COD) is defined as the O_2 equivalent of the organic matter, that can be oxidized with a strong chemical oxidizing agent (a mixture of dichromate and sulphuric acid with silver sulphate as catalyst), at an elevated temperature (150°C) for two hours. COD of wastewater is always greater than BOD, as more compounds can be chemically oxidized by biological oxidation.

The COD of wastewater comprises of both biodegradable and non-biodegradable organics. Each of these two forms can be further classified into particulate and dissolved organics. Different physical processes (eg. screening, adsorption) of biological reactors contribute to the removal of both biodegradable and non-biodegradable particulate matter. The biodegradable particulate matter is initially subjected to flocculation, adsorption on biofilm cell wall, followed by hydrolysis of the adsorbed material into soluble organic matter. In contrast, the biodegradable dissolved organics usually pass through the biofilm cell (due to extremely small size), thereby subjected to metabolism. The non-biodegradable dissolved organic portions (of wastewater) leave the process, without being captured. Employing S as a generic symbol of influent organic matter (denoted as COD), the total influent COD concentration (S_{ti}) can be defined as:

$$S_{ti} = S_{bi} + S_{ni}$$

$$= S_{bsi} + S_{bpi} + S_{nsi} + S_{npi} \qquad 6.18$$

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where S_{ti} = influent COD concentration

S_{bi} = biodegradable influent COD concentration

S_{ni} = non-biodegradable influent COD concentration

S_{bsi} = biodegradable dissolved influent COD concentration

S_{bpi} = biodegradable particulate influent COD concentration

S_{npi} = non-biodegradable particulate influent COD concentration

S_{nsi} = non-biodegradable dissolved influent COD concentration

Figure 6.8 illustrates a diagram of influent COD fractions usually present in wastewater.

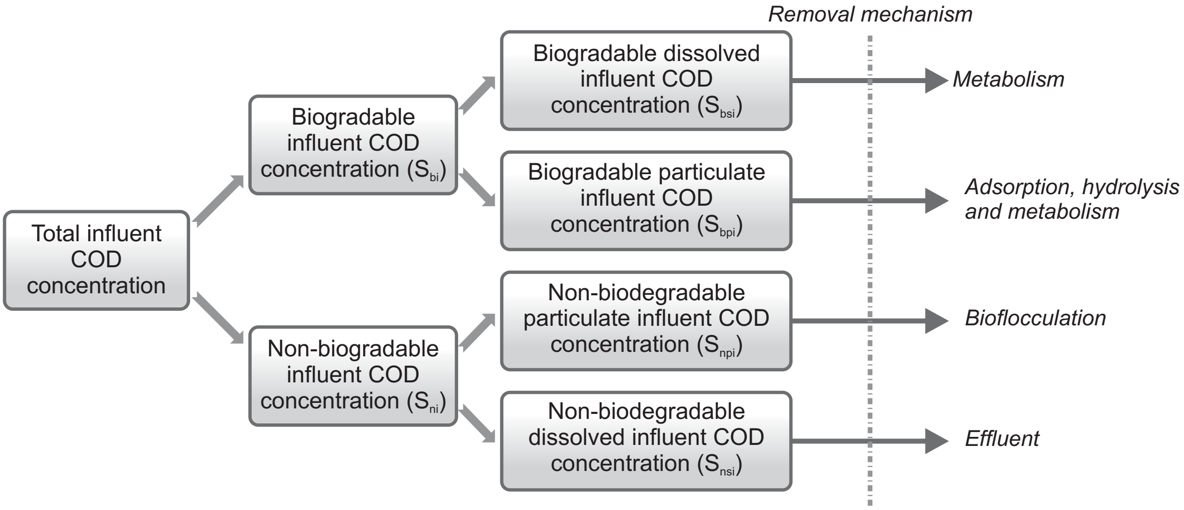

Figure 6.8 **Influent COD characterization according to composition fractions in wastewater.**

COD values of wastewater are always greater than BOD values, as more compounds can be oxidized by the strong chemical agent. Typical ratio of BOD/COD for untreated domestic wastewater is in the range of 0.3-0.8; a BOD/COD ratio of 0.5 or greater indicates that the wastewater is easily degradable, while the ratio below 0.3 indicates that the wastewater is difficult to degrade by microorganisms.

6.5 Factors Effecting Aerobic Degradation of Organics

The important factors that influence aerobic degradation of organics are: (a) temperature; (b) oxygen; (c) pH; and (d) nitrogen and phosphorus.

Temperature. The dependency of organic reaction kinetics on temperature (in biological process) can be illustrated through Equation (6.19):

$$\mu_{max}(T) = \mu_{max\,(20^\circ C)} e^{(k(T-20))} \qquad 6.19$$

This expression is applicable within temperature range 0-32°C, in aerobic process, and becomes constant at temperature range 32-40°C, and then shifted towards zero at 45°C.

Oxygen. The dependency of aerobic organic degradation on the availability of oxygen can be defined through Monod equation (Equation 6.20):

$$\mu = \frac{\mu_{max}\, S_{O_2}}{S_{O_2} + K_{S,O_2}} \quad 6.20$$

where S_{O_2} = oxygen concentration in the reactor, mg/L

K_{S,O_2} = half saturation oxygen constant, mg/L

Combining Equation (6.20) with classical Monod Equation (Equation 6.2) yields double Monod expression, illustrating the dependency of aerobic organics degradation on the availability of oxygen and organics (in wastewater), as expressed in Equation (6.21):

$$\mu = \frac{\mu_{max}\, S_{O_2}}{S_{O_2} + K_{S,O_2}} \frac{S}{S + K_S} \quad 6.21$$

pH. The influence of pH on aerobic organic degradation is defined by Equation (6.22):

$$\mu\,(pH) = \mu_{max\,(opt\,pH)} \frac{K_{pH}}{K_{pH} + I} \quad 6.22$$

where K_{pH} is pH constant

and $I = 10\,[\text{optimum pH} - \text{pH}]$

Alkalinity consumption due to nitrification, or chemical precipitation of phosphorus often results in lower pH values, which can cause major problems in biological treatment processes.

Nitrogen and Phosphorus. Lower nitrogen and phosphorus concentration in wastewater can inhibit the growth of aerobic biofilms, due to lack of nutrient requirement.

The typical constants of the kinetic equations (as discussed previously) have been illustrated in Table 6.3.

Table 6.3 Reaction rate constants for aerobic conversions at 20°C[a].

	Symbol	Unit	Quantity
Maximum specific growth rate	μ_{max}	d^{-1}	4-8
Substrate saturation constant	$K_{S,COD}$	$gCOD/m^3$	5-30
Oxygen saturation constant	$K_{S,O2}$	g_{O2}/m^3	0.5-1
Maximum yield constant	$Y_{max,\,H}$	gCOD/gCOD	0.5-0.7
pH constant	K_{pH}		150-250

[a] *after* Henze et al., (2002).

6.6 Classification of Biological Reactors

The biological reactors to accomplish secondary treatment metabolism can be classified into suspended growth and attached growth process.

Suspended growth: Microorganisms responsible for the conservation of organic matter to gas and cell tissue are remained in suspension within the liquid.

Attached growth: Organisms responsible for conversion of organic matter are attached to a medium.

6.7 Suspended Growth System: Activated Sludge Process

The activated sludge process consists of an aeration tank, followed by a secondary clarifier. The aeration tank (Figure 6.9a) is fed with a mixture of influent and sludge, referred as mixed liquor suspended solid (MLSS) or mixed liquor volatile suspended solid (MLVSS). The MLVSS is the volatile portion (eg. microorganisms) of the MLSS, ranging between 0.75-0.85 of MLSS. The aerators in the tank provide necessary turbulence for keeping the sludge flocs in suspension; in addition, the aerators of such tank also provide necessary oxygen for the oxidation of organic matter. The oxidized MLSS is then transferred to the following secondary clarifier, where solids are separated from liquid phase. The separated liquid (i.e. supernatant) is discharged, referred as effluent whereas a portion of the settled sludge (known as returned sludge) containing microorganisms, is again transferred to the aeration tank. The return of this sludge portion is critically important for maintaining the pollutant removal performances of the aeration tanks, as it increases biomass availability, and accelerate associated kinetic reactions.

The microorganisms in the activated sludge process comprise of filamentous and unicellular bacteria; predominant bacteria include *Pseudomonas, Bacillus, Flavobacterium* and *Alcaligenes*. The microorganisms in MLSS are composed of 70-90% organic and 10-30% inorganic matter (Okun 1949).

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The mass balance of organics removal across an activated sludge system can be calculated assuming steady state conditions; the assumption defines no accumulation of influent organic material inside the system. As such, the daily mass of applied influent organic load (into the system) is the sum of the following fluxes (Figure 6.9b): (a) influent organic matter, not removed in the process escapes as effluent organic material (MS_{te}); (b) transformation of a portion of organic material to organic sludge, and being discharged as excess sludge (MS_{xv}); and (c) oxidation of a portion of the organic matter (MS_0). The routes that transform organic matter are: (a) transformation of organic sludge by anabolism, decay, flocculation and adsorption; and (b) oxidation into inorganic products.

(a)

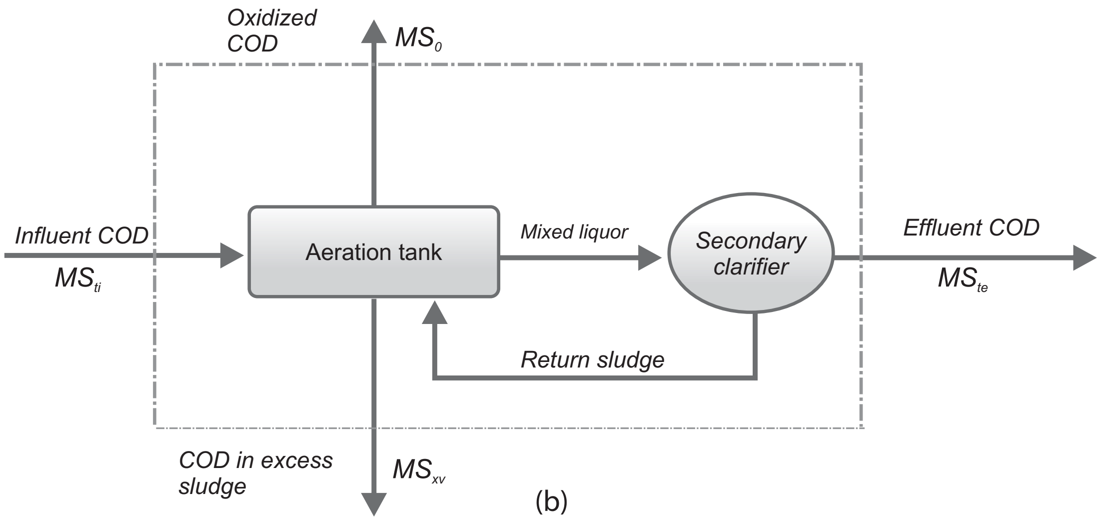

Figure 6.9 (a) Pictorial view of an aeration tank; and (b) organic flow diagram (expressed as COD) across activated sludge system.

The sludge mixing pattern in the aeration tank (of activated sludge process) follows either a completely mixed or a plug flow approach.

Complete mix with recycle. In a complete mix reactor or continuous flow stirred tank reactor (CSTR), the flow is continuously stirred to enhance complete mixing in the reactor. The substrate concentration in a CSTR reactor is same as the effluent substrate concentration. The hydraulic retention time in a CSTR reactor is defined according to Equation (6.23).

$$\theta = \frac{V}{Q} \qquad 6.23$$

where θ =hydraulic retention time, d
V =tank volume, m^3
Q =influent wastewater flow, m^3/d

The sludge age (θ_C) is a crucial factor for designing activated sludge process, which can be defined as the ratio of the organism's mass in the aeration tank, and the mass removed from the system per day. Such definition of sludge age can be expressed through Equation (6.24) (for sludge wasting from aeration tank-Figure 6.10a), and Equation (6.25) (for sludge wasting from return sludge line-Figure 6.10b).

$$\theta_c = \frac{VX}{Q_{wa}\,X + Q_e X_e} \qquad 6.24$$

$$\theta_c = \frac{VX}{Q_{wr} X_r + Q_e X_e} \qquad 6.25$$

Where, θ_C = mean cell residence time, d
X = concentration of MLVSS maintained in the tank, mg/L
Q_{wa}, Q_e = waste sludge flow removed from aeration tank and flow of treated effluent, m^3/d

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X_e = microorganism concentration in effluent (VSS), mg/L
Q_{wr} = waste sludge flow from return sludge line, m^3/d
X_r = microorganism concentration in return sludge line, mg/L

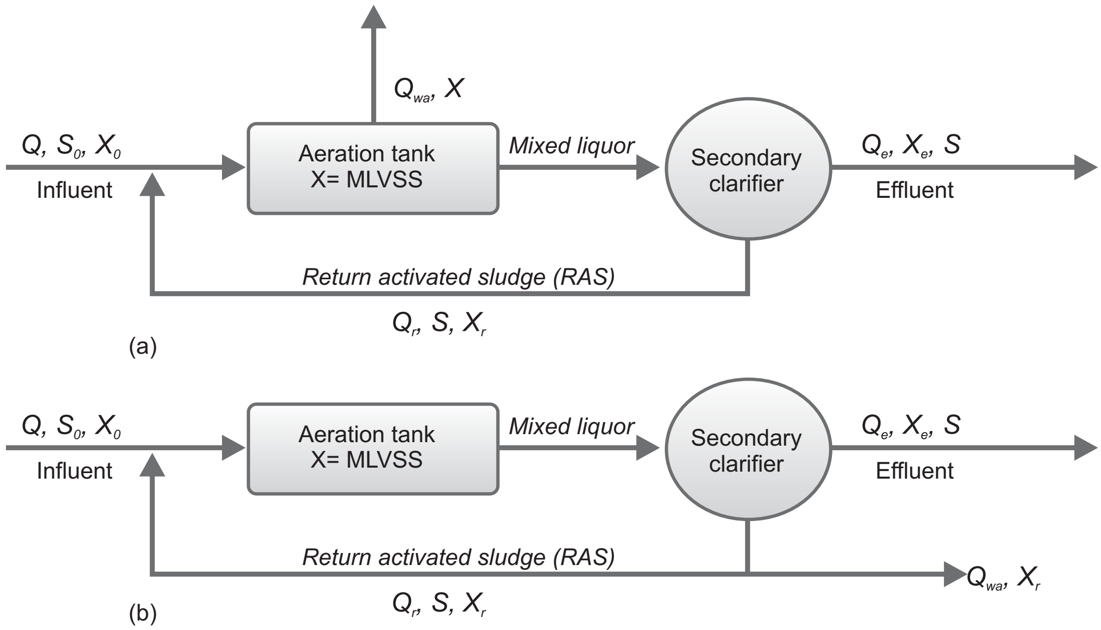

Figure 6.10 **Schematic diagram of a completely mixed reactor with: (a) wasting of sludge from aeration tank; and (b) wasting from return sludge line.**

CHAPTER 6

Biomass and substrate mass balance. The mass balance in the entire activated sludge process can be defined as the rate of accumulation of microorganisms in the inflow plus net growth, minus that in the outflow, as defined in Equation (6.26):

$$V\frac{dX}{dt} = QX_0 + V(r'_g) - (Q_{wa}X + Q_eX_e) \qquad 6.26$$

where, V= aeration tank volume, m^3
dX/dt = rate of change of microorganism concentration (VSS), mg/(L.m^3.d)
X_0 = microorganism concentration (VSS) in influent, mg/L
X = microorganism concentration in tank, mg/L
r'_g = net rate of micro-organism growth (VSS), mg/(L.d)

Bacterial net growth rate (Equation 6.26) growth can be expressed according to Equation (6.27), after replacing the right term of Equation (6.11) with Equation (6.7):

$$r'_g = -Yr_s - k_dX \qquad 6.27$$

where Y = maximum yield coefficient, mg/mg
r_s = substrate utilization rate, mg/m^3.d
k_d = endogenous decay coefficient, d^{-1}

Table 6.4 denotes the values of yield coefficient and endogenous decay coefficient for activated sludge process.

Table 6.4 **Yield and endogenous decay coefficients.**[a]

Coefficient	Unit	Range	Typical
Y	g VSS/g BOD_5	0.4-0.84	0.6
Y	g VSS/g COD	0.24-0.4	0.4
k_d	d^{-1}	0.004-0.10	0.06
k	d^{-1}	11 - 20	5

[a] After Lawrence and McCarty (1970), Metcalf and Eddy (1991), and WEF and ASCE (1992)

Substituting Equation (6.27) into Equation (6.26), and assuming: (a) the cell concentration (X_0) in the influent is zero; and (b) steady-state conditions, results in equation (6.28):

$$\frac{Q_{wa}X + Q_e X_e}{VX} = -Y\frac{r_s}{X} - k_d \qquad 6.28$$

Since the left hand side of equation (6.28) is the inverse of mean cell residence time θ_C (Equation 6.24), Equation (6.28) can be expressed as:

$$\frac{1}{\theta_c} = -Y\frac{r_s}{X} - k_d \qquad 6.29$$

Substrate utilization rate r_s can be computed from Equation (6.30):

$$r_s = \frac{Q}{V}(S_0 - S) = \frac{S_0 - S}{\theta} \qquad 6.30$$

where, S_0 = influent substrate concentration, mg/L
S = effluent substrate concentration, mg/L
θ = hydraulic retention time, d

The mass concentration of microorganisms (i.e. biomass) X in the aeration tank can be derived by substituting Equation (6.30) into equation (6.29):

$$X = \frac{\theta_c Y(S_0 - S)}{\theta(1 + k_d\theta_c)} \qquad 6.31$$

Substituting hydraulic retention time $\theta = V/Q$ in equation (6.31) for solving volume of reactor (V), yields equation (6.32):

$$V = \frac{\theta_c Q Y(S_0 - S)}{X(1 + k_d\theta_c)} \qquad 6.32$$

The substrate concentration (S) in the effluent can also be determined from the substrate mass balance as demonstrated in Equation (6.33):

$$S = \frac{K_s(1 + \theta_c k_d)}{\theta_c(Yk - k_d) - 1} \qquad 6.33$$

Where k can be defined as maximum rate of substrate utilization per unit mass of microorganism, per day.

Substituting the value of X given in Equation (6.31) for r'_g in Equation (6.27), and dividing by the term S_0-S, the observed yield in the system (with recycle) is:

$$Y_{obs} = \frac{Y}{1 + k_d \theta_c} \qquad 6.34$$

where, Y_{obs} = observed yield in the system with recycle, mg/mg

The prediction of effluent organics concentration, employing Equation (6.31) and (6.33) is difficult, due to presence of substantial parameters. As such, the correlation between substrate utilization rate, mean cell residence time, and food to microorganism ratio is often used for the design of activated sludge system. The term r_s/X in equation (6.29) is defined as specific substrate utilization rate, U (Equation 6.35).

$$U = -\frac{r_s}{X} \qquad 6.35$$

Applying the expression of r_s (Equation 6.30) in Equation (6.35), the specific substrate utilization rate can be computed:

$$U = \frac{Q(S_o - S)}{VX} = \frac{S_o - S}{\theta X} \qquad 6.36$$

Substituting the term U in Equation (6.29) yields the following equation:

$$\frac{1}{\theta_c} = YU - k_d \qquad 6.37$$

In completely mixed activated sludge process the VSS in the effluent (X_e) is very small, thereby can be neglected. As such, Equation (6.24) can be written as:

$$\theta_c = \frac{VX}{Q_{wa} X}$$

$$\text{or, } Q_{wa} = \frac{V}{\theta_c} \qquad 6.38$$

Similarly, Equation (6.25) can be expressed as:

$$Q_{wr} = \frac{VX}{\theta_c X_r} \qquad 6.39$$

The term food-to microorganism ratio (F/M) is extensively employed for activated sludge process design. The F/M ratio (d^{-1}) is denoted as the ratio of influent soluble BOD_5 concentration (S_0), to the product of hydraulic retention time (θ) and MLVSS concentration (X) in the aeration tank (Equation 6.40).

$$\text{F/M} = \frac{S_0}{\theta X} = \frac{QS_0}{VX} \qquad 6.40$$

F/M and U are interrelated, which can be implemented for the computation of the efficiency (E) of the activated-sludge process, defined in Equation (6.41):

$$U = \frac{(F/M)E}{100} \qquad 6.41$$

$$\text{and, } E = \frac{S_0 - S}{S_0} \times 100 \qquad 6.42$$

where S_0 = influent substrate concentration, mg/L
S = effluent substrate concentration, mg/L

Sludge production. The amount of sludge production (from activated sludge process) per day can be calculated by equation (6.43).

$$P_x = \frac{Y_{obs} Q (S_0 - S)}{1000 \text{ g/kg}} \qquad 6.43$$

Where, P_x = waste activated sludge (VSS), kg/d
Y_{obs} = observed yield, g/g

Oxygen requirement. The oxygen requirement in the activated sludge (for BOD removal) is determined from Equation (6.44).

$$\text{kg } O_2/\text{d} = \frac{Q (S_0 - S)}{(1000 \text{ g/kg}) f} - 1.42 P_x \qquad 6.44$$

The normal air requirements for most of the activated sludge systems are 1.1 kg O_2 (93.5 m^3 of air) per kg BOD_5. In general, air requirements vary between 3.75-15 m^3 air/m^3 water (Metcalf and Eddy, 1991). Table 6.5 describes the air requirement in activated sludge processes, based on F/M ratio.

Table 6.5 Air requirements and F/M ratio correlationship.[a]

F/M ratio	Air requirements
>0.3d^{-1}	31-56 m^3/kg BOD_5 removal
<0.3d^{-1}	75-112m^3/kg BOD_5 removal

[a] *After* Lin (2007).

Recycle rate. The recycle rate (R) can be defined as the ratio of return sludge and raw wastewater (Figure 6.11), and can be expressed through Equation (6.45).

$$R = \frac{Q_4}{Q_1} \qquad 6.45$$

where R = recycle rate
Q_1 = influent flow rate, m^3/d
Q_4 = flow rate of return sludge, m^3/d

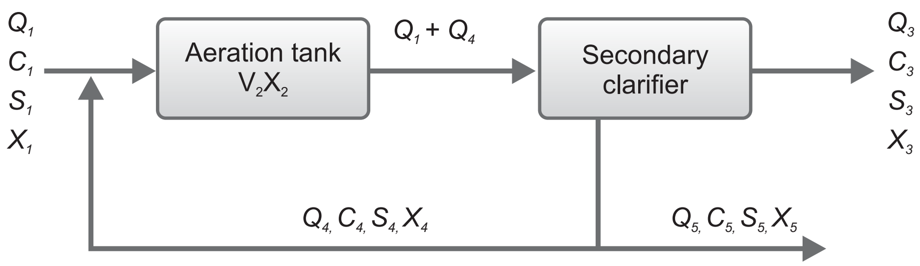

Figure 6.11 Activated sludge process with recycle.

Volumetric loading. The volumetric loading (kg BOD/m^3.d), which is often employed for the design of activated sludge plants can be expressed by Equation (6.46).

$$V_L = \frac{Q_1 C_1}{V_2} \quad 6.46$$

where C_1 = input BOD load rate, kg BOD/m^3
V_2 = aeration tank volume, m^3
V_L = volumetric loading, kg BOD/m^3.d

6.8 Plug- Flow Pattern: Activated Sludge Process

In plug flow (PF) regime, it is assumed that complete mixing occurs in the transverse plane, but minimal mixing takes place towards the flow direction. Typically, long and narrow aeration tanks are employed (Figure 6.12a) to provide such mixing regime.

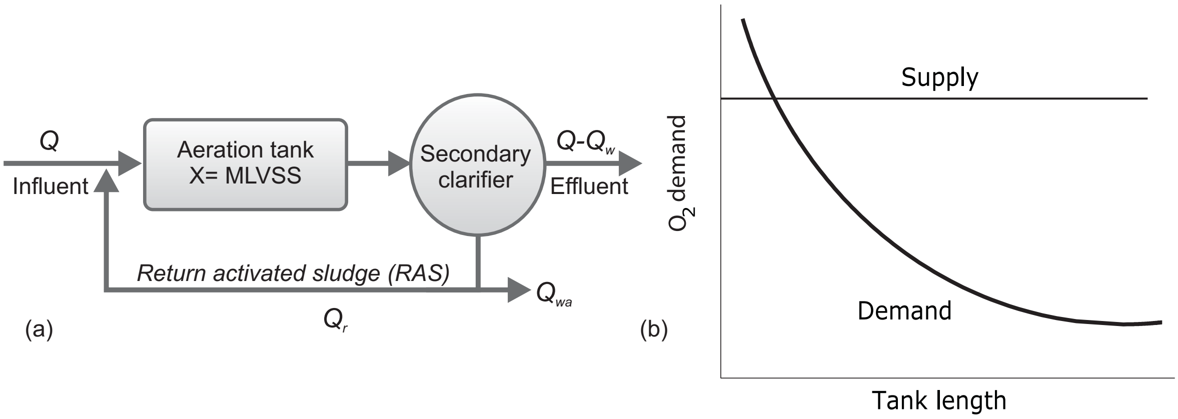

Figure 6.12 (a) Long aeration tank to achieve plug flow regime; and (b) oxygen concentration profile in a PF tank.

In a PF system, the oxygen demand is higher at the influent end (of the basin), due to intensive degradation of readily degradable substrates (Figure 6.12b). At the effluent end of the basin oxygen demand is lower, which can be attributed to lower rate of endogenous metabolism coupled with absence of readily degradable substrates.

The widely accepted kinetic model for a PF reactor was developed by Lawrence and McCarty (1969); the expressions derived (by the authors) for average MLSS ($\bar{X}$) and food utilization are illustrated in Equations (6.47)-(6.48). It should be noted that these equations are valid only when $\theta_c/\theta \geq 5$.

$$\bar{X} = \frac{\theta_c Y (S_0 - S)}{\theta (1 + k_d \theta_c)} \quad 6.47$$

$$r_s = \frac{kSX}{K_s + S} \quad 6.48$$

Integrating Equation (6.47) over detention time in reactor and substituting the appropriate boundary conditions, and recycle factor, yields Equation (6.49).

$$\frac{1}{\theta_c} = \frac{K_0(S_0 - S)}{(S_0 - S) + (1 - \infty)\left(K_s In \frac{S_i}{S}\right)} - k_d \quad 6.49$$

where K_0 = maximum growth rate constant, d^{-1}

Q_r = recycled flow rate, m^3/d

S_i = substrate concentration after mixing with recycled sludge, mg/L

$$S_i = \frac{S_o + \sigma S}{1 + \sigma}$$

σ = recycle factor, $\frac{Q}{Q_r}$

6.9 Secondary Clarifiers

The secondary clarifiers of activated sludge processes fulfill two objectives: (a) producing effluent that fulfills discharge criteria; and (b) concentrating biological solids to minimize sludge quantity that has to be treated. As such, proper designing of secondary clarifiers are very critical for enhancing the performances of activated sludge process.

The settling phenomenon of solids, and various settling zones observed in secondary clarifiers can be described by column batch analyses. If a column (in column batch analyses) is filled with concentrated suspension and allowed to settle, the contents will be divided into different zones, as observed in Figure 6.13a. In zone B, initial concentration C_0 is preserved and settles at a uniform velocity, resulting clarified zone (A zone); the clarified zone is lengthened at the same velocity.

Below the uniform velocity zone, two other zones develop. Particles which reach at the bottom gain resting position; particles fall on them form a compression zone, referred as zone D (Figure 6.13a) which is supported from below. The intermediate zone between D and B contains a concentration gradient slightly higher than C_0 just below zone B, and slightly less than the concentration at the top of compression zone (D zone). Such intermediate zone (between D and B) can be defined as thickening zone.

As time progresses, relative movement occurs between the zone interfaces. C-D interface moves upward, due to drop of particles from zone C into zone D. Such drop of particle results upward displacement of B-C interface at the same velocity of C-D interface; this condition continues as long as velocity gradient of C zone remains constant, resulting constant width of this zone (C zone). However, A-B interface also moves downward due to uniform settling of particles (with initial concentration). The simultaneous downward movement of A-B interface, and upward displacement of C-D interface erodes particles of zone B at top to bottom, until it becomes nonexistent ($t = t_3$). After this time the newly formed A-C interface settles at a decreasing rate, as the interfacial solids concentration increases from C_0 (just at the disappearance of zone B), to the concentration of the top layer of the compression zone just as zone C also disappears ($t = t_5$). The newly formed A-D interface will subside at a slow uniform rate; the solids consolidate under their own weight, releasing some of interstitial water to the clarified zone located above.

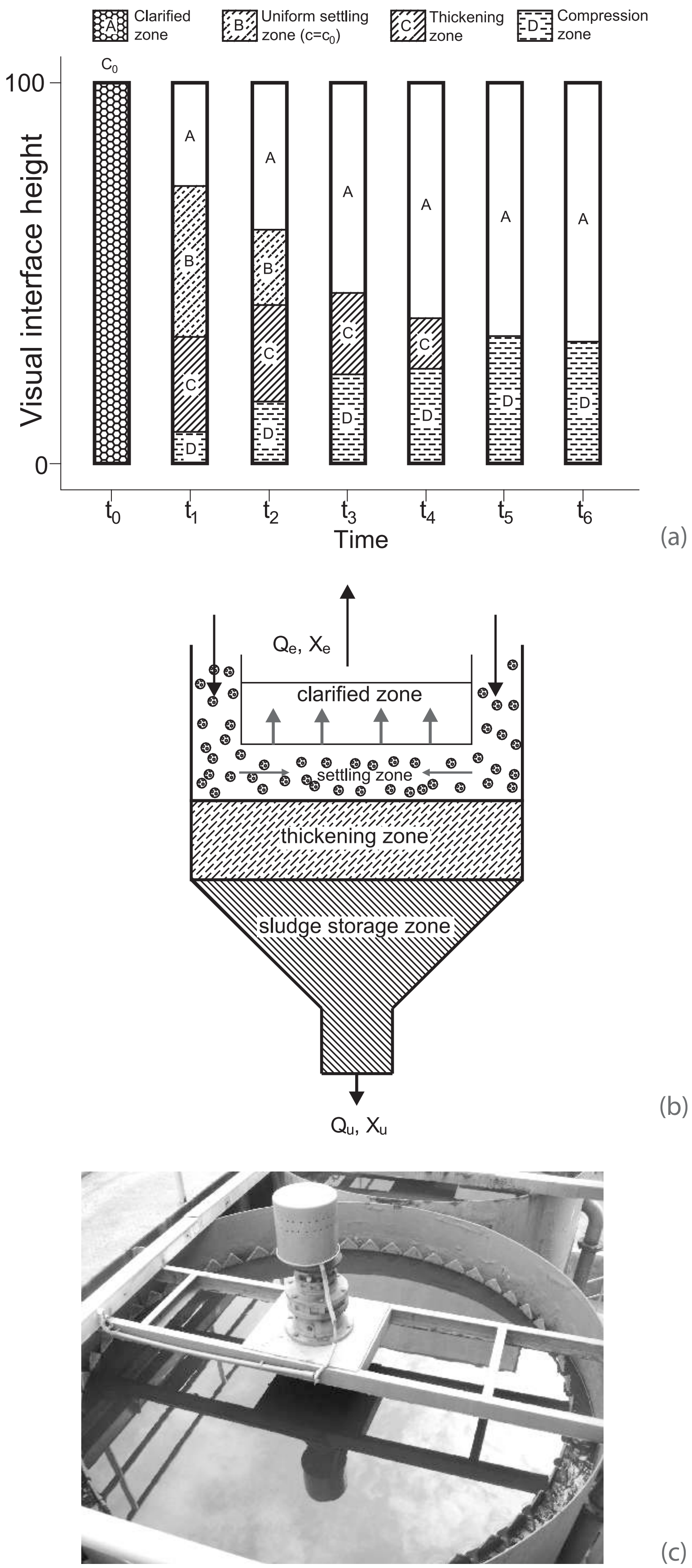

Figure 6.13 **(a) Different settling zones observed in batch column analyses; (b) settling zones of secondary clarifiers; and (c) pictorial view of a secondary clarifier.**

The settling phenomena, as observed in column batch analyses are also applicable for continuous flow secondary clarifiers. If steady-state conditions are imposed on flow rate and solids concentration for both influent and underflow, all the zones will be in static positions (Figure 6.13b). Due to this assumption, the A-B interface will be stationary, resulting in rising of water (in the clarified zone) towards overflow rate at a rate of settling velocity of C_0 concentration.

The zone settling principles for continuous flow secondary clarifier are same as batch analysis. The thickening function is accomplished by concentration gradient in thickening and compression zone. Solid flux method can be used to determine thickening function. The downward velocity of solids in secondary clarifier has two components:

- transport velocity due to sludge withdraw
- gravity settling of solid relative to water

The transport velocity (v_u) is a function of underflow rate (Q_u) and tank area (A), illustrated in Equation (6.50):

$$v_u = \frac{Q_u}{A} \tag{6.50}$$

The resulting solid flux (i.e. the product of underflow rate and the solids concentration) can be defined in Equation (6.51):

$$G_u = v_u X_i = \left(\frac{Q_u}{A}\right) X_i \tag{6.51}$$

Here, G_u is solid flux at particular depth where solid concentration is X_i.

The solid flux due to gravity settling is illustrated in Equation (6.52):

$$G_g = v_g X_i \tag{6.52}$$

where v_g = settling velocity of solids at X_i concentration.

The total solid flux in secondary clarifier is the sum of underflow transport and gravity flux (Figure 6.14) as observed in Equation (6.53).

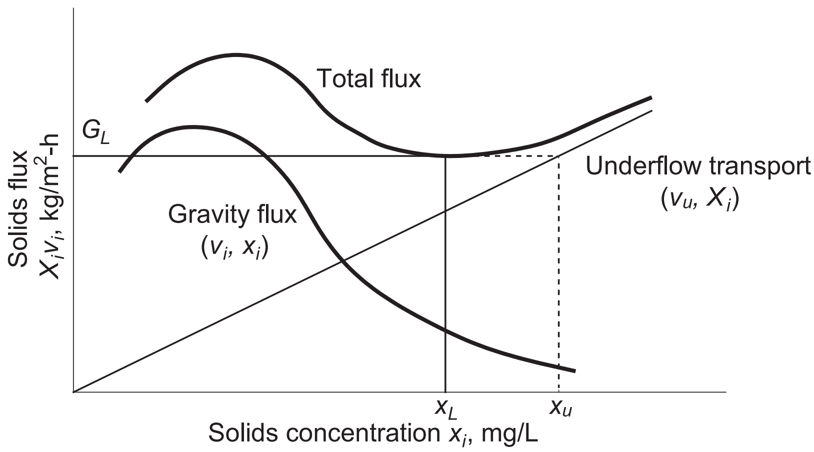

Figure 6.14 Solid flux vs. solids concentration curve.

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$$G = G_u + G_g \tag{6.53}$$

The total solid flux is limited by a minimum value from gravity thickening. For a given underflow rate, the limiting gravity flux also determines underflow concentration X_u.

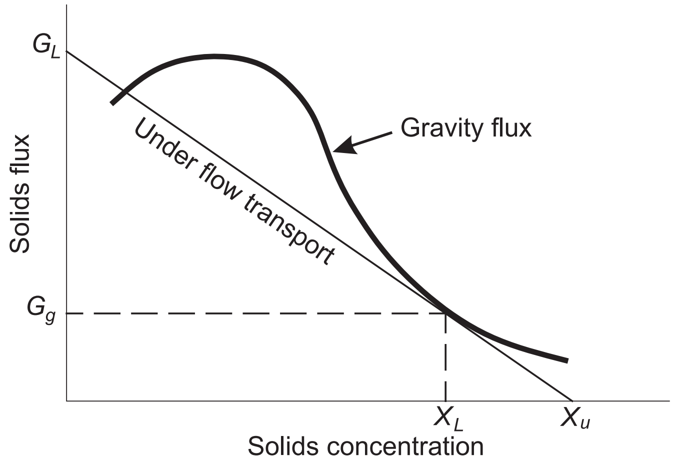

Figure 6.15 **Yoshoika's modification for matching underflow concentrations to limiting flux.**

Yoshioka et al., (1957) employed modification of the above graphical procedures, for matching underflow concentrations to their related limiting flux rates. According to such modification, a line beginning at the desired underflow concentration (X_u) is drawn in tangent to the gravity flux curve (Figure 6.15), which intersects solid flux ordinate at limiting flux. The absolute value of the tangent slope is referred as the underflow velocity; the abscissa value at tangency point is the limiting gravity flux concentration. The ordinate at the tangency point is gravity solid flux (G_g), while the intercept G_L-G_g is the flux due to underflow transport.

Table 6.6 **Guidelines for the design of secondary clarifiers.**

Treatment	Overflow rate $m^3/m^2.d$		Loading $kg/m^2.h$		Depth m
	Avg.	Peak	Avg.	Peak	
Settling following air activated sludge	16 - 32	40 - 48	3 - 6	9	3.5 - 5
Settling following extended aeration	8 - 16	24 - 32	1 - 5	7	3.5-5

The procedures and equations described in the previous paragraphs require extensive experimental analyses. However, when such data sets are not available, the engineer may depend on literature for the design of secondary clarifiers. Table 6.6 presents such data sets for the design of secondary clarifier. However the implementation of these data is critically dependent on the wastewater characteristics.

Operational control of secondary clarifier. The smooth performance of secondary clarifiers (in activated sludge process) is dependent on a set of critical operational tools, such as: sludge volume index, sludge density index, return activated sludge flow rate, sludge settleability, and waste activated sludge.

a. Sludge volume index: The sludge volume index (SVI) is defined as the volume (in mL), that is occupied by 1 g suspension after 30 minutes settling. SVI often has an impact on return sludge, and can be calculated employing Equation (6.54).

$$SVI = \frac{SV \times 1000 \text{ mg/g}}{MLSS} \quad 6.54$$

where SVI = sludge volume index, mL/g
SV = settled sludge volume, mL/L

b. Sludge density index: Sludge density index is also employed for calculating the settleability of sludge in a secondary clarifier. The weight (grams) of 1 mL sludge after 30 minutes settling can be calculated through the following equation:

$$SDI = \frac{100}{SVI} \quad 6.55$$

where SDI = sludge density index, g/mL

c. Return activated sludge and flow rate: Returned activated sludge (RAS) is the settled sludge of the secondary clarifier, that is returned to the aeration tank. RAS is a crucial parameter for proper functioning of the activated sludge process, because it maintains a certain proportion of microbial population in the aeration tank for pollutant removal (from wastewater).The minimum percentage of RAS is related to SVI, and can be expressed according to Equation (6.56):

$$\% \, of \, RAS = \frac{100}{\dfrac{100}{(SVI)P} - 1} \quad 6.56$$

where P = solid percentage of the mixed liquor

The flow rate at which sludge is returned to the aeration tank can be determined by mass balance analyses around aeration tank (Figure 6.10). Assuming no accumulation in the aeration tank, and influent solids concentration in the tank is negligible, the mass inflow is equal to the outflow. Equation (6.57) denotes a typical mass balance analyses around aeration tank.

$$X_r Q_r = X(Q + Q_r) \quad 6.57$$

where X_r = returned activated sludge suspended solids, mg/L
Q_r = RAS flow rate, m^3/s

and $$Q_r = \frac{X}{X_r - X} Q \quad 6.58$$

The sludge settleablity method can also be utilized, to determine the RAS flow rate. Such method requires establishing a relation between influent flow rate, and settled sludge volume defined in Equation (6.59).

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$$Q_r = \frac{(SV)Q}{1000 - SV} \qquad 6.59$$

where SV=settled sludge volume (after 30 minutes), mL/L

d. Waste activated sludge: In activated sludge processes, a portion of the settled sludge (in the secondary clarifier) is returned to the aeration tank. The excess portion of the settled sludge, commonly referred as waste activated sludge (WAS) is discharged to sludge thickeners for treatment and disposal. The objectives of WAS are: (a) to maintain a certain F/M ratio; (b) to remove excess number of microorganisms from the system.

The WAS can be calculated from mass balance analyses around the secondary clarifier. Assuming no net change of sludge blanket in the clarifier, and effluent solids concentration from the tank to be negligible, the mass balance around the clarifier must be equal to the solids entering and leaving the tank. This relationship can be written from Figure 6.10 b.

$$(Q + Q_r)\ MLSS = Q_{wa}(WAS) + Q_r\ (RAS) \qquad 6.60$$

and $$Q_r = \frac{Q\,(MLSS) - Q_{wa}(WAS)}{RAS - MLSS} \qquad 6.61$$

where Q_r= return activated sludge flow, m^3/d

Q_{wa}= waste activated sludge flow, m^3/d

WAS= SS of waste activated sludge, mg/L

RAS= SS of return activated sludge, mg/L

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e. Sludge bulking: The main purpose of secondary clarifiers is to enhance the settling of sludge, thereby producing clear, odorless supernatant (i.e. effluent). However, these objectives are often hindered due to poor sludge settling, and formation of foam, resulting in floating of such materials in the supernatant, followed by escaping (of sludge materials), which in turn increase BOD and solids concentration of the effluent. This phenomenon is referred as sludge bulking.

Sludge bulking is caused by: (a) growth of filamentous (eg. *Fungi, Beggiatoa, Thiotrix*, and *Leucothrix*) and foam forming organisms (eg. *Nocardia amarae*, *Microthrix parvicella*, *N. amarae*, and *N. pinesis*), which donot readily settle in secondary clarifiers; and (b) adverse environmental and operating conditions such as: excessive flow, BOD loading, inadequate aeration, flow short circuiting, lack of nutrients, and presence of toxic materials.

If sludge bulking is observed in secondary clarifiers, it can be mitigated by adjusting the operating conditions (as described in the previous paragraphs). In addition, microscopic examination is also necessary, to detect the presence of filamentous organisms in the clarifier. Chlorination of 2-3 mg/L (of chlorine) per 1000 mg/L of MLVSS is also beneficial (Metcalf and Eddy, Inc., 1991) for eliminating sludge bulking. Applying hydroperoxide oxidant or pH adjustment can also improve sludge bulking condition.

Example 6.2. Operational parameter calculation. The influent flow rate (Q) of a return activated sludge process is 40,000 m^3/d, MLSS (X) in aeration tank is 3000 mg/L, and settling sludge volume (SV) in 30 min is 250 mL in a secondary clarifier. Compute: (1) sludge volume index; (2) return flow ratio and rate; and (3) suspended solid (SS) concentration in return activated sludge.

Solution

1. Sludge volume index (SVI)

Employing Equation (6.54);

$$SVI = \frac{SV \times 1000 \text{ mg/g}}{MLSS}$$

$$= \frac{250 \text{ mL/L} \times 1000 \text{ mg/g}}{3000 \text{ mg/L}} = 84 \text{ mL/g}$$

SVI is in the typical range of 80-150 mL/g for MLSS concentration of 2000-3500 mg/L.

2. Computing return flow ratio (Q_r/Q) and rate (Q_r)

From Equation (6.59):

$$Q_r = \frac{(SV)\,Q}{1000 - SV}$$

Return flow ratio, $\frac{Q_r}{Q} = \frac{250}{1000 - 250} = 0.33$

Return flow rate, $Q_r = 0.33\,Q = 13200 \text{ m}^3\text{/d}$

3. Suspended solid (SS) concentration (Xr) in return activated sludge

Employing Equation (6.57):

SS concentration, $X_r = \frac{X(Q + Q_r)}{Q_r}$

$$= \frac{3000 \text{ mg/L} \times (40000 + 13200) \text{ m}^3\text{/d}}{13200 \text{ m}^3\text{/d}} = 12090 \text{ mg/L}$$

6.10 Aeration in Activated Sludge Process

Aeration theory. The aeration theory is based on Henry's law; at equilibrium conditions, the partial pressure of a component in the gas phase is proportional of this component dissolved in the liquid phase. For aeration, the liquid phase is the mixed liquor, and the gas phase is air; the component is the oxygen concentration. So at equilibrium condition:

$$DO_s = k_H P_{O_2} \qquad 6.62$$

where, DO_S = dissolved oxygen saturation concentration in mixed liquor, mg/L
k_H = Henry's constant
P_{O_2} = partial pressure of oxygen in air

In biological treatment units, DO concentration in the mixed liquor is less than the saturation value, due to oxygen consumption by the microbes. As such, under normal conditions the atmospheric oxygen is transferred to the mixed liquor. According to Fick's law, this transfer rate is proportional to the difference between saturation and actual DO concentration in the mixed liquor, as illustrated in Equation (6.63):

$$\frac{dDO_l}{dt} = k_{la}(DO_s - DO_l) \qquad 6.63$$

where, dDO_l/dt = transfer rate of atmospheric oxygen, mg O_2.L^{-1}.h^{-1}
DO_s = saturation dissolved oxygen concentration in the mixed liquor, mg O_2/L
DO_l = dissolved oxygen concentration in the mixed liquor, mg O_2/L
k_{la} = oxygen transfer coefficient, h^{-1}

The value of transfer coefficient depends on aeration system, reactor geometry, operational conditions, and mixed liquor impurities. As such, the concept of oxygenation capacity is often used for designing aeration systems in biological units. The oxygenation capacity of an aerator is the maximum oxygen transfer rate, under standard operating conditions. The oxygen transfer is calculated in pure water without oxygen ($DO_l = 0$) under atmospheric pressure (760 mm Hg), at a temperature 20° C. Considering these factors, the oxygenation capacity (OC, mg O_2.L^{-1}.h^{-1}) can be measured employing Equation (6.64).

$$OC = \left(\frac{dDO_l}{dt}\right)_{max} = k_{la} DO_s \qquad 6.64$$

Aerator types. The objective of aeration in activated sludge process can be enlisted as: (a) to oxidize organic matter, NH_4-N, and H_2S; and (b) providing adequate turbulence to maintain sludge flocs suspension. Such objectives (of aeration) in biological units are achieved by aerators, which can be classified into two groups: (a) diffused air systems, where air bubbles are introduced from the bottom part of the liquid phase, following upward direction; and (b) mechanical aerators, where air bubbles are introduced in the liquid phase.

The diffused aerators can be classified as:

- Fine bubble aeration: composed of porous ceramic domes or discs mounted on aeration tank bottom; the oxygen transfer efficiency is high.
- Coarse bubble aeration: comprises of non porous domes, discs or tubes, producing larger bubbles; oxygen transfer efficiency is lower.

The mechanical aerators can be distinguished as:

- Horizontal shaft aerators: mounted on fixed platforms; each surface aerator has its own motor. The rotating speed of such aerators is 20-60 rpm. Examples include brush and disk aerators.
- Vertical shaft aerators: a propeller agitates the water, introducing air bubbles into the mixed liquor, and suspending liquid droplets in the air. The motor can be directly coupled to the propeller, which in turn increases the rotation speed. A gear box is sometimes used to achieve lower rotation speed of the propeller.

Oxygenation capacity of surface aerators. Using Equation (6.64), the oxygen transfer capacity under non-standard conditions can be related to transfer capacity under standard conditions, as demonstrated in Equation (6.65):

$$\frac{OC_a}{OC_s} = \frac{k_{laa}\,(D0_{sa} - DO_l)}{k_{las} DO_{ss}} \tag{6.65}$$

where OC_a = actual oxygenation capacity
OC_s = oxygenation capacity under standard conditions
DO_{ss} = saturation DO concentration at 20°C = 9.1 mg/L
DO_{sa} = saturation DO concentration under actual conditions
DO_l = saturation DO concentration under actual operational conditions
l_{aa} = transfer constant

The values of k_{laa} and DO_{sa} can be expressed as:

$$k_{laa} = \alpha k_{las}\,\theta^{T-20} \tag{6.66}$$

$$DO_{sa} = \frac{(p - p_w)\,51.6\beta DO_{ss}}{(p_s - p_w)\,(31.6 + T)} \tag{6.67}$$

where T = temperature in °C
θ = temperature dependency factor
=1.020-1.028 for diffused air systems, and 1.012 for mechanical aeration (Landberg et al., 1969)
p = actual atmospheric pressure, mm Hg
p_w = water vapour pressure, mm Hg
p_s = standard pressure,760 mm Hg
α, β = constants

Putting the values of Equations (6.66) and (6.67) into Equation (6.65) results:

$$\frac{OC_a}{OC_s} = \alpha\theta^{T-20}\left[\frac{(p - p_w)\,51.6\beta DO_{ss}}{(p_s - p_w)\,(31.6 + T)} - DO_l\right] / DO_{ss} \tag{6.68}$$

The power requirement for surface aerators is expressed by Equation (6.69):

$$P_m = \frac{MO_t}{OC_a} \tag{6.69}$$

where P_m = required motor power, kW
OC_a = actual oxygen transfer capacity, $kgO_2.kWh^{-1}$
MO_t = oxygen consumption, $kgO_2.h^{-1}$

Example 6.3. Calculation of aerator power. A mechanical aerator has an oxygenation capacity of 3 kg $O_2.kWh^{-1}$ under standard conditions. What will be the oxygenation capacity at an altitude 1000 m (673 mm Hg pressure) in winter (at T = 10°C, vapour pressure 9.1 mm Hg)? Also calculate the required motor power, assuming daily oxygen consumption (MO_t) is equal to 3000 kg $O_2.d^{-1}$. Assume, $\alpha = 0.8$, $\theta = 1.012$, $\beta = 0.9$ and $DO_l = 2$ mg/L.

Solution

At 10 °C: Employing Equation (6.68):

$$\frac{OC_a}{OC_s} = \alpha\theta^{T-20}\left[\frac{(p-p_w)\,51.6\beta DO_{ss}}{(p_s-p_w)\,(31.6+T)} - DO_l\right] / DO_{ss}$$

$$= 0.8\times(1.012)^{10-20}\times\left[\frac{(673-9.1)\text{ mm }Hg\times 51.6\times 0.9\times 9.1}{(760-9.1)\text{ mm }Hg\times(31.6+10)°C} - 2\text{mg/L}\right]/9.1\text{ mg/L}$$

$$= 0.54$$

From Equation (6.69):

$$\text{Power requirement, } P_m = \frac{3000}{24\left(OC_s\left(\frac{OC_a}{OC_s}\right)\right)} = \frac{3000}{24\times 3\times 0.54} = 77\text{ kW}$$

6.11 Common Variations of Activated Sludge Process

To improve the organics removal efficacy, the plug flow and completely mixed systems had been modified over the past years. Such modification was primarily based on F/M ratio, BOD_5 loading and hydraulic retention time (HRT). The modified processes include step aeration, tapered aeration, contact stabilization, pure oxygen activated sludge system, Kraus process, two sludge system, extended aeration and sequential batch reactors. This section provides a brief operational description of such modified processes.

Step aeration. The step aeration process distributes waste flow to a number of points along the basin (Figure 6.16), thus avoiding locally high O_2 demand; the return activated sludge (RAS) is introduced at the aeration tank head. The flow distribution lessens the effect of peak hydraulic and organic loads, and may provide sufficient dilution to protect bacteria against toxic materials. Step aeration follows plug flow regime.

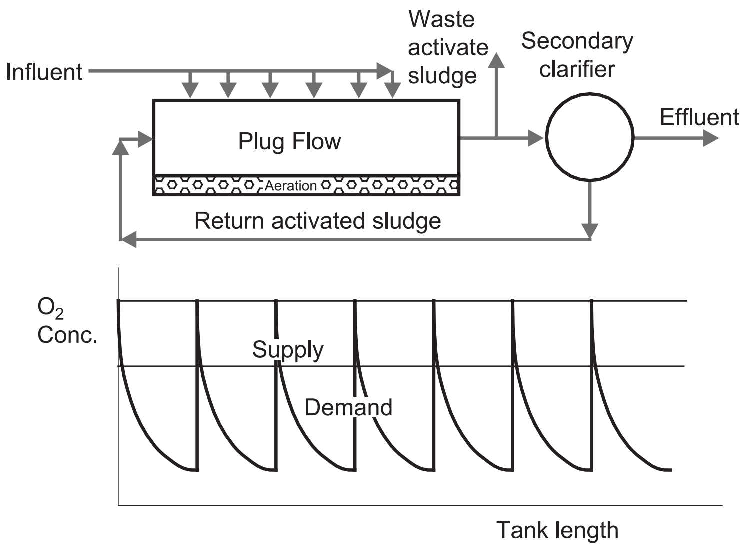

Figure 6.16 Step aeration process.

Tapered aeration. This process attempts to match O_2 supply to demand (through plug flow regime) by introducing more air at the head end (Figure 6.17). In this process, the diffusers are spaced close together (at the influent section) to achieve a high oxygenation rate. The intensive air supply at the head end promotes higher organic degradation. As the mixed liquor traverses the aeration tank, synthesis of new cells occurs, increasing the number of microorganisms and decreasing available food concentration. Such inverse correlation is associated with lower food to microorganism ratio and lower of oxygen demand at the outlet section (Figure 6.17). The spacing of diffusers is therefore increased toward the tank outlet, to match the lower oxygen demand. A disadvantage of the process can be attributed to its vulnerability towards shock loading, or toxic materials.

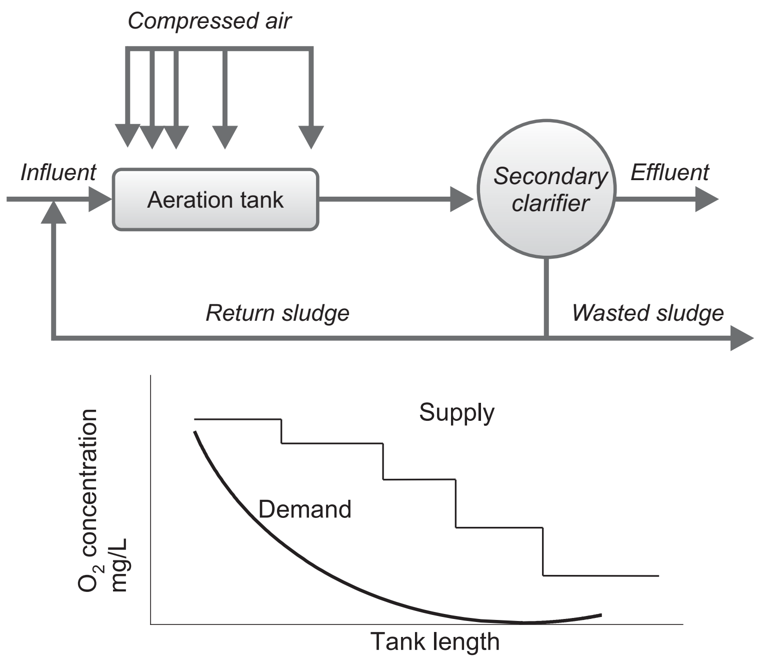

Figure 6.17 Tapered aeration process.

Contact stabilization. According to this process, returned sludge (containing adsorbed organics) from the secondary clarifier is being transferred to a stabilization tank, where it is aerated for 3-6h (Figure 6.18), and is mixed again with influent wastewater for 20-40 min. Such stabilized sludge promotes the adsorption of suspended and colloidal solids (from

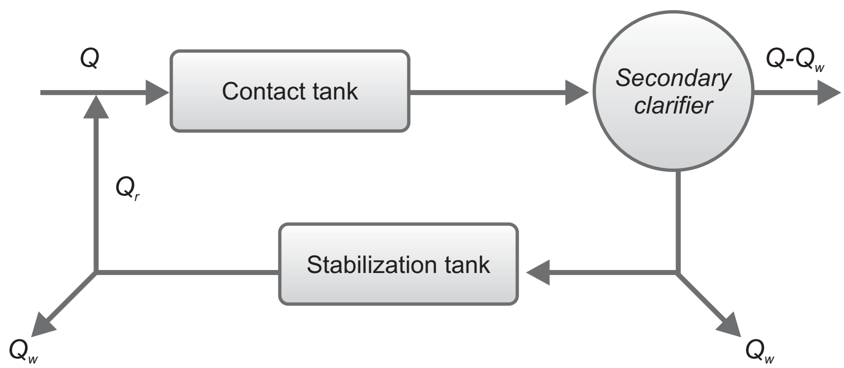

Figure 6.18 Contact stabilization activated sludge process.

influent wastewater), but cannot adsorb dissolved organic matter. After the contact period, the activated sludge is separated from mixed liquor in the secondary clarifier. Contact stabilization is a plug flow process with shorter retention time, which enhances the vulnerability of the system to organic and hydraulic loading variations.

Pure oxygen activated sludge system. In this process, the aeration basin is divided into several compartments by baffle walls; each compartment includes an agitator. The incoming wastewater is mixed with high purity oxygen in the first compartment. Wastewater flows from one compartment to other through baffle walls, following plug flow regime. Exhausted gas, mixture of nitrogen, carbon dioxide and 10% of supplied oxygen emits from the last compartment. The mixed liquor is then transferred to the following secondary clarifier. The settled activated sludge (in secondary clarifier) is again recirculated to the aeration tank, and a portion of the settled sludge is wasted. The mixed liquor of this system has dissolved oxygen concentration varying between 4-10 mg/L. Figure 6.19 provides an operational diagram of pure oxygen activated sludge system.

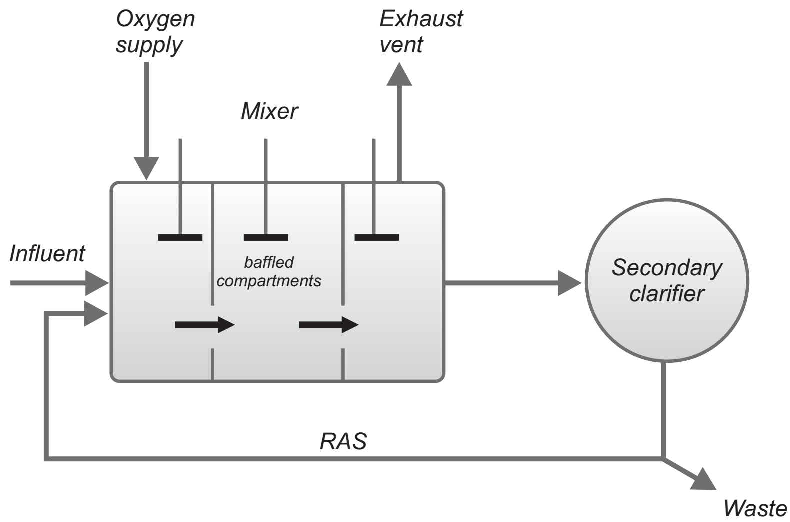

Figure 6.19 Pure oxygen activated sludge process.

The advantages of this system include higher transfer of oxygen, small reaction chambers, peak loading tolerance, higher efficiency in terms of treating soluble wastewater, reduction of sludge bulking, and effective odor control.

Kraus process. This process is effective for the treatment of nitrogen deficient wastewater. The digester supernatant (source of food) is added with a portion of the return sludge, and transferred to the aeration tank for nitrification. The resulting mixed liquor is then transferred to the main plug flow aeration tank. Figure 6.20 provides a schematic diagram of the Kraus process.

Two sludge system. This system is divided into two stages; the first stage removes BOD, whereas the second stage removes ammoniacal nitrogen (NH_4-N) at longer sludge age (Figure 6.21). BOD removal in the first stage promotes treatment of toxic substances, thereby protecting more sensitive nitrifying bacteria in the second stage. A portion of the influent wastewater (of the first stage) can be by passed around the nitrifying stage, to enhance organics and solids availability for nitrification.

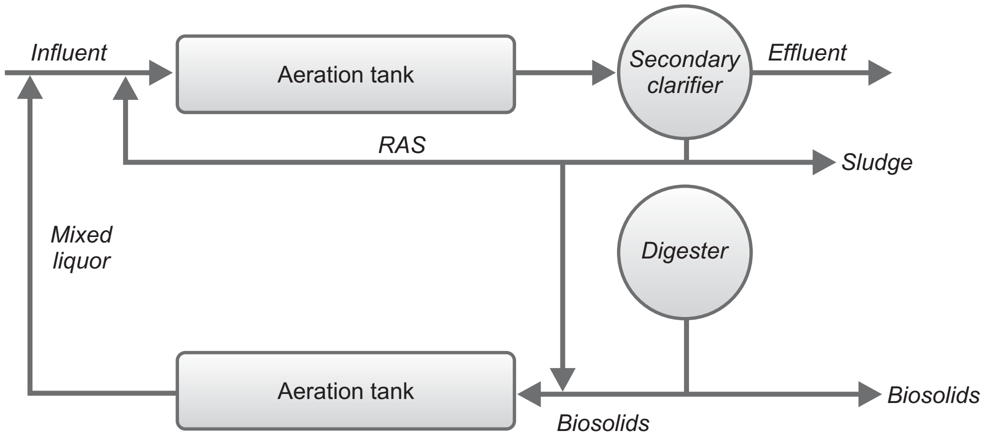

Figure 6.20 Schematic flow diagram of Kraus process.

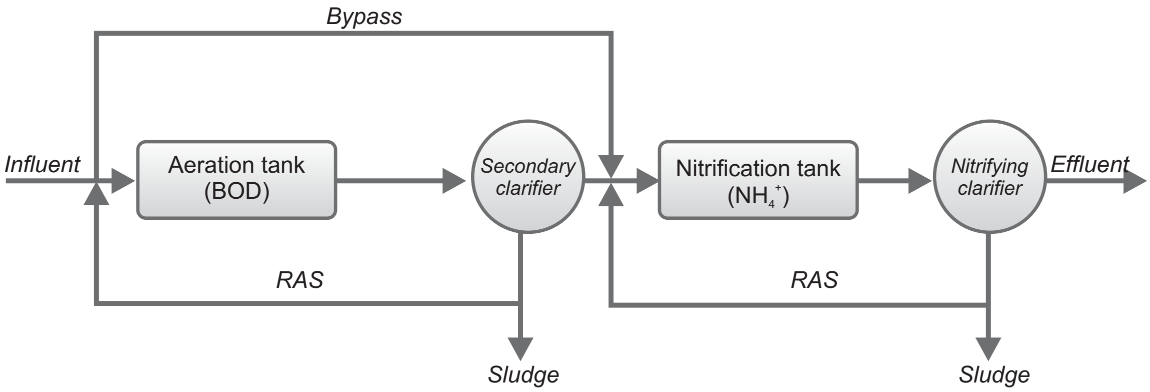

Figure 6.21 Two sludge process.

Extended aeration. The extended aeration process is operated under longer hydraulic retention time and sludge age. The system is also subjected to longer aeration time, resulting higher MLSS concentration, higher RAS pumping rate, and substantial endogenous decay of sludge mass. Such prolonged endogenous phase allows equal proportion of cell mass production and decay, resulting in no net mass production, thereby minimizing sludge production and wastage.

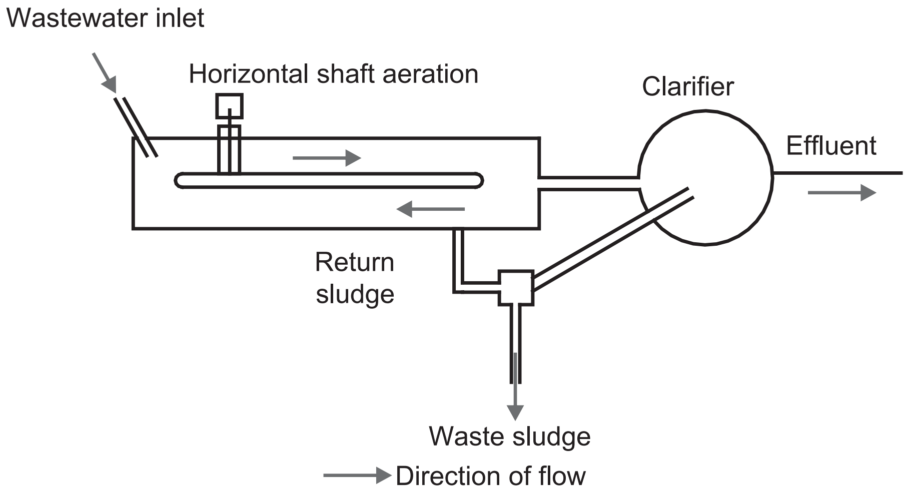

Figure 6.22 Oxidation ditch process.

The oxidation ditch (modification of conventional plug flow process-Figure 6.22) is an example of extended aeration for the treatment of wastewater. It comprises of a single or closed loop elongated oval channel (Figure 6.22); the liquid depth in the channel is maintained within 1.2-1.8 m, with 45 degree sloping sidewalls. The wastewater usually receives preliminary treatment prior to the entrance into oxidation ditch. The wastewater in the channel is aerated with mechanical aerators (i.e. Kessener brush), operating at 60-110 rev/min which allows liquid motion within 0.24-0.37 m/s. Such velocity is maintained to prevent solids deposition; it also allows the wastewater to complete a tank circulation within 5-15 min. The DO concentration profile of wastewater increases near the aerators, and falls when it transverses the circuit.

The oxidation ditch usually exhibits higher BOD_5 removal efficacy. Nitrification also occurs in the system due to longer hydraulic and solids retention time. This system is popular for wastewater treatment generated by small communities, depending on larger land availability. However, in colder periods, surface aerators may be iced, which can be mitigated by installing electrical heating.

Sequential batch reactors (SBR). These systems consist of a single reactor, for aeration of wastewater and sludge settling. The wastewater is applied in a batch mode, resulting four operational phases such as: fill, react, settle and discharge phase (Figure 6.23). During the fill phase, wastewater is applied (in batch mode) to the reactor, which have sludge mass. The

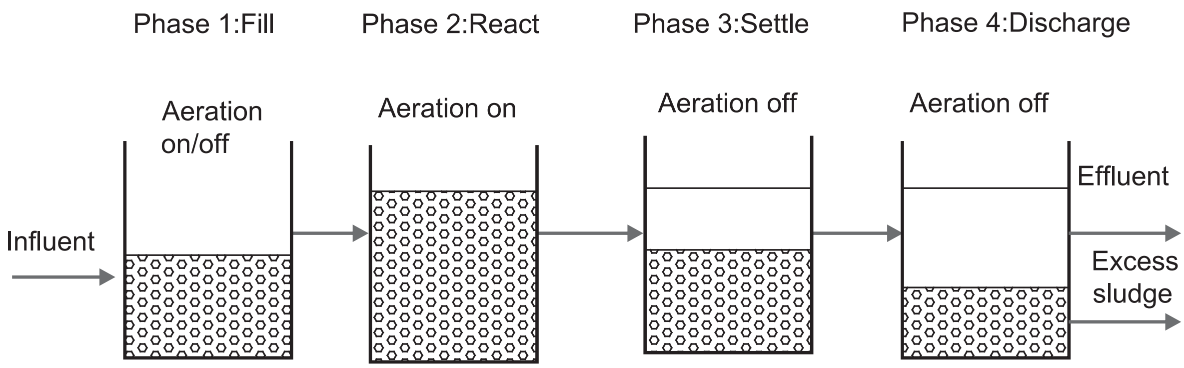

Figure 6.23 **Cyclic operations of a sequential batch reactor.**

aerator may or may not be switched on in this phase. In the react phase, the aerator is turned on to provide treatment of organics and solids, present in wastewater. The settling phase includes settling of sludge in the reactor, while the aerator is turned off. The discharge phase allows withdrawal of the clarified effluent from the reactor, along with excess sludge. The main advantage of SBR can be attributed to simultaneous achievement of equalization, primary clarification, biological treatment and secondary clarification. However, higher maintenance costs, substantial sludge production and greater sludge volume index (SVI) are the major disadvantages of SBR systems.

Typical operational parameters, for the design of some of the above described activated sludge processes have been illustrated in Table 6.7.

Table 6.7 Typical operational parameters of activated sludge processes.

	Operational parameters					
	BOD_5 loading $kg/(m^3.d)$	MLSS concentration, X mg/L	HRT hrs	Sludge age, θ_c d	F/M ratio d^{-1}	BOD_5 removal efficiency %
Step aeration	0.6-1	2000-3500	3-5	5-15	0.2-0.4	85-95%
Contact stabilization	1-1.2					80-90%
Pure oxygen	1.6-3.2	3000-8000	1-3	8-20	0.25-1	85-95%
Kraus process	0.6-1.6	2000-3000	4-8		0.3-0.8	
Oxidation ditch	0.08-0.48	1500-5000			0.05-0.3	85-95%

Example 6.4. Estimate the volume of the aeration tank of high purity oxygen activated sludge system for municipal wastewater treatment employing the following data set, and check with design parameters. Also find out the expected effluent BOD_5 concentration of the system.

Parameter	Value
Design avaerage flow, Q	4000 m^3/d
Influent BOD	300 mg/L
Influent TSS	150 mg/L
F/M	0.6 lb BOD applied/lb MLVSS.d
MLSS	7000 mg/L
VSS/TSS	0.8
Maximum volumetric BOD load	4 $kg/(m^3.d)$
Minimum aeration time	2 h
Minimum cell residence time	4 d
K_s	60 mg/L of BOD
k_d	0.06/d
Y	0.6 VSS/mg BOD
k	6 d^{-1}

Solution

1. *Calculation of tank volume*

$$MLVSS, X = MLSS \times 0.8 = 7000 \times 0.8 = 5600 \text{ mg/L}$$

Employing Equation (6.40):

$$F/M = \frac{S_0}{\theta X} = \frac{QS_0}{VX}$$

$$V = \frac{4000 \text{ m}^3/\text{d} \times 300 \text{ mg/L}}{0.6 \text{ d}^{-1} \times (7000 \times 0.8) \text{ mg/L}} = 357 \text{ m}^3$$

2. Check BOD load

$$BOD_{load} = \frac{4000\ m^3/d \times 300\ mg/L}{357\ m^3} = 3.2\ kg/(m^3d) < 4\ kg/(m^3d)$$

3. Check aeration time

$$HRT = \frac{V}{Q} = \frac{357\ m^3}{4000\ m^3/d} = 2.1\ h > 2\ h\ (OK)$$

4. Check mean cell residence time (sludge age)

Influent VSS, $X_i = 150 \times 0.8 = 120\ mg/L$

Sludge age, $\theta_c = \frac{VX}{QX_i}$

$$= \frac{357\ m^3 \times 5600\ mg/L}{4000\ m^3/d \times 120\ mg/L} = 4.4\ d > 4\ d\ (OK)$$

5. Estimate effluent BOD_5 concentration

Effluent BOD_5 concentration (S) can be computed employing Equation (6.33):

$$S = \frac{K_s(1+\theta_c k_d)}{\theta_c(Yk - k_d) - 1}$$

$$= \frac{60\ mg/L\ (1 + 4.4\ d \times 0.06\ d^{-1})}{4.4\ d\ (0.6 \times 6\ d^{-1} - 0.06\ d^{-1}) - 1} = 5.2\ mg/L$$

6.12 Attached Growth Process: Trickling Filters

Attached culture systems are characterized by the adhesion of bacteria to a solid media. Wastewater passes through this attached film (and porous spaces of media) in thin sheets, allowing the attached microorganisms for adsorbing soluble and colloidal organic matter (from wastewater). The required oxygen for such biodegradation is being supplied by incoming wastewater and atmospheric air. Waste products (from metabolic process) are diffused outwards and carried away by water, through the void of the medium. The widely implemented attached growth process for the treatment of wastewater includes: (a) trickling filters; and (b) rotating biological contactors.

Theory of trickling filters. Trickling filters are non-submerged fixed film biological reactors. Trickling filters consist of: (a) bed of coarse material, such as: stone slates or plastic media; (b) distributors; and (c) underdrain system. Primary treated effluent is sprayed to the filter surface bed, by spray nozzles or rotary distributors (Figure 6.24). The underdrain portion carries the wastewater (passing through biological filter), and drains it to the following treatment units; in addition, the underdrain portion also develops aerobic condition inside the filter media.

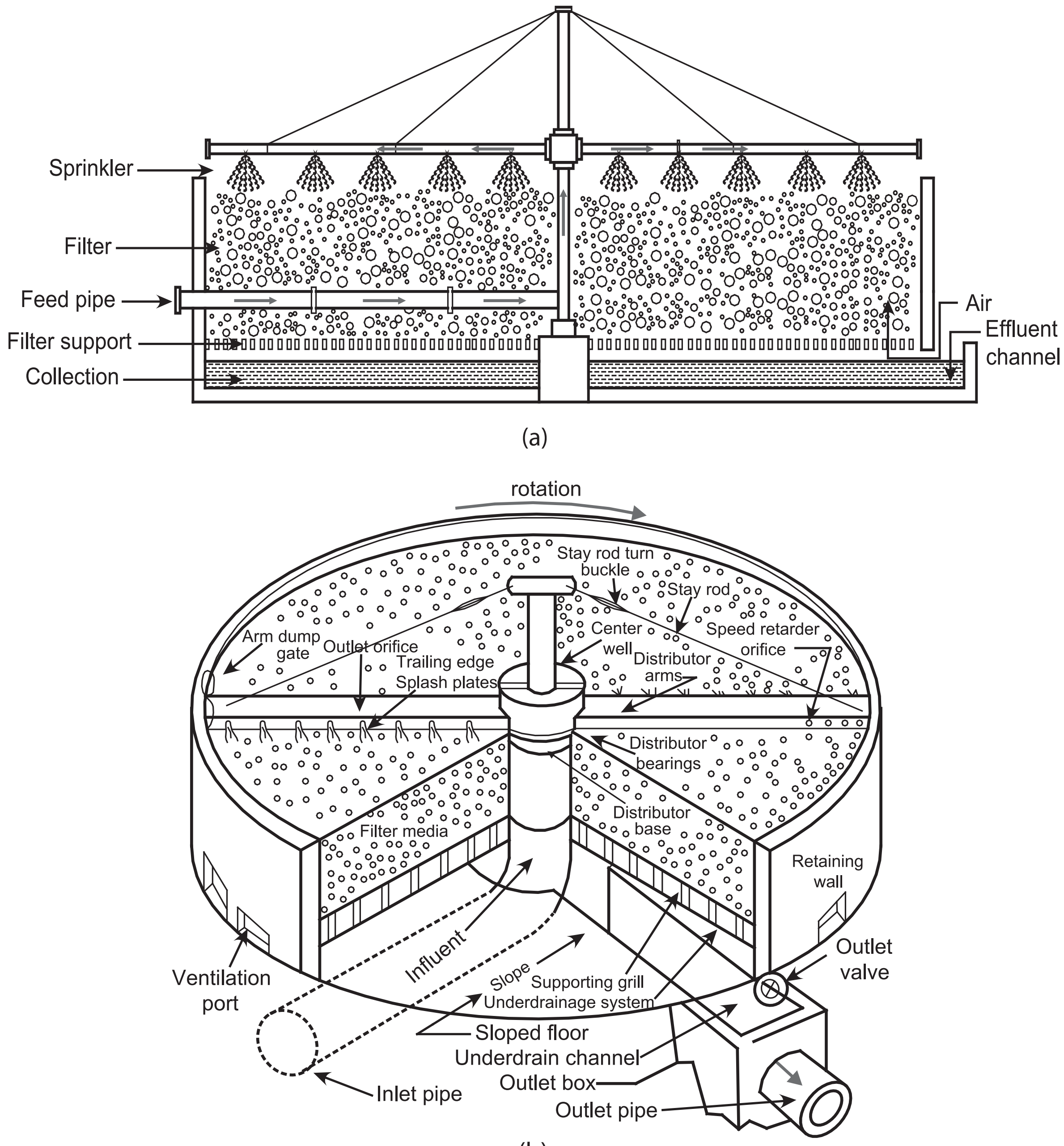

Figure 6.24 **(a) Front elevation; and (b) 3D view of a trickling filter.**

As wastewater passes through the filter media, it promotes the growth of biological slime layer on the media surface. Such developed slime layer comprises of microorganisms, which enhance biodegradation of substrates (from wastewater). The biological community includes aerobic, facultative bacteria, fungi, algae and protozoa. Higher animals such as worms, larvae, and snails can also be an integral part of the biological slime.

The common bacteria species of trickling filters include *Achromobacter, Flavobacterium, Pseudomonas,* and *Alcaligenes*. Within the slime layer, the filamentous forms such as: *Sphaerotilusnatans* and *Beggiatoa* dominate due to adverse environmental conditions. Nitrifying bacteria are observed in the lower part of the filter. The common fungi species of trickling filters are: *Fusazium, Mucor, Penicillium, Geotrichum, Yeasts* etc; rapid growth of fungi often clogs the filter media.

Algae can be observed in the upper portion of the filter, exposed to sunlight. During daytime, algae add oxygen to the incoming wastewater. *Phormidiun*, *Chlorella* and *Ulothrix* are the most common algae species of trickling filters. The protozoa organisms consume the biological films resulting effluent turbidity decrease, and higher growth of biological films.

The removal of organic material (from wastewater) in trickling filter is performed through adsorption by the attached slime layer. In the outer portion of the layer, organic degradation is performed by the aerobic bacteria, which in turn maintains the growth of microorganisms, thereby allowing the increase of slime layer thickness (Figure 6.25). As the thickness of the layer increases, oxygen cannot penetrate in the lower part (near the media), promoting the development of anaerobic environment. The increase of slime layer thickness also enhances the consumption of organics (from wastewater) by the upper aerobic bacterial biomass. As such, the biomass populations of the lower part are deprived of food, thereby creating endogenous phase, where these interior cells die and washed away by the wastewater flow. This phenomenon of losing the slime layer is known as sloughing, and is dependent on organic and hydraulic loading of the filter. The washed out slime layer is removed by the following sedimentation tank (Figure 6.26).

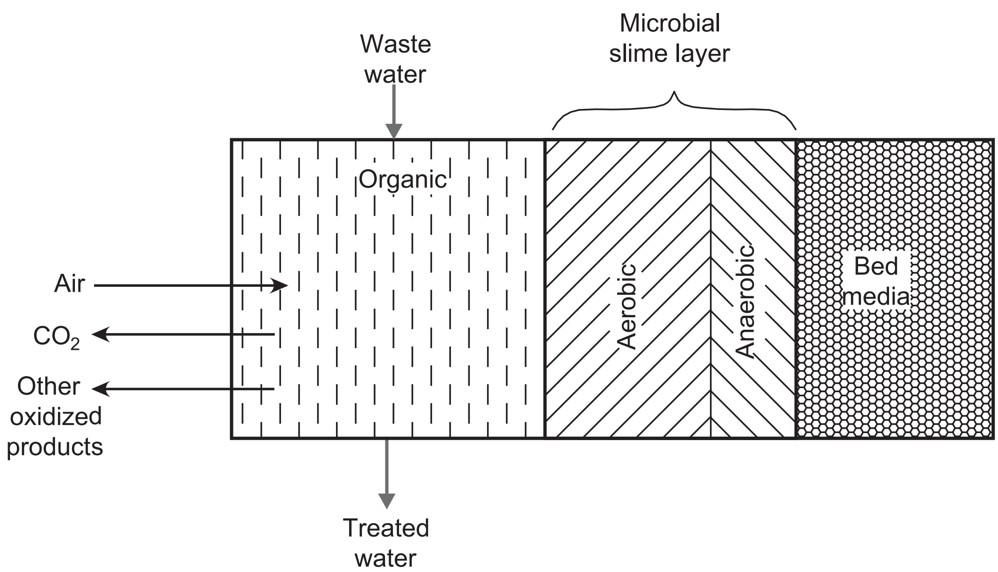

Figure 6.25 **Slime layer metabolism in a trickling filter.**

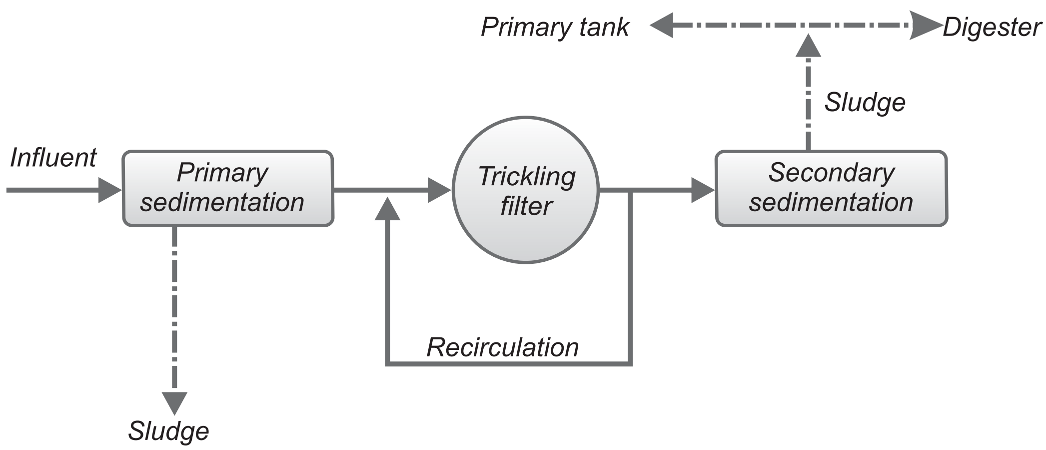

Figure 6.26 **Arrangement of trickling filters for wastewater treatment.**

Classification of trickling filters. Trickling filters can be classified according to the applied hydraulic or organic loading rate, such as: low rate, intermediate, high rate filters, and roughing filters.

In low rate filters, wastewater is fed from small dosing tanks; wastewater passes through the filter only once. A dosing chamber is used to correct poor distribution problems, and inadequate velocities caused by low flows. These dosing chambers hold the incoming flow till a large volume has been accumulated. Contents are then released quickly to provide equal flow by fixed nozzles. In most low rate filters, the top 0.6-1.2 m of the media contains appreciable biological slime. Nitrifying bacteria develops in the lower part of the media. Depending on the climatic conditions, wastewater characteristics and presence of biomass population, a low rate trickling filter may provide substantial BOD and ammoniacal nitrogen removal.

High rate filters use rock or plastic medium; such filters are subjected to continuous flow mode. Recirculation of filter effluent permits higher organic loading and dose rates, better control of slime layer thickness, improved liquid distribution and provides more O_2 in waste water flow.

Roughing filters are high rate type filters (with plastic packing) that treat organic load of more than 1.6 kg/m^3.d, and hydraulic loading up to 190 m^3/m^2.d. In such filters, lower energy is required for BOD removal (from high strength wastewater), when compared with activated sludge processes, because energy is required only to pump influent wastewater and in flow recirculation. Table 6.8 indicates typical design guidelines of different types of filters.

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Table 6.8 Guidelines for the design of trickling filters[a,b].

	Filter type			
	Low rate	**Intermediate rate**	**High rate**	**Roughing**
Packing	Rock	Rock	Rock	Rock/Plastic
Hydraulic loading m^3/m^2.d	1-4	4-10	10-40	40-200
Organic load kg BOD/m^3.d	0.07-0.22	0.24-0.48	0.4-2.4	>1.5
Recirculation ratio	0	0-1	1-2	0-2
Sloughing	Intermittent	Intermittent	Continuous	Continuous
Depth m	1.8-2.4	1.8-2.4	1.8-2.4	0.9-6
BOD removal efficacy %	80-90	50-80	50-90	40-70
Effluent quality	Nitrified	Some nitrification	No nitrification	No nitrification
Power kW/10^{3}m^3	2-4	2-8	6-10	10-20

[a] after Metcalf and Eddy, Inc. (1979), [b] WEF (2000).

Design of trickling filters: NRC formula. The NRC (National Research Council) formula is an empirical formula, developed by the National Research Council of the United States. The formula can be applied for single stage and multi stage rock filters, with varying recirculation ratios. The equation for the removal of BOD in a first stage rock filter is:

$$E_1 = \frac{100}{1 + 0.532\sqrt{\frac{W}{VF}}} \qquad 6.70$$

where E_1 = BOD removal efficiency at the first stage at 20°C, %
W = filter BOD loading, kg/d
V = filter media volume, m^3
F = recirculation factor

Equation (6.71) illustrates the calculation method of the recirculation factor; it represents the average number of passes of incoming organics, through the trickling filter.

$$F = \frac{1+r}{(1+0.1r)^2} \qquad 6.71$$

where r = recirculation ratio, Q_r/Q
Q_r = recirculation flow, m^3/d
Q = wastewater flow, m^3/d

For the second filter, the removal efficacy can be calculated using Equation (6.72):

$$E_2 = \frac{100}{1+\frac{0.532}{1-E_1}\sqrt{\frac{W'}{VF}}} \qquad 6.72$$

where E_2 = BOD removal efficiency at the second stage, %
W' =second stage filter BOD loading, kg/d

The overall BOD removal efficacy of the two-stage filters can be described as:

$$E = E_1 + E_2 - E_1 E_2 \qquad 6.73$$

BOD removal efficacy is influenced by temperature (Equation 6.17), which can be expressed as:

$$E_T = E_{20} 1.035^{T-20} \qquad 6.74$$

Performance of the plastic media: Germain formula. To date, several investigations have been applied to predict the performance of the plastic media (of trickling filters). The application of Schulze formula by Germain in the year 1965 is the widely implemented formula (Equation 6.75), for predicting plastic media performance of the trickling filter.

$$\frac{S_e}{S_i} = exp\left[\frac{-k_{20}D}{q^n}\right] \qquad 6.75$$

where S_e = BOD_5 of the effluent, mg/L
S_i = influent BOD_5 of the filter, mg/L
D = filter depth, m
q = hydraulic loading, $m^3/m^2.d$
n = exponent constant of the media, 0.5
k_{20} = treatability constant related to filter depth at 20°C

The treatability constant is different for various media depth; Albertson and Davis (1984) proposed the following equation to determine the constant for a certain depth, provided that the constant is known for another depth.

$$k_2 = k_1\left(\frac{D_1}{D_2}\right)^x \qquad 6.76$$

where k_2 = treatability constant for depth D_2 of filter 2
k_1 = treatability constant for depth D_1 of filter 1
D_2 = depth of filter 2, m
D_1 = depth of filter 1, m
x = 0.3-0.5

Table 6.9 **Typical *k* values for different wastewater in 6m trickling filters.**[a]

Wastewater type	k values in $(L/s)^{0.5}/m^2$
Domestic	0.18-0.27
Domestic and food waste	0.16-0.22
Fruit canning waste	0.054-0.14
Meat packing waste	0.081-0.14
Paper mill	0.054-0.11
Potato processing	0.095-0.14
Refinery	0.054-0.19

[a] *after* Lin (2007)

The values of k in $(L/s)^{0.5}/m^2$ for a 6 m tower trickling filter with plastic media (at 20 °C) is illustrated in Table 6.9. These values can be used for calculating treatability constant at another depth, employing Equation (6.76).

Distributor speed. The dosing rate to a trickling filter is a function of distributor speed or the on-off times of a fixed distributor. The rotational speed of a rotary distributor is expressed through Equation (6.77):

$$n = \frac{0.00044\, q_t}{\phi\,(DR)} \qquad 6.77$$

where n = distributor rotational speed, rev/min
q = influent hydraulic load, $m^3/m^2.d$
q_r = recycle hydraulic load, $m^3/m^2.d$
q_t = total hydraulic load, $m^3/m^2.d = q+q_r$
φ = number of arms in distributor
DR = dosing rate, cm; dosing rate is determined by multiplying the loading rate by 0.30.

Formulation of plastic media: Eckenfelder formula. An exponential formula had been developed by Eckenfelder (1963) and Eckenfelder and Barnhart (1963), based on the rate of waste removal (Equation 6.78).

$$\frac{S_e}{S_i} = exp\left[-KA_s^{\,1+m} D/q^n\right] \qquad 6.78$$

where S_e = effluent soluble BOD_5, mg/L
S_i = influent soluble BOD_5, mg/L
K = observed reaction rate constant, m/d
A_s = specific surface area, m^2/m^3
D = media depth, m
q = influent volumetric flow rate, Q/A
Q = influent flow, m^3/d
A = cross sectional area, m^2
m, n = empirical constants

Equation (6.78) may be simplified into following form, as illustrated in Equation (6.79).

$$\frac{S_e}{S_i} = exp\,[-kD/q^n] \qquad 6.79$$

where k is a new rate constant, per day.

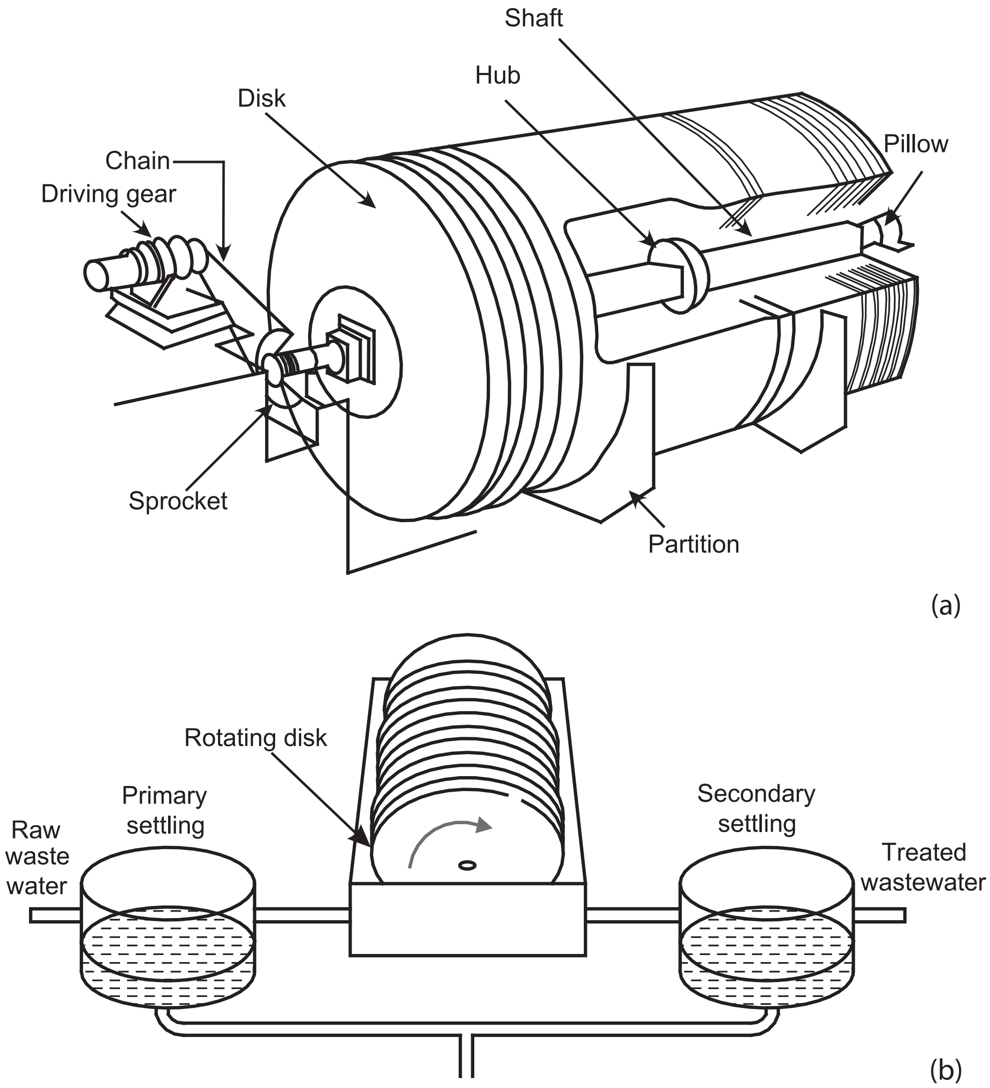

Figure 6.27 **(a) Cross section of circular disks mounted on a shaft; and (b) operational diagram of RBC processes for wastewater treatment.**

6.13 Attached Growth Process: Rotating Biological Contactor

Rotating biological contactors (RBC), also defined as rotating biological filters, are fixed-bed reactors comprising of closely spaced circular plastic media mounted on a horizontal shaft (Figure 6.27a). The disks are partially submerged and rotated as wastewater flows through (Figure 6.27b). The biofilms attached to the surface of the disks are alternately exposed to wastewater and atmosphere, allowing assimilation, aeration and degradation of organic pollutants, nutrients (from primary treated effluent).

The basic components of the RBC system include: media, shaft, bearings, drive and cover. The plastic media are made of corrugated polyethylene or polystyrene material; the diameters of the media can range between 1.22-3.66 m, while the surface area of the disks is about 9300 m^2. The shaft, on which the plastic media is mounted, is supported by bearings, and rotated by the driving gear. The length of the shaft varies between 6.62-8.23 m. The rotational speed of the shaft is 1.5-1.7 rev/min for mechanical drive, and 1-1.3 rev/min for air drive. The required power of the motors can be 3.7/5.6 kW. The typical volume of a RBC tank is 45 m^3; approximately 40% of the media is submerged in wastewater.

Process principle. When employed for domestic wastewater treatment, the microorganisms present in wastewater adhere to the disks within 1 week of the start up phase. During the continuous rotation of the disks, the attached biofilms are exposed to air and wastewater alternatively (Figure 6.28). When the disks are submerged to wastewater, the organics penetrate through the biofilms. As the disks rotate to atmospheric conditions, the oxygen (from air) is diffused in the biofilms, allowing aerobic organic degradation. The DO rich biofilm replenishes oxygen concentration of the wastewater, when the disk is alternatively submerged. Attached biofilms are sloughed in wastewater (Figure 6.28), and being removed in the following secondary clarifiers.

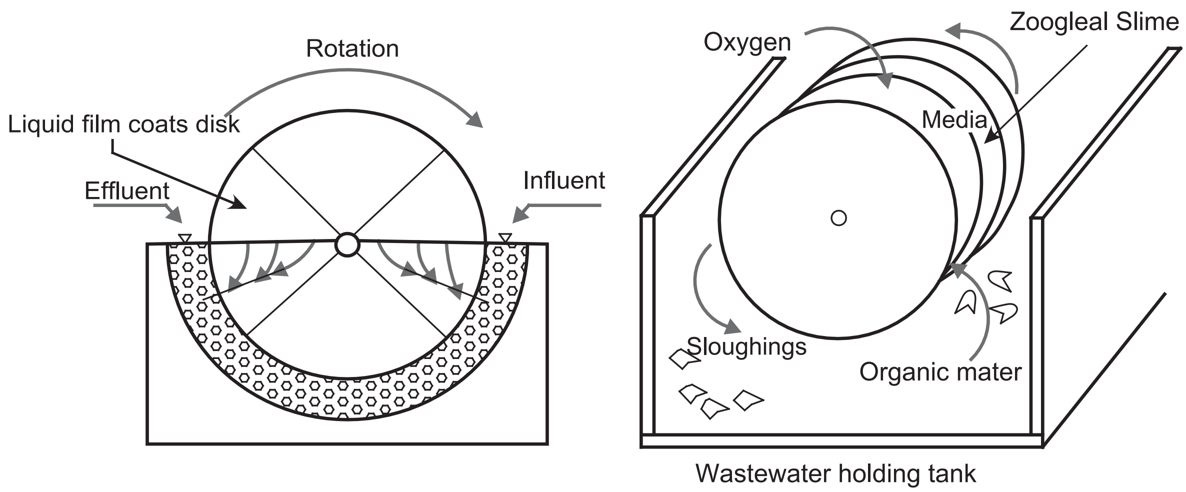

Figure 6.28 **Slime formation across the plastic media of RBC reactors.**

The RBC system includes a number of units in series to prevent short circuiting of incoming wastewater, as illustrated in Figure 6.29. The number of stages depends on treatment objectives; for the removal of BOD 2-4 stages are required, whereas nitrification process can

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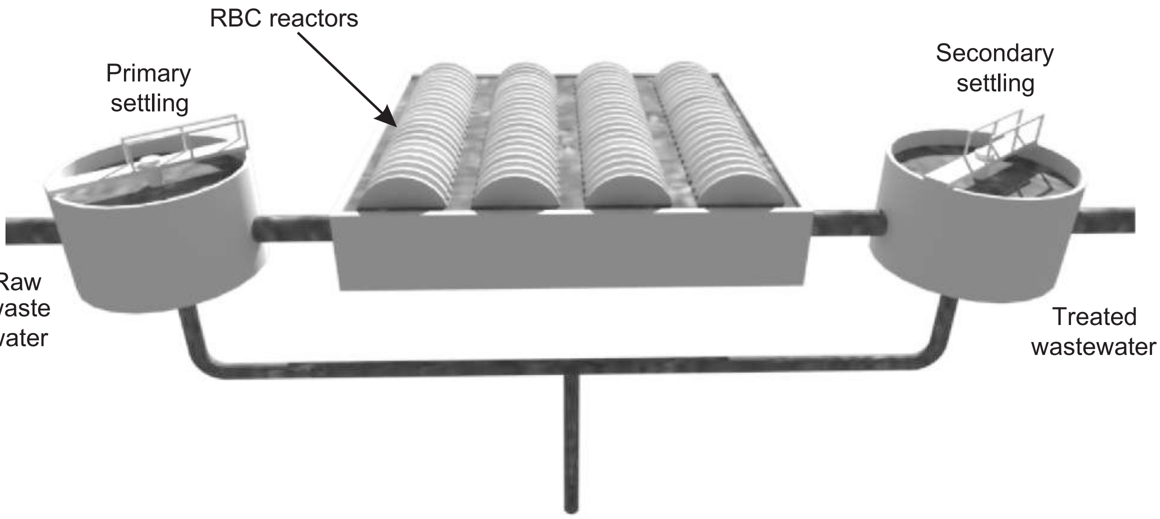

Figure 6.29 **RBC reactors arranged in series for wastewater treatment.**

be enhanced with six or more stages. The establishment of these stages can be accomplished by a single tank with baffles, or by multiple tanks. During stage treatment, the influent organic concentration in the subsequent stage is lower than the previous stage; the hydraulic retention time in each stage is usually shorter (i.e. 20 min under normal loading).

The RBC system has several benefits over the trickling filter process (for wastewater treatment), as enlisted below:

- Reduced loadings on the shaft and bearings.
- Ease of retrofit into existing aeration tanks.
- Lower power consumption.
- Good process stability.
- Better control of short circuiting.

However, overloading of RBC systems is often associated with the development of anaerobic conditions in the deeper portion of the attached biofilms. In such conditions, sulfate is reduced to H_2S and is diffused outwards in oxygen rich environment, followed by oxidation (of H_2S) via *Beggiatoa* bacteria. The continuous oxidation of H_2S enhances the formation of whitish *Beggiatoa* biofilms, that do not slough under normal RBC loading. Such problems can be mitigated by adding hydrogen per oxide. Increase of the rotational speed of the disks also enhances the shear level of the attached biomass. The organic loading to the first stage of an RBC should also be restricted to 3.1kg total $BOD_5/100\ m^2/d$ or 1.2 kg soluble $BOD_5/100\ m^2/d$ (WEF and ASCE, 1992) for smooth operation.

6.14 Design of Secondary Treatment Processes

This section provides comprehensive design procedures of the suspended and attached growth processes, when employed for the treatment of wastewater. The step-wise design procedures of suspended growth process have been illustrated in example 6.5. Subsequently, example 6.6 provides detailed designing of attached growth (i.e. trickling filters) processes.

Example 6.5. **Design of completely mixed activated sludge process.** A completely mixed activated sludge process is required for the treatment of municipal wastewater.

(a) Design the aeration tank employing the following data set:

- Average design flow = 0.60 m^3/s
- Influent BOD_5 concentration = 400 mg/L
- Influent TSS concentration = 350 mg/L
- Effluent BOD_5 concentration = 20 mg/L
- Effluent TSS concentration = 25 mg/L
- Sludge age, $\theta_c = 10$ d
- MLVSS = 3000 mg/L
- VSS/TSS = 0.8
- TSS concentration in RAS = 10000 mg/L
- $Y = 0.5$ mg VSS/mg BOD_5, $k_d = 0.06$/d
- $BOD_5 = 0.65\ BOD_u$
- BOD_5 removal in primary clarifiers = 25%
- TSS removal in primary clarifiers = 60%
- Specific gravity of primary sludge is 1.05, with solid content 4.4%
- Oxygen consumption is 1.42 mg per mg of cell oxidized

(b) Design secondary clarifiers from the data of aeration tank design. The MLSS settling characteristics of the solids are demonstrated below.

MLSS, mg/L	1100	1900	2500	3400	4100	5500	6700	8500
Velocity, m/h	4.5	3.5	2.5	1.4	0.7	0.25	0.1	0.07

Solution

a. Aeration tank design

Step 1. BOD and TSS loading into the plant

Design flow $Q = 0.60\ m^3/s \times 86400\ s/d = 51840\ m^3/d$

BOD loading $= 0.4\ kg/m^3 \times 51840\ m^3/d = 20736\ kg/d$

TSS loading $= 0.35\ kg/m^3 \times 51840\ m^3/d = 18144\ kg/d$

Step 2. Primary sludge characteristics

BOD removed $= 20736\ kg/d \times 0.25 = 5184\ kg/d$

TSS removed $= 18144\ kg/d \times 0.6 = 10886\ kg/d$

Solids concentration = 4.4%=0.044kg/kg

$$\text{Sludge flow rate} = \frac{10886\ kg/d}{1.05 \times 1000\ kg/m^3 \times 0.044\ kg/kg} = 236\ m^3/d$$

Step 3. Effluent characteristics of primary effluent

Flow $= 51840\ m^3/d - 236\ m^3/d = 51604\ m^3/d$

$$\text{Effluent BOD} = 20736 \text{ kg/d} - 5184 \text{ kg/d} = 15552 \text{ kg/d}$$

$$= \frac{15552 \text{ kg/d} \times 1000 \text{ g/kg}}{51604 \text{ m}^3\text{/d}} = 300 \text{ mg/L}$$

$$\text{Effluent TSS} = 18144 \text{ kg/d} - 10886 \text{ kg/d} = 7258 \text{ kg/d}$$

$$= \frac{7258 \text{ kg/d} \times 1000 \text{ g/kg}}{51604 \text{ m}^3\text{/d}} = 140 \text{ mg/L}$$

Step 4. Calculate soluble BOD_5 (S) escaping treatment

Effluent BOD=influent soluble BOD escaping treatment (S) + BOD of effluent suspended solids.

Assuming 60% portion of the effluent solids is biodegradable, the BOD_5 of the suspended solids is = 25 mg/L × 0.6 = 15 mg/L

$$\text{Ultimate BOD}_u \text{ of the effluent solids} = 15 \text{ mg/L} \times 1.42 \text{ mg } O_2\text{/mg cell} = 21 \text{ mg/L}$$

$$BOD_5 = 0.65 BOD_u = 0.65 \times 21 \text{ mg/L} = 13.6 \text{ mg/L}$$

$$\text{Effluent BOD, } 20 \text{ mg/L} = S + 13.6 \text{ mg/L}$$

$$S = 6.4 \text{ mg/L}$$

Step 5. Efficiency (E) of the biological reactor (based on soluble BOD)

$$E = \frac{S_0 - S}{S_0} \times 100 = \frac{(300 - 6.4) \text{ mg/L}}{300 \text{ mg/L}} \times 100 = 98\%$$

Step 6. Reactor volume

Employing Equation (6.32):

$$V = \frac{\theta_c Q Y (S_o - S)}{X (1 + k_d \theta_c)}$$

$$= \frac{10 \text{ d} \times 51604 \text{ m}^3\text{/d} \times 0.5 \times (300 - 6.4) \text{ mg/L}}{3000 \text{ mg/L} (1 + 0.06 \text{ d}^{-1} \times 10 \text{ d})} = 15782 \text{ m}^3$$

Step 7. Tank dimensions

Provide 5 rectangular tanks with common walls; provide a length-width ratio of 1:2, with a water depth of 5.5 m and 0.5 m freeboard.

$$w \times 2w \times 5.5 \text{ m} \times 5 = 15782 \text{ m}^3$$

$$w = 17 \text{ m}$$

Dimension of each tank: width = 17m; length = 34 m, total depth = 6 m (0.5 m freeboard).

Step 8. Sludge wasting flow from aeration tank

Employing Equation (6.24):

$$\theta_c = \frac{VX}{Q_{wa}X + Q_e X_e}$$

$$10\text{ d} = \frac{15782\text{ m}^3 \times 3000\text{ mg/L}}{Q_{wa}\ (3750\text{ mg/L}) + (51604\text{ m}^3/\text{d})\ (25\text{ mg/L} \times 0.8)}$$

$$Q_{wa} = 987\text{ m}^3/\text{d}$$

$$\text{Here, } X = \frac{MLVSS}{0.8}$$

Step 9. Calculate the amount of sludge to be wasted

Employing Equation (6.34):

$$Y_{obs} = \frac{Y}{1 + k_d\theta_c}$$

$$= \frac{0.5}{1 + 0.06 \times 10} = 0.3125$$

Increase of MLVSS mass can be calculated, employing Equation (6.43),

$$P_x = \frac{Y_{obs}Q\,(S_0 - S)}{1000\text{ g/kg}}$$

$$= 0.3125 \times 51604\text{ m}^3/\text{d} \times (300 - 6.4)\text{ g/m}^3 \times 0.001\text{ kg/g} = 4735\text{ kg/d}$$

Increase in MLSS, $p_{ss} = \dfrac{4735\text{ kg/d}}{0.8} = 5918\text{ kg/d}$

Loss of TSS in effluent, $p_e = (51604 - 987)\text{ m}^3/\text{d} \times 25\text{ g/m}^3 \div 1000\text{ g/kg} = 1265\text{ kg/d}$

Amount of sludge to be wasted= $p_{ss} - p_e = (5918 - 1265)\text{ kg/d} = 4653\text{ kg/d}$

Step 10. Calculate RAS

Taking mass balance of VSS around aeration tank (Figure 6.9a):

$$3000\,(Q + Q_r) = 10000 \times 0.8Q_r$$

$$3000Q = 5000Q_r$$

$$Q_r = 30962\text{ m}^3/\text{d}$$

Step 11. Calculate HRT

$$\theta = \frac{V}{Q} = \frac{15782\text{ m}^3}{51604\text{ m}^3/\text{d}} = 7.3\text{ h}$$

Step 12. Calculate F/M ratio

F/M ratio can be calculated with Equation (6.40):

$$F/M = \frac{S_0}{\theta X}$$

$$= \frac{300 \text{ mg/L}}{0.30 \text{ d} \times 3000 \text{ mg/L}} = 0.33 \text{ d}^{-1}$$

Step 13. Check organic loading

$$Loading = \frac{QS_0}{V} = \frac{51604 \text{ m}^3\text{/d} \times 300 \text{ g/m}^3}{15782 \text{ m}^3 \times 1000 \text{ g/kg}} = 0.98 \text{ kg } BOD_5\text{/m}^3\text{d}$$

Step 14. Calculate oxygen requirements

Employing Equation (6.44):

$$\text{kg } O_2\text{/d} = \frac{Q\,(S_o - S)}{(1000 \text{ g/kg})\, f} - 1.42 P_x$$

$$= \frac{51604 \text{ m}^3\text{/d} \times 293.6 \text{ g/m}^3}{0.65 \times 1000 \text{ g/kg}} - 1.42 \times 4735 \text{ kg/d} = 16585 \text{ kg/d}$$

Step 15. Calculate air volume requirements

Assume, density of air is 1.202 kg/m^3, with 23.2% oxygen by weight. The transfer efficiency of aeration equipment is 8%, and a factor of safety 2.5 will be used for measuring the actual blower size.

$$\text{The theoretical requirement of air} = \frac{16585 \text{ kg/d}}{1.202 \text{ kg/m}^3 \times 0.232 \text{g } O_2\text{/g air}} = 59473 \text{ m}^3\text{/d}$$

$$\text{The actual air required at an 8\% oxygen transfer efficacy} = \frac{59473 \text{ m}^3\text{/d}}{0.08} = 743418 \text{ m}^3\text{/d}$$

$$= 516 \text{ m}^3\text{/min}$$

$$\text{The design air requirement} = 516 \text{ m}^3\text{/min} \times 2.5 = 1290 \text{ m}^3\text{/min}$$

$$\text{Air requirement per kg of BOD}_5 \text{ removal} = \frac{743418 \text{ m}^3\text{/d} \times 1000 \text{ g/kg}}{51604 \text{ m}^3\text{/d} \times (300 - 6.4) \text{ g/m}^3}$$

$$= 49.1 \text{ m}^3\text{air/kg } BOD_5$$

b. Secondary clarifiers design

Step 1. Calculate solid flux

For MLSS concentration 1100 mg/L, $G = MLSS \text{ (kg/m}^3) \times \text{velocity (m/h)}$

$$= 1.1 \times 4.5 = 4.9 \text{ kg/m}^2\text{h}$$

Similar procedure can be followed for determining G, employing other MLSS concentrations.

MLSS, mg/L	1100	1900	2500	3400	4100	5500	6700	8500
G, kg/m^2. h	4.9	6.6	6.2	4.7	2.0	1.3	0.6	0.5

Step 2. Plot MLSS vs. G curve

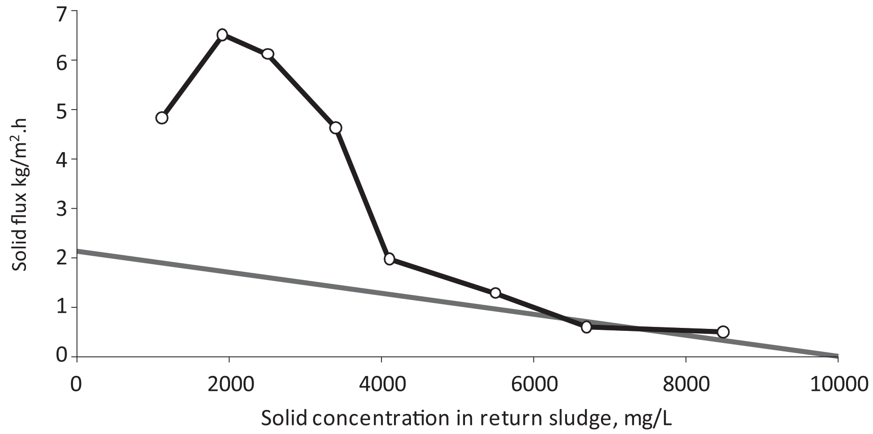

Figure 6.30 **MLSS vs. G curve for the design of secondary clarifiers.**

From the graph G_L is calculated as 2.3 kg/m^2.h.

Step 3. Determine flow rate to the clarifiers

$$\text{Clarifier flow rate} = Q + Q_r - Q_{wa} \text{ (from aeration tank design)}$$

$$= (51604 + 30962 - 987)\ \text{m}^3/\text{d} = 81579\ \text{m}^3/\text{d} = 0.95\ \text{m}^3/\text{s}$$

Provide 4 clarifiers, each receiving a flow rate of 20394 m^3/d (849m^3/h).

Step 4. Determine surface area of each clarifier

$$\text{Surface area of each clarifier, } A = \frac{QX}{G_L}$$

$$= \frac{849\ \text{m}^3/\text{h} \times \dfrac{(3000/1000)\ \text{kg/m}^3}{0.8}}{2.3\ \text{kg/m}^2\text{h}} = 1384\ \text{m}^2$$

Step 5. Determine diameter of each clarifier

$$\frac{\pi d^2}{4} = 1384\ \text{m}^2$$

$$d = 42\ \text{m}$$

Step 6. Check overflow rate

$$\text{Overflow rate} = \frac{Q}{A} = \frac{20394\ \text{m}^3/\text{d}}{1384\ \text{m}^2} = 14.7\ \text{m}^3/\text{m}^2\text{d} = 0.612\ \text{m/h}$$

From the graph (Figure 6.31), for settling rate 0.612 m/h the MLSS concentration is 4680 mg/L, which is higher than the design MLSS concentration (3750 mg/L). So the design area is adequate.

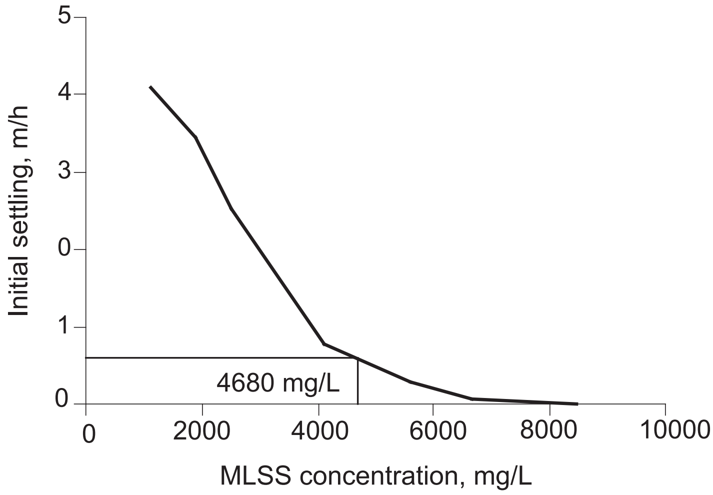

Figure 6.31 **Settling rate of particles.**

Step 7. Determine recycle ratio to maintain MLSS concentration of 3750 mg/L

$$(Q+Q_r)\times 3750 = QX + Q_r X_u$$

$$Q\,(3750-X) = Q_r\,(X_u - 3750)$$

X = 140 mg/L (step 3, aeration tank design)

$$\frac{Q_r}{Q} = \frac{3750-X}{X_u-3750} = \frac{3750-140}{10000-3750} = 0.57$$

Step 8. Depth of thickening zone

The total depth of secondary clarifiers is the sum of clear water zone, solid thickening zone, and sludge storage zone. The mass of solids, stored in the secondary tank is 30% solid mass of the aeration tank, with average solid concentration of 7000 mg/L (Metcalf and Eddy, 1991).

The system has 5 aeration tanks and four secondary clarifiers. As such, solid mass in each aeration tanks can be computed as (considering MLSS concentration):

$$3.75\ \text{kg/m}^3 \times 5.5\ \text{m} \times 34\ \text{m} \times 17\ \text{m} = 11921\ \text{kg}$$

Assuming storage capacity for 4 days due to peak flow (3 times of average flow), and 8 days peak BOD loading (2 times of average BOD loading), the mass of solids in a clarifier for 4 days can be computed as: $11921\ \text{kg} \times 0.3 \times 4 = 14305\ \text{kg}$

$$\text{The depth of sludge zone} = \frac{\text{mass}}{\text{area} \times \text{concentration}}$$

$$= \frac{14305\ \text{kg} \times 1000\ \text{g/kg}}{1384\ \text{m}^2 \times 7000\ \text{g/m}^3} = 1.5\ \text{m}$$

Step 9. Depth of sludge storage zone

Employing Equation (6.43):

$$P_x = Y_{obs}\,Q\,(S_o - S)/(1000\text{ g kg})$$

For peak flow and BOD loading conditions (as discussed above), the value of the parameters of Equation (6.43) is:

Y_{obs}=0.3125 (from aeration tank design), Q=3×0.6m^3/s=1.8 m^3/s=155520m^3/d, S_0=300 mg/L×2=600mg/L, S=6.4 mg/L×2=12.8 mg/L.

$$P_x = 0.3125 \times 155520\text{ m}^3/\text{d} \times (300 - 12.8)\text{ g m}^3/(1000\text{ g/kg}) = 13958\text{ kg}$$

$$\text{Total solids to be stored for 4 days} = 13958\text{ kg} \times 4\text{ d}/0.8 = 69790\text{ kg}$$

$$\text{Solids to be stored in each clarifier} = 69790\text{ kg}/4 = 17448\text{ kg}$$

$$\text{Total solids in each secondary clarifier} = 17448\text{kg} + \text{solids stored in thickening zone}$$

$$= (17448 + 14305)\text{ kg} = 31753\text{ kg}$$

$$\text{The required sludge storage depth in each clarifier} = \frac{31753\text{ kg} \times 1000\text{ g/kg}}{7000\text{ g/m}^3 \times 1384\text{ m}^2} = 3.27\text{m}$$

Step 10. Total depth of each clarifier

The depth of clear water and settling zone ranges within 1.5-2m.

$$\text{Depth} = 2\text{ m} + 1.5\text{ m} + 3.27\text{ m} = 6.77\text{ m} \approx 7\text{ m}$$

Adding 0.5 m freeboard, the total water depth is 7.5m.

Step 11. Engineering diagram of the secondary clarifier

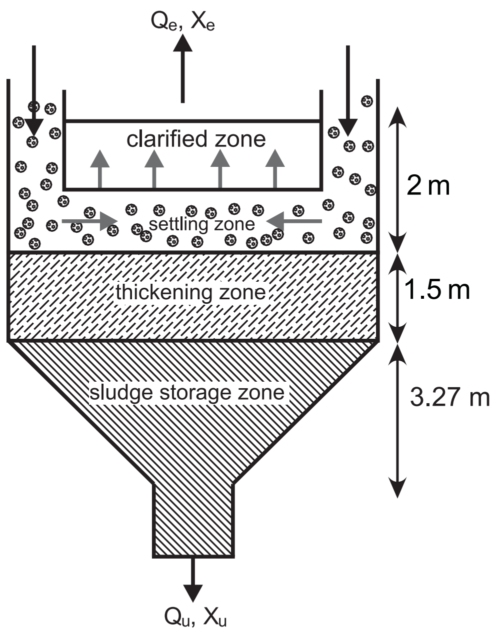

Figure 6.32 **Dimensions of the secondary clarifier.**

CHAPTER 6

Step 12. Flow analyses diagram across the system

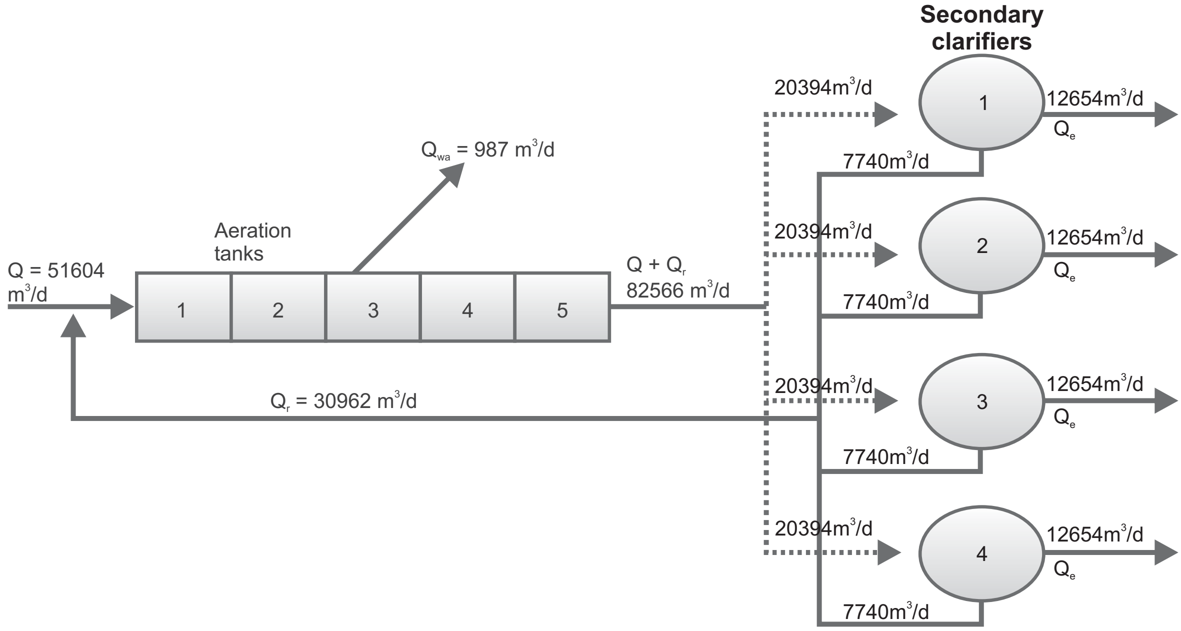

Figure 6.33 **Wastewater flow analyses diagram across activated sludge process.**

Example 6.6. Design of trickling filters. The domestic wastewater generated from a community is discharged to open water bodies, without any treatment. Due to severe quality degradation of such water bodies, the local authority has proposed a combination of primary and secondary treatment processes, for domestic wastewater treatment. The authority has selected trickling filters, as secondary treatment process. Under such circumstances:

(a) Design two stage trickling filters (for the community) using NRC formula from the following dataset:

- Water temperature = 30°C
- Incoming wastewater = 2000 m^3/d
- Influent BOD = 150 mg/L
- Estimated effluent BOD = 20 mg/L
- Depth of each filter = 2 m
- Recirculation for filter 1 and 2 ($r_1 = r_2$) = 1.5
- Assume both filters will have equal BOD removal efficacy

(b) Owing to rapid expansion of the community, the wastewater generation by the same community has increased to a factor of 1.5 after 10 years. In addition, a food industry has also been established in the area during this time period, and the wastewater produced from the industry is being mixed with domestic wastewater prior to discharge. As such the existing two-stage trickling filters cannot meet BOD discharge criteria, and replacement/upgrading are required. After the necessary feasibility studies, the local authority has decided to replace the two-stage trickling filters with a single stage roughing filter, for the treatment of the combined domestic and food processing wastewater. Design a single stage roughing filter (with Germain formula) for the community employing original (Example 6.6a), and additional dataset given below:

- Total domestic wastewater flow = 1.5Q
- Industrial wastewater flow = 1500 m^3/d
- Influent domestic and industrial BOD = 500 mg/L
- Final BOD = 20 mg/L
- K value at 26°C and at 6 m = 0.27 $(L/s)^{0.5}/m^2$
- Depth of filter = 8m

Solution

a. Design of two stage filters

Step 1. Determine E_1 and E_2

$$\text{Overall efficiency, } E = \frac{(150-20)\text{ mg/L} \times 100}{150\text{ mg/L}} = 87\%$$

Equation (6.73): $E_1 + E_2(1 - E_1) = 0.87$

Since $E_1 = E_2$

Therefore $E_1 = 64\%$

Step 2. Calculation of recirculation factor F

From Equation (6.71):

$$F = \frac{1+r}{(1+0.1r)^2}$$

$$= \frac{1+1.5}{(1+0.1\times 1.5)^2} = 1.9$$

Step 3. Calculation BOD load (W) to the first stage

Influent BOD $= 150\text{ mg/L} = 0.15\text{ kg/m}^3$

$$W = QC_1 = 2000\text{ m}^3/\text{d} \times 0.15\text{ g/m}^3 = 300\text{ kg/d}$$

Step 4. Calculate the volume of the first stage

Employing Equation (6.70):

$$E_1 = \frac{100}{1+0.532\sqrt{\dfrac{W}{VF}}}$$

$$64 = \frac{100}{1+0.532\sqrt{\dfrac{300}{1.9V}}}$$

$$V = 142\text{ m}^3$$

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Step 5. Calculate the diameter of the first stage

$$\text{Area} = \frac{\pi d^2}{4} = \frac{V}{\text{depth}} = \frac{142\ \text{m}^3}{2\ \text{m}} = 71\ \text{m}^2$$

$$d = 9.5\ \text{m}$$

Step 6. Calculate the mass BOD loading to the second stage

$$W' = W\ (1-E) = 300\ \text{kg/d}\ (1-0.64) = 108\ \text{kg/d}$$

Step 7. Calculate the volume of the second stage

$$E_2 = \frac{100}{1+\dfrac{0.532}{1-E_1}\sqrt{\dfrac{W'}{VF}}}$$

$$64 = \frac{100}{1+\dfrac{0.532}{1-0.64}\sqrt{\dfrac{108}{1.9V}}}$$

$$V = 400\ \text{m}^3$$

Step 8. Calculate the diameter of the first stage

$$A = \frac{\pi d^2}{4} = \frac{400\ \text{m}^3}{2\text{m}} = 200\ \text{m}^2$$

$$d = 16\ \text{m}$$

Step 9. Check BOD loading to each unit

$$\text{BOD loading (filter 1)} = \frac{W}{V} = \frac{300\ \text{kg/d}}{142\ \text{m}^3} = 2.1\ \text{kg/m}^3\text{d}$$

$$\text{BOD loading (filter 2)} = \frac{W'}{V} = \frac{108\ \text{kg/d}}{400\ \text{m}^3} = 0.27\ \text{kg/m}^3\text{d}$$

Step 10. Check hydraulic loading to each unit

$$\text{HLR loading (filter 1)} = \frac{(1+1.9)\times 2000\ \text{m}^3/\text{d}}{71\ \text{m}^2} = 81.6\ \text{m}^3/\text{m}^2\text{d}$$

$$\text{HLR loading (filter 2)} = \frac{(1+1.9)\times 2000\ \text{m}^3/\text{d}}{200\ \text{m}^2} = 29\ \text{m}^3/\text{m}^2\text{d}$$

Step 11. Calculate the rotation speed of the rotary distributor in the first stage

DR can be calculated by multiplying BOD load rate of filter 1 (step 9) with 0.30.

$$DR = 0.30 \times 2.1 = 0.63\ \text{cm/pass}$$

Use 2 arms in the rotary distributor, i.e. φ=2

Employing Equation (6.77):

$$n = \frac{0.00044\, q_t}{\varphi\, (DR)}$$

$$= \frac{0.00044 \times 81.6}{2 \times 0.63} = 0.03 \text{ rev/min}$$

where $q_t = q + q_r = (81.6 + 0)\ \text{m}^3/\text{m}^2\text{d} = 81.6\ \text{m}^3/\text{m}^2\text{d}$

b. Design of single stage roughing filter

Step 1. Calculate value of k at 30°C at 6m.

$$k_{30,6} = k_{26,6}\, \theta^{T-26}$$

$$= (0.27)\ (\text{L/s})^{0.5}/\text{m}^2 \times 1.035^{30-26}$$

$$= 0.30\ (\text{L/s})^{0.5}/\text{m}^2$$

Step 2. Calculate value of k at 30°C at 8m

Employing Equation (6.76):

$$k_2 = k_1 \left(\frac{D_1}{D_2}\right)^x$$

$$k_{30,8} = 0.30\left(\frac{6}{8}\right)^{0.5} = 0.25(\text{L/s})^{0.5}/\text{m}^2$$

Step 3. Compute the total flow (Q_t)

Q_t = domestic + industrial flow

$$= 1.5Q + 1500\ \text{m}^3/\text{d}$$

$$= 1.5 \times 2000 + 1500 = 4500\ \text{m}^3/\text{d} = 52.1\ \text{L/s}$$

Step 4. Compute the required area

Employing Equation (6.75):

$$\frac{S_e}{S_i} = exp\left[\frac{-k_{30} D}{q^n}\right]$$

$$\ln\frac{S_e}{S_i} = -\frac{k_{30} D}{\left(\dfrac{Q}{A}\right)^n}$$

$$A = Q\left[-\left(\ln\frac{S_e}{S_i}\right)/k_{30} D\right]^{1/n}$$

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$$= 52.1\left[-\left(\ln\frac{20}{500}\right)/(0.25\times 8)\right]^{\frac{1}{0.5}} = 135\text{m}^2$$

Step 5. Compute hydraulic load

$$HLR = 4500\ \text{m}^3\ \ \text{d}/135\ \text{m}^2 = 3.3\ \text{m}^3/\text{m}^2\text{d}$$

Step 6. Compute organic load

$$\text{Filter volume}=8\ \text{m}\times 135\ \text{m}^2 = 1080\ \text{m}^3$$

$$\text{BOD load}=\frac{4500\ \text{m}^3/\text{d}\times 500\ \text{g/m}^3}{1080\ \text{m}^3\times 1000\ \text{g/kg}} = 2.1\ \text{kg/m}^3\text{d}$$

Step 7. Calculate the rotation speed of the rotary distributor in the first stage

$$DR = 0.30\times 2.1 = 0.63\ \text{cm/pass}$$

Use 2 arms in the rotary distributor, i.e. φ=2

Employing Equation (6.77):

$$n = \frac{0.00044q_t}{\phi\ (DR)} = \frac{0.00044\times 3.3}{2\times 0.63} = 0.001\ \text{rev/min}$$

Questions

1. Write a short note on different microorganisms of wastewater treatment process.
2. Describe the different phases usually observed in bacterial growth curve.
3. Derive the necessary equations for calculating biomass concentration in activated sludge process.
4. Why sludge bulking is observed in secondary clarifiers?
5. Write short notes on: (a) tapered aeration; (b) contact stabilization; and (c) sequential batch reactors.
6. What is the main difference between trickling filters and rotating biological contactors employed for wastewater treatment?
7. Design a two stage trickling filters for domestic wastewater treatment from the following dataset. Use NRC formula.
   - Water temperature= 20°C
   - Incoming wastewater= 3000 m^3/d
   - Influent BOD=200 mg/L
   - Estimated effluent BOD=20 mg/L
   - Depth of each filter=1.5 m
   - Recirculation for filter 1 and 2 (r_1=r_2)=1.5

Assume both filters will have equal BOD removal efficacy.

CHAPTER 6

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