Principles of Wastewater Treatment

PDH: 5.0 Hours

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1. Course Overview

This course introduces the principles of wastewater microbiology, characteristics of domestic and industrial wastewater, on-site disposal, municipal wastewater treatment systems, unite operations of pretreatment, primary treatment, unit processes for secondary treatment, disinfection, and advanced wastewater treatment. In this course we define several terms related to secondary treatment processes mostly activated sludge. we solve several problems to understand the basic concept of wastewater treatment and design some unit processes. This course is suggested for civil engineers, environmental engineers, and water and wastewater treatment plant managers and operators.

2. Learning Objectives

Upon successful completion of this course, the participants will be able to:

- identify/interpret the role of microorganisms in wastewater treatment.
- categorize domestic and industrial wastewater and suggest best available technology (BAT) for wastewater treatment.
- define and design simple unit processes for preliminary/pretreatment, primary, and secondary wastewater treatment.
- discuss the concept and purpose of advanced wastewater treatment.

3. Introduction

Wastewater has historically been considered nuisance to be discarded in the cheapest, least offensive manner possible.

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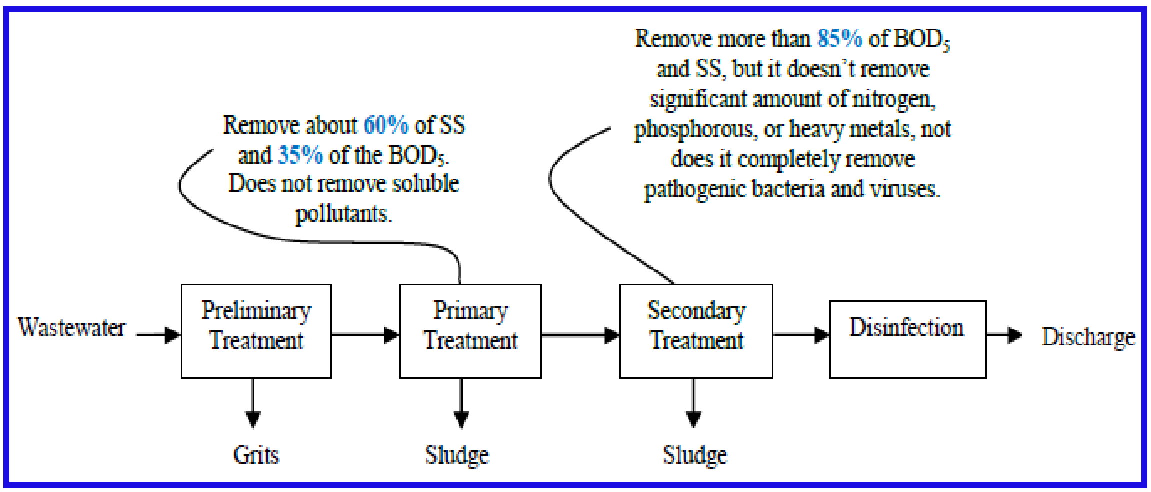
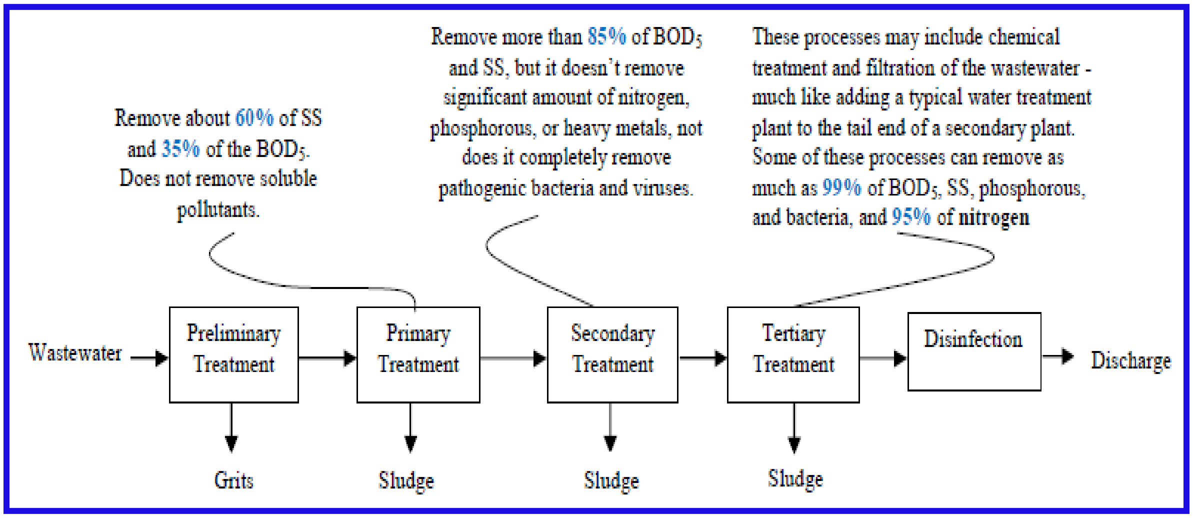


Typical WWTP Flowchart w/o Tertiary Treatment



Typical WWTP Flowchart with Tertiary Treatment

4. Wastewater Microbiology

4.1 Role of Microorganisms

The microorganisms convert the colloidal and dissolved carbonaceous organic matter into various gases and into protoplasm. Protoplasm has specific gravity slightly greater than water, so it can be removed from the treated liquid by gravity settling. Unless protoplasm, produced from the organic matter, is removed from the solution, treatment will not be complete as protoplasm itself is an organic and will be measured as **BOD** in the effluent.

4.2 Classification of Microorganisms

Microorganisms are **organized into five broad groups** (called kingdom) based on their structural and functional differences:

1. **Animal** - multicellular, motile, exhibit tissue differentiation, heterotrophic (Examples: Rotifers, Crustaceans).

2. **Plants** - multicellular, nonmotile, exhibit tissue differentiation, most photosynthetic (Examples: Mosses, Ferns, Some algae).

3. **Protista** - most unicellular, most motile, exhibit tissue differentiation, heterotrophic (Examples: Amoebas, some algae).

4. **Fungi** - most multicellular, nonmotile, heterotrophic, decomposers (Examples: Mashrooms, Yeasts).

5. **Bacteria** - unicellular, some motile, some nonmotile, some heterotrophic, decomposers, some photosynthetic (Examples: Salmonella, Escherichia).

4.2.1 By Energy and Carbon Source

Carbon is a basic building block for cell synthesis.

Based on Carbon Source:

Heterotrophs - use organic material as a supply of carbon.
Autotrophs - use CO_2 to supply their carbon needs.

Based on Energy Source:

1. **Phototrophs** - rely on the sun for energy supply
2. **Chemotrophs** - extract energy from organic or inorganic oxidation/reduction reactions.
3. **Organotrophs** - use organic material as energy source.
4. **Lithotrophs** -oxidize inorganic compounds for energy.

4.2.2 By Their Relationship To Oxygen

Bacteria are classified based on their ability or inability to utilize oxygen as a terminal electron acceptor in oxidation/reduction reactions.

1. **Obligate aerobes** - must have oxygen as a terminal electron acceptor. When wastewater contains oxygen and can support obligate aerobes, it is called aerobic.
2. **Obligate anaerobes** - microorganisms that cannot survive in the presence of oxygen and they cannot use oxygen as a terminal electron acceptor. Wastewater that contains no oxygen is called anaerobic.
3. **Facultative aerobes** - can use oxygen as a terminal electron acceptor, and under certain conditions, they can also grow in the absence of oxygen.

Under anoxic conditions, a group of facultative anaerobes called *denitrifiers* utilizes nitrites (NO_2^-) and nitrates (NO_3^-) as the terminal electron acceptor. Nitrate nitrogen is converted to nitrogen gas in the absence of oxygen. This process is called *anoxic denitrification.*

4.2.3 By Their Preferred Temperature Regime

Four types of bacteria based on the temperature range:

1. **Psychrophiles** - Those that grow best at temperatures below 20°C.
2. **Mesophiles** - Those that grow best at temperature between 25°C to 40°C.
3. **Thermophiles** - Those that grow best at temperature between 45°C to 60°C.
4. **Stenothermophiles** - Those that grow best at temperature above 60°C.

*The growth range of **facultative thermophiles** extends from the thermophilic range into the mesophilic range.*

4.3 Some Microbes of Interest in Wastewater Treatment

Bacteria:

- Highest population of microorganisms in wastewater treatment.
- They are single-celled organisms and use soluble food.
- Chemoheterotrophs are the most suitable for wastewater treatment.
- No particular species is selected as "the best."

Fungi:

- They are multicelluar, nonphotosynthetic, and hetorotrophic.
- They are obligate aerobes that reproduce by variety of methods including fission, budding, and spore formation.
- Their cells require only half as much nitrogen as bacteria so that in a nitrogen-deficient wastewater, they predominate over bacteria.

Algae:

- They are photoautotrophs and may be either unicellular or multicellular.
- Because of the chlorophyll contained in most species, they produce oxygen through photosynthesis.
- In the absence of sunlight, the photosynthetic production of oxygen is greater than the amount used in respiration.
- At night they use up oxygen in respiration.
- Net production of oxygen if daylight hours exceeds night hrs.

Protozoa:

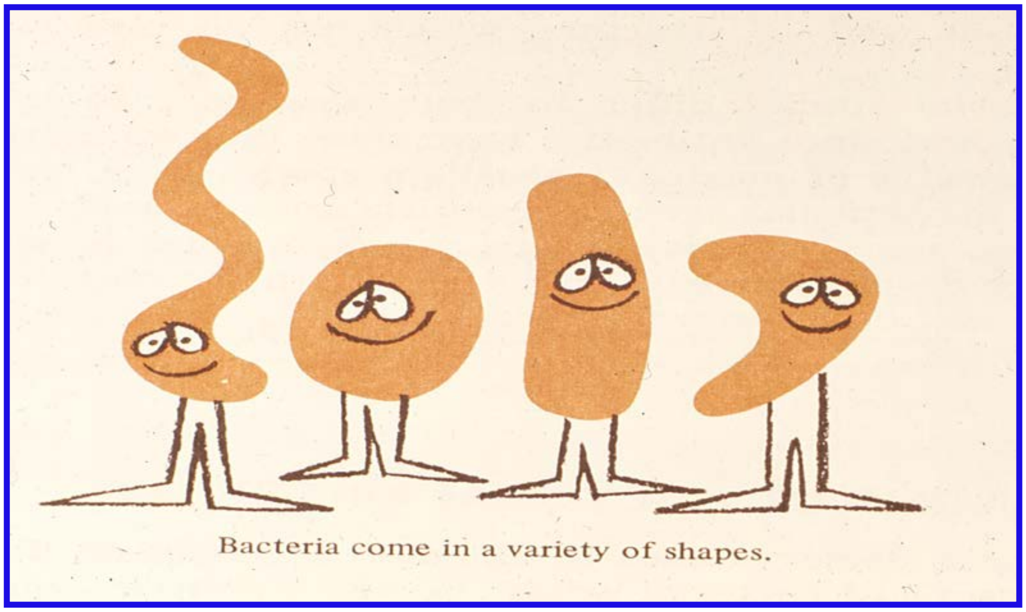
- They are single-celled organisms that can reproduce by binary fission (dividing into two).
- Most are aerobic chemoheterotrophs and they often consume bacteria.
- They are desirable in wastewater effluents because they act as polishers in consuming bacteria.

Rotifers and crustaceans:

- They are animals - aerobic, multicellular chemoheterotrophs.
- **Rotifers** drive its name from the apparent rotating motion of two sets of cilia on its head. The cilia provide mobility and a mechanism of creating food. Rotifers consume bacteria and small particles of organic matter.

– Crustaceans, a group of that includes shrimp, lobster, and barnacles, are characterized by their shell structure. They are a source of food for fish and are not found in wastewater treatment system to any extent except in underloaded lagoons.
– Crustaceans' presence is an indicative of a high level of dissolved oxygen and a very low level of organic matter.

4.4 Bacterial Biochemistry

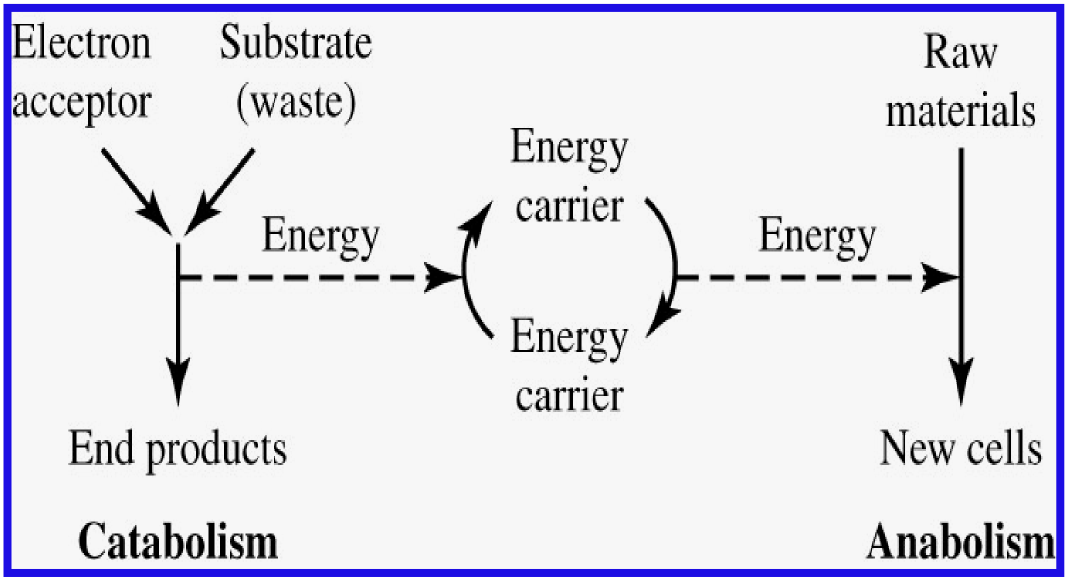


Heterogeneous microorganism population employed in wastewater treatment

- **Metabolism:** the general term that describes all of the chemical activities formed by a cell is metabolism. Metabolism is divided into two parts: *Catabolism* and *Anabolism*.

  – **Catabolism:** includes all the biochemical processes by which a substrate is degraded to end products with the release of **energy.** In wastewater treatment, the substrate is oxidized, and the oxidation process releases energy that is transferred to an energy carrier which stores it for future use by bacteria.

  – **Anabolism:** includes all the biochemical processes by which a bacterium synthesizes new chemical compounds needed by cells to live and reproduce. The synthesis process is driven by the energy that was stored in the energy carrier.



General scheme of bacterial metabolism

4.5 Decomposition of Wastes

The type of electron acceptor available for catabolism determines the type of decomposition: The types are:

4.5.1 Aerobic

- **Electron Acceptor** = O_2
- **End Products** = CO_2 and H_2O and Bacterial Cells
- **Odor Potential** = Low
- **Ecosystem** = Healthy Systems
- **WWT** = Dilute Wastes

- **Aerobic Decomposition:** molecular oxygen (O_2) must be present as the terminal electron acceptor for decomposition to proceed by aerobic oxidation. In natural water bodies, oxygen is measured as DO. In this case, chemical end products are primarily **CO2, water**, and **new cell** material.

4.5.2 Anoxic

- **Electron Acceptor** = NO_3^-
- **End Products** = N_2 and CO_2 and H_2O and Bacterial Cells
- **Odor Potential** = Low
- **Ecosystem** = Upper Hypolimnion
- **WWT** = Denitrification

- **Anoxic Decomposition:** when microorganisms can use nitrate (NO_3^-) as the terminal electron acceptor in the absence of molecular oxygen. Oxidation by this route is called denitrification. The end products from denitrification are gas, CO_2, water, and new cell material.

4.5.3 Anaerobic

- **Electron Acceptor** = CO_2 and organic compounds
- **End Products** = CH_4 and CO_2 and H_2O and bacterial cells
- **Odor Potential** = High
- **Ecosystem** = Sediments
- **WWT** = Sludge Stabilization

- **Anaerobic Decomposition:** molecular oxygen and nitrate must not be present as terminal electron organic acceptors. Sulfate compounds that (SO_4^{2-}), CO_2, and can be reduced serve as terminal electron acceptors. The reduction of sulfate results in the production of hydrogen sulfide (H_2S) and an equally odiferous organic group of sulfur compounds called mercaptans.
- The anaerobic decomposition (fermentation) of organic matter is considered to be a two-step process.

  ✓ In the first step, complex organic compound is fermented to low-molecular-weight fatty acids (volatile acids).

  ✓ In the 2nd step, the organic acids are converted to methane. CO_2 serves as the electron acceptor.

- The major end products of this decomposition are **carbon dioxide, methane,** and **water**. The additional end products are ammonia, hydrogen sulfide, and mercaptans.

Waste decomposition end products

	Representative end products		
Substrates	**Aerobic decomposition**	**Anoxic decomposition**	**Anaerobic decomposition**
Proteins and other organic nitrogen compounds	Amino acids Ammonia → nitrites → nitrates Alcohols, Organic acids } → $CO_2 + H_2O$	Amino acids Nitrates → nitrites → N_2 Alcohols, Organic acids } → $CO_2 + H_2O$	Amino acids Ammonia Hydrogen sulfide Methane Carbon dioxide Alcohols Organic acids
Carbohydrates	Alcohols, Fatty acids } → $CO_2 + H_2O$	Alcohols, Fatty acids } → $CO_2 + H_2O$	Carbon dioxide Alcohols Fatty acids Methane
Fats and related substances	Fatty acids + glycerol Alcohols, Lower fatty acids } → $CO_2 + H_2O$	Fatty acids + glycerol Alcohols, Lower fatty acids } → $CO_2 + H_2O$	Fatty acids + glycerol Carbon dioxide Alcohols Lower fatty acids Methane

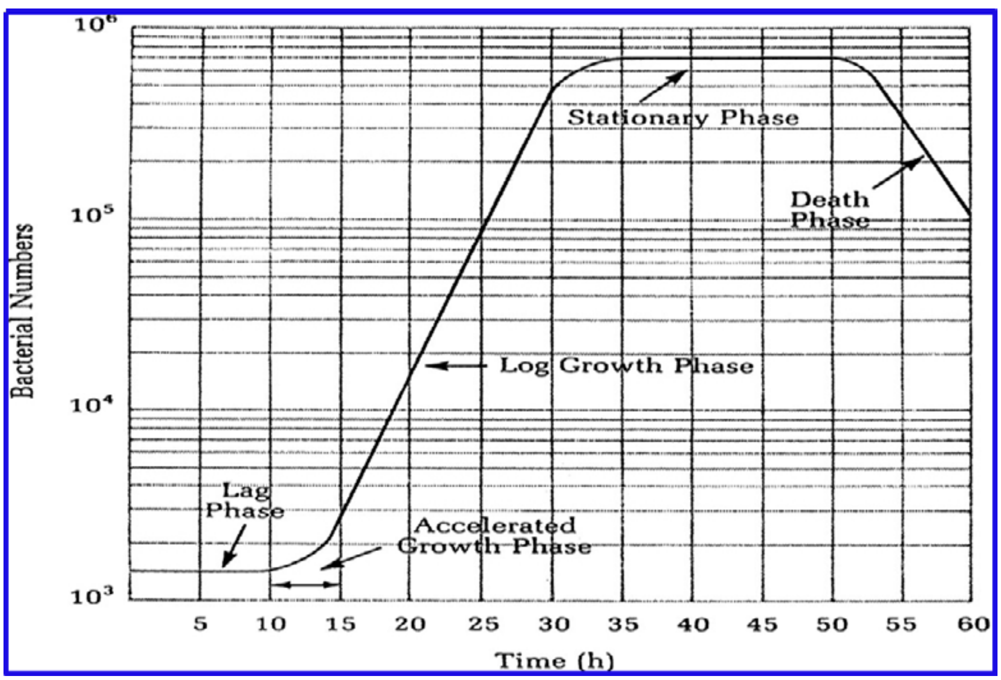
Source: After Pelczar and Reid, *Microbiology*, New York: McGraw-Hill, 1958.

Waste decomposition end products

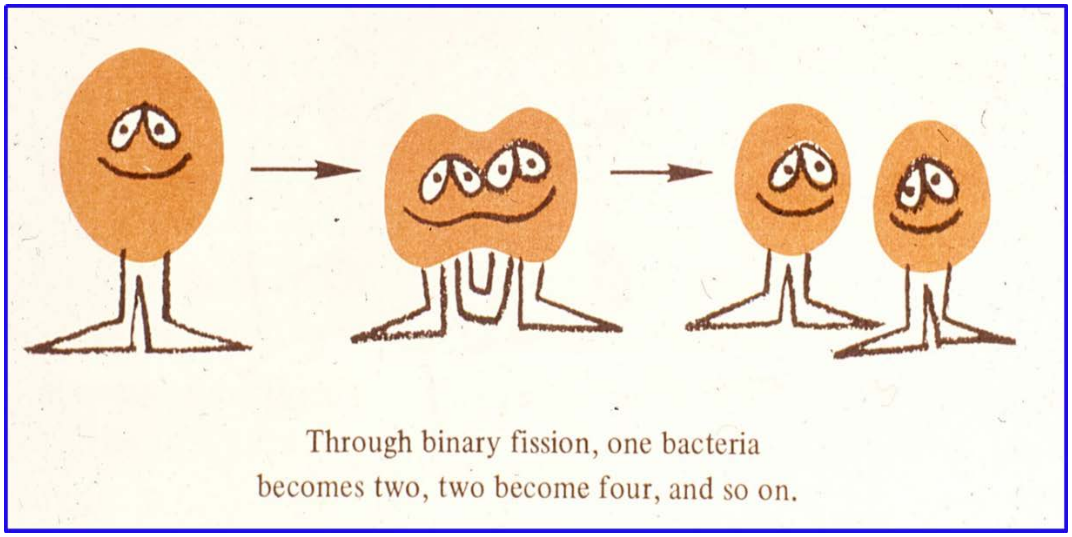
4.6 Population Dynamics - Bacterial growth requirements

The following list summarizes the major requirements that must be satisfied:

1. A terminal electron acceptor.
2. Macronutrients:
   a. Carbon to Build Cells
   b. Nitrogen to Build Cells
   c. Phosphorous for ATP (Energy Carrier) And DNA
3. Micronutrients
   a. Trace elements
   b. Vitamins are required by some bacteria
4. Appropriate environment
   a. Moisture
   b. Temperature
   c. pH



Bacterial growth in a pure culture "log-growth curve"



Microorganisms reproduce by binary fission

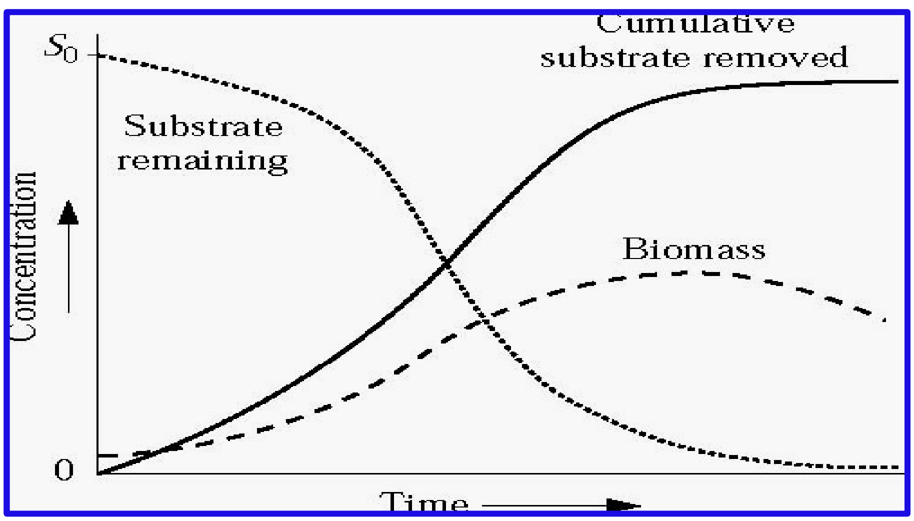
The population of bacteria (**P**) after the **nth** generation is given by the following expression:

$$P = P_o(2)^n \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Where, **Po** is the initial population at the end of accelerated growth phase. Taking log both sides, we get:

$$\log P = \log P_o + n \log 2 \ldots\ldots\ldots (2)$$

$$\therefore n = \frac{\log\left(\frac{P}{P_0}\right)}{\log(2)} = 3.32 \log\left(\frac{P}{P_0}\right) = 1.44 \ln\left(\frac{P}{P_0}\right) \ldots\ldots\ldots (3)$$



Variation of substrate removal, remaining, and biomass production with time

Example 1: If the population of microorganisms is $3.0x10^5$ at time t_0 and 36 hours later it is $9.0x10^8$, how many generations have occurred.

Example 1: Solutions

Given, $P_0 = 3.0x10^5$; $P = 9.0x10^8$

Using Eqs.(2) & (3) $\log P = \log P_o + n \log 2$

$$n = \frac{log\,P - log\,P_0}{log\,2} = 3.32\,log\left(\frac{P}{P_0}\right)$$
$$= 3.32\,log\left(\frac{9.0 \times 10^8}{3.0 \times 10^5}\right) = 11.54 \approx 12 \text{ generations}$$

The rate at which cell divide is the specific growth, μ. It is measured as generations per unit time, or the inverse of the doubling time.

Without die-off, the bacterial cells accumulate in the batch culture at an exponential rate (log-growth phase), which can be defined as:

$$\frac{dX}{dt} = \mu X \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

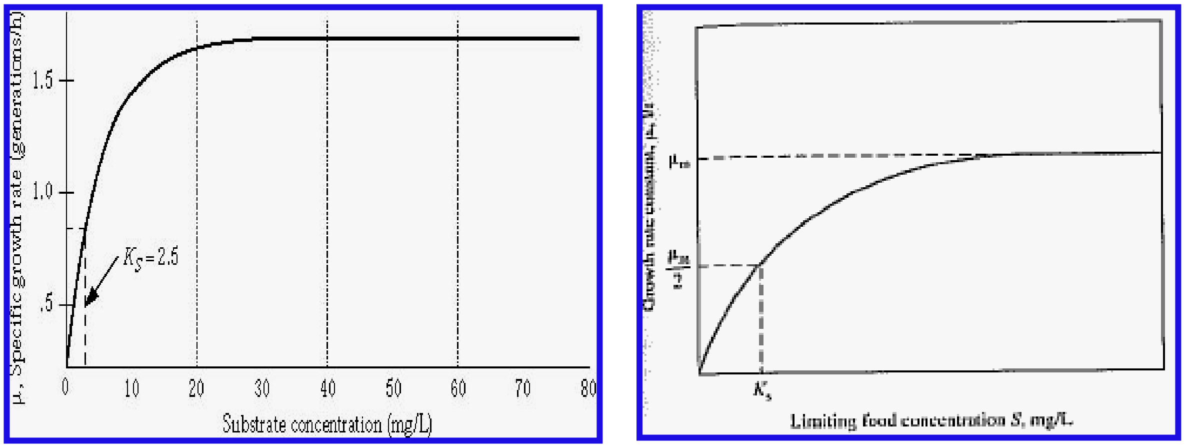
where, X = concentrations of biomass, mg/L
μ = specific growth rate, t^{-1}; t = time

The relationship between bacterial growth rate and substrate concentration are formulated according to Monod equation (1942):

$$\mu = \mu_m \frac{S}{K_s + S} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

where, μ = specific growth rate, t^{-1}
μ_m = maximum specific growth rate, t^{-1}
S = concentration of substrate in solution, mg/L
Ks = half-velocity constant, mg/L (i.e., substrate concentration at which $\mu = ½\,\mu_m$)

The growth of biomass follows a hyperbolic function as in the Figure.



Monod half-velocity constant (hypothetical example)

Substituting Eq.(5) into Eq.(4), we get:

$$\frac{dX}{dt} = \frac{\mu_m SX}{K_s + S} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6a)$$

Equation (6a) assumes only growth of microorganisms and does not consider natural die-off. It is generally assumed that the death or decay of biomass is a first-order expression in biomass and hence Eq.(6a) expanded to:

$$\frac{dX}{dt} = \frac{\mu_m SX}{K_s + S} - k_d X \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6b)$$

where, k_d = endogenous decay rate, t^{-1}

If all of the food in the system were converted to biomass, the rate of food utilization (dS/dt) would equal to the rate of biomass production. Because of the inefficiency of the conversion process, the rate of food utilization will be greater than the rate of biomass production, so

$$-\frac{dS}{dt} = \frac{1}{Y}\frac{dX}{dt} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

where, ***Y*** = decimal fraction food mass converted to biomass $=$ yieldcoefficient $\times \frac{\text{mg/Lbiomass}}{\text{mg/L food utilized}}$

Combining equation, 4, 6, and 7, we get,

$$-\frac{dS}{dt} = \frac{1}{Y}\frac{\mu_m SX}{K_s + S} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Equations 6 and 8 are a fundamental part of the development of the design equations for wastewater treatment processes.

5. Characteristics of Domestic Wastewater

5.1 Physical Characteristics of Domestic Wastewater

Fresh, aerobic, domestic wastewater - have the odor of kerosene or freshly turned earth. It is typically gray in color. Temperature - 10 to 20°C. Aged, septic sewage - considerably more offensive to the olfactory nerves and provide the characteristic of rotten-egg odor of H2S, and mercaptans. It is black in color. Temperature - 10 to 20°C.

5.2 Chemical Characteristics of Domestic Wastewater

BOD_5 - Amount of oxygen used by microorganisms to degrade organic matter in 5-day.

COD - amount of oxygen equivalent of organic matter that can be oxidized by a strong chemical oxidizing agent (potassium dichromate) in an acid medium. COD > BOD_5 as more compounds can be oxidized chemically than can be oxidized biologically and BOD_5 does not equal to ultimate BOD.

Total Kjeldal nitrogen (TKN)- is a measure of the total organic and ammonia nitrogen in the wastewater.

Parameters	Weak	Medium	Strong (all mg/L)
BOD5 (as O_2)	100	200	300
COD (as O_2)	250	500	1,000
TKN (as N)	20	40	80

5.3 Characteristics of Industrial Wastewater

The characteristics and levels of pollutants vary significantly from industry to industry.

EPA has grouped the pollutants into three categories: (1) Conventional, (2) Nonconventional, and (3) Priority pollutants.

Conventional Pollutants

- BOD
- Total Suspended Solids (TSS)
- Oil and grease
  - Oil (animal, vegetable)
  - Oil (mineral)
- pH

Nonconventional Pollutants:

- Ammonia (as N)

- Chromium VI (hexavalent)
- COD
- COD/BOD_7
- Fluoride (F)
- Manganese (Mn)
- Nitrate (as N)
- Organic Nitrogen (as N)
- Pesticide active ingredients (PAI)
- Total Phenols
- Total Phosphorous
- Total Organic carbon (TOC)

Examples of industrial wastewater concentrations for BOD_5 and SS.

Industry	**BOD_5, mg/L**	**SS, mg/L**
Ammunition	50 – 300	70 – 1,700
Fermentation	4,500	10,000
Slaughterhouse (cattle)	400 – 2,500	400 – 1,000
Pulp and Paper (kraft)	100 – 350	75 – 300
Tannery	700 – 7,000	4,000 – 20,000

6. Wastewater Treatment Standards

- In Public Law 92-500, the Congress required municipalities and industries to provide secondary treatment before discharging wastewater into natural water bodies.

- EPA established a definition of secondary treatment based on 3 wastewater characteristics: BOD_5, SS, and pH.

Characteristics of Discharge	**Average Monthly Contribution**	**Average Weekly Contribution**
BOD_5, mg/L	30	45
Suspended Solids (SS - mg/L)	30	45
pH (Hydrogen ion conc.)	6.0 – 9.0 at all times	
$CBOD_5$ (mg/L)	25	40

- PL 92-500 also directed EPA to establish a permit system called National Pollutant Discharge Elimination System (NPDES) permit.

- Under **NPDES** program, all facilities that discharge pollutants from any point source into waters of the US are required to obtain a **NPDES** permit.

7. On-Site Disposal

Individual On-Site Disposal System

Without Water Carriage

- Pit Privy
- Vault Toilet
- Chemical Toilet

With Water Carriage

- Septic Tank and File Field
- BLWRS Barriered-landscape water-renovation system)

7.1 Without Water Carriage

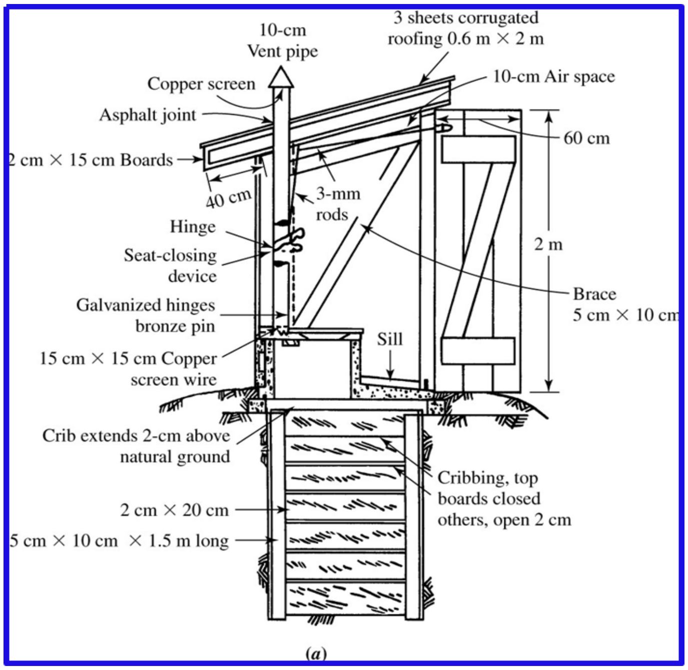
- **The Pit Privy**:

  The principle of operation of pity privy is that the liquid materials percolate into the soil through the cribbing and the solids "dry out". A fair size pit should last a family of four about 10 years. Rainwater is to be prevented from entering the pit. A cup of kerosene at weekly intervals discourages mosquito breeding, and odor can be reduced by the use of a cup of hydrated lime. Disinfectants should never be used.

- **The Vault Toilet:**

  This the modern version of pit privy. Its construction is the same as that of the pity privy with exception that the pit is formed as a watertight vault. A special truck (fondly called a "honey wagon") is used to pump out the vault at regular intervals. These are much more odiferous than the old pit privy because of the liquefying action of the bacteria and incipient anaerobic decomposition.

The pit privy:



Cross-section of the construction details for the pit privy

- **The Chemical Toilet:**

The airplane toilet, the coach-bus toilet, and the self-contained toilets of recreation vehicles are all versions of the chemical toilet. The essence of the system is a strong disinfectant chemical used to carry the waste to a holding tank and render it inoffensive until it can be pumped from the holding tank.

The chemical used must be selected with the consideration of the impact on the treatment system which ultimately must receive it. The chemical toilet has not found widely acceptance in permanent installations. This is due to the cost of the chemical and to the impracticality of maintenance.

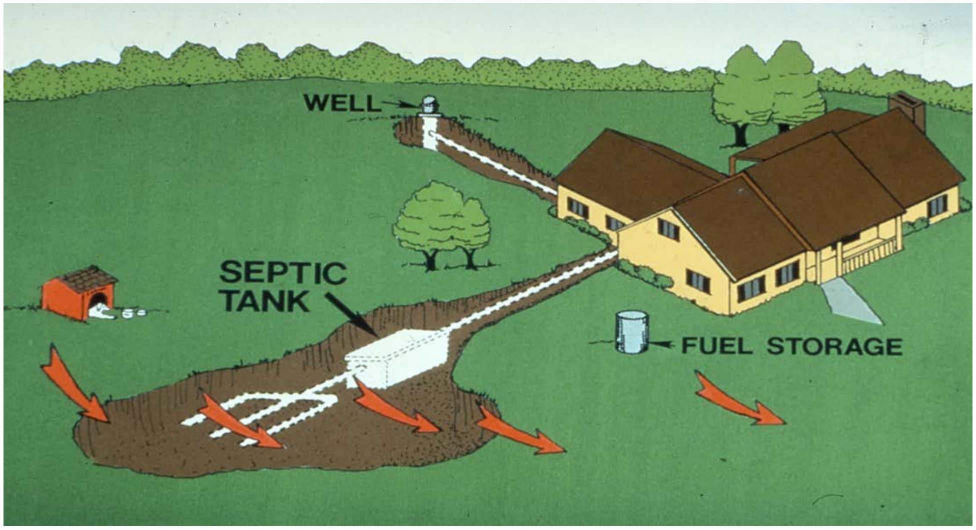
7.2 With Water Carriage

- **Septic tanks and tile fields:**
- **Barriered-landscape water-renovation system (BLWRS):**

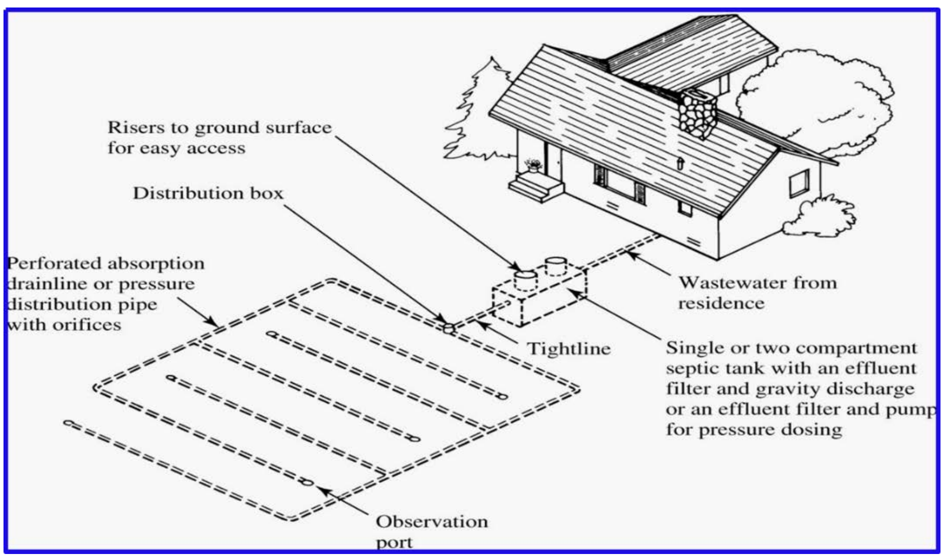
Septic Tanks and Tile Fields

- The septic tank and tile field are a unit. Neither part will function as intended without the other.
- The main function of septic tank is to remove large particles and grease which would otherwise clog the tile field.
- 24-hr hydraulic retention at the design flow.
- Minimum size, not less than 4.0 m3 in volume.
- Tile field is to hold the liquid from the septic tank while it seeps into the soil.

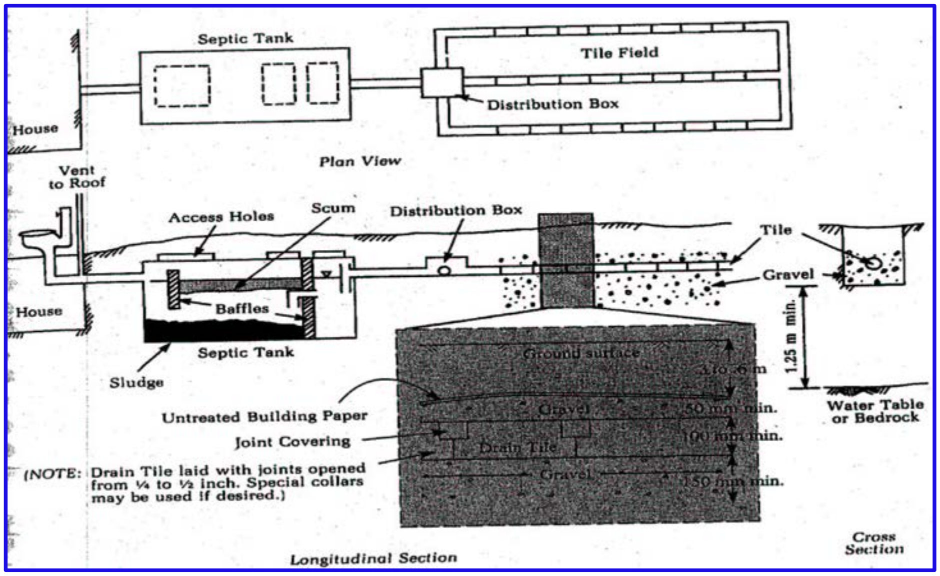
- Tile field must be located > 30 m from any well, surface water, footing drain, or storm drain.
- Tile field must be located at least 3 m from any property line.



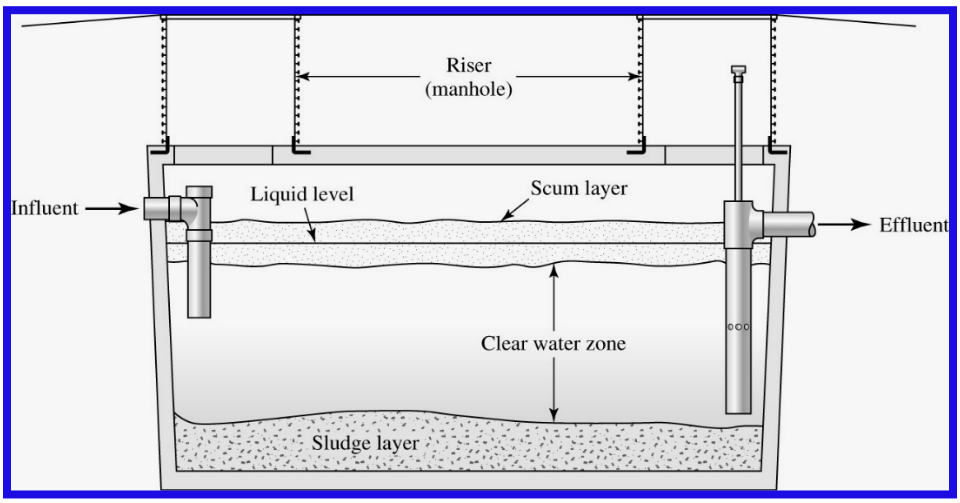
Septic tank and tile field system showing placement with respect to well and groundwater flow



Schematic of a conventional septic tank



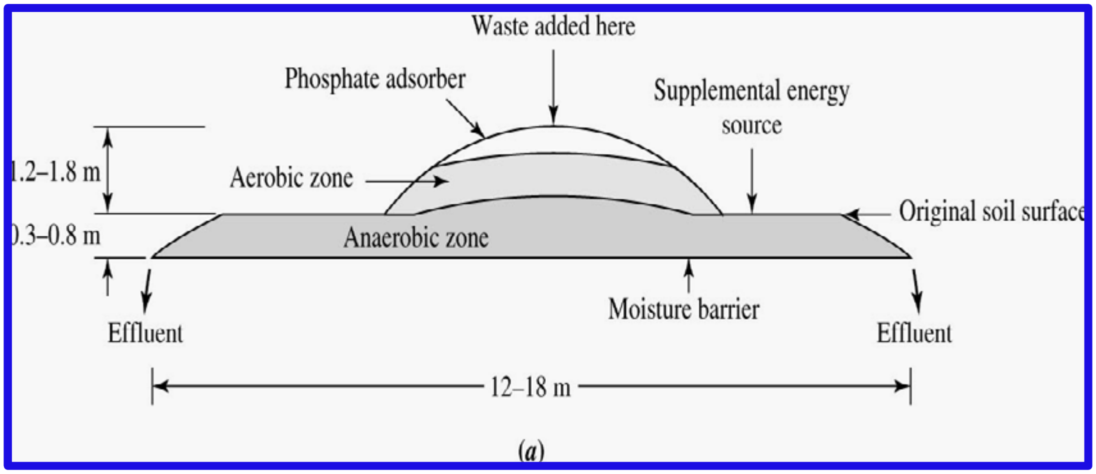
Schematic layout of a septic tank and tile field



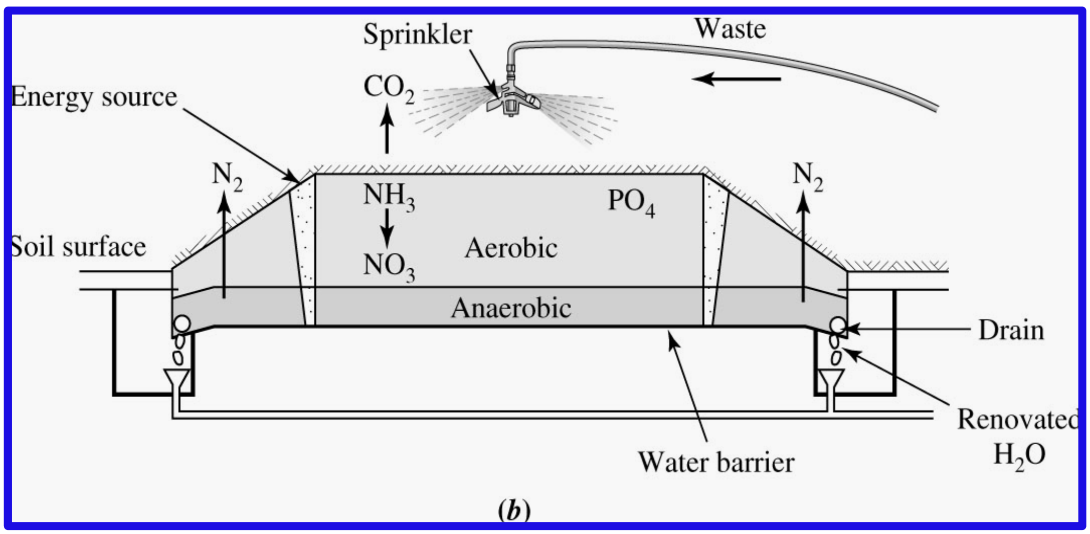
Definition sketch for the sludge, clear water, and scum zones that form in a septic tank

Barriered-landscape water-renovation system (BLWRS):

The BLWRS consists of a mound of soil underlain by an impervious water barrier.



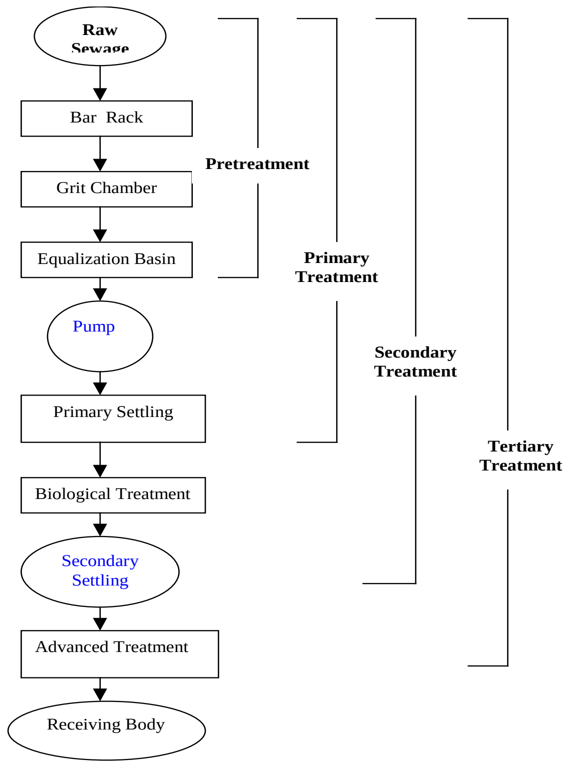
(a) Common dimension of BLWRS



(b) water chemistry change in a BLWRS

8. Municipal Wastewater Treatment Systems

Degrees of Treatment

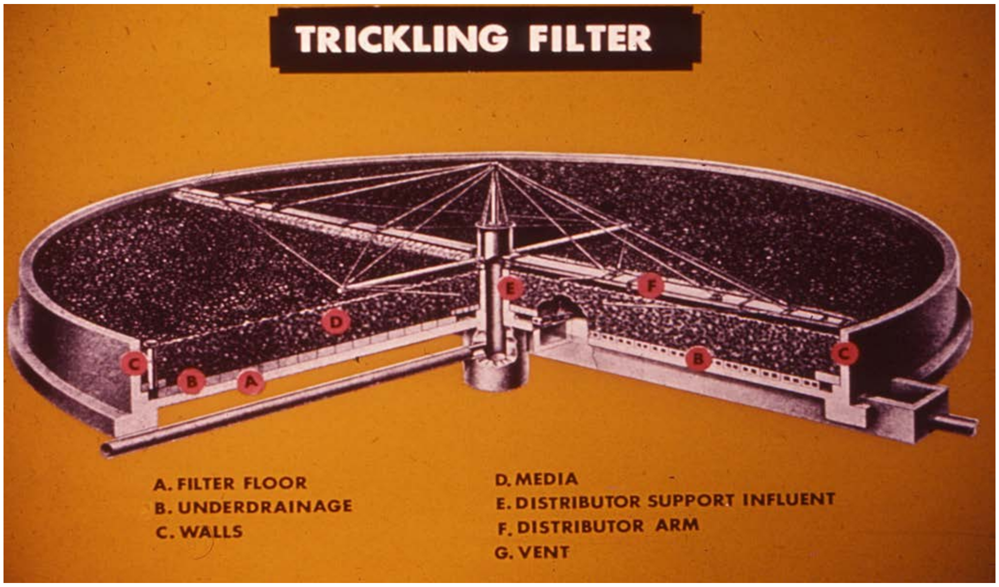


8.1 Biological Treatment

Aerobic wastewater treatment systems use oxygen-feeding bacteria, protozoa, and other specialty microbes to clean water as opposed to anaerobic systems that do not need oxygen. These systems optimize the naturally occurring process of microbial decomposition to break down wastewater contaminants so they can be removed.

- Trickling Filter (Attached Growth)
- Activated Sludge (Suspended Growth)
- Rotating Biological Contactor
- Lagoons (Oxidation Pond)

Trickling Filter

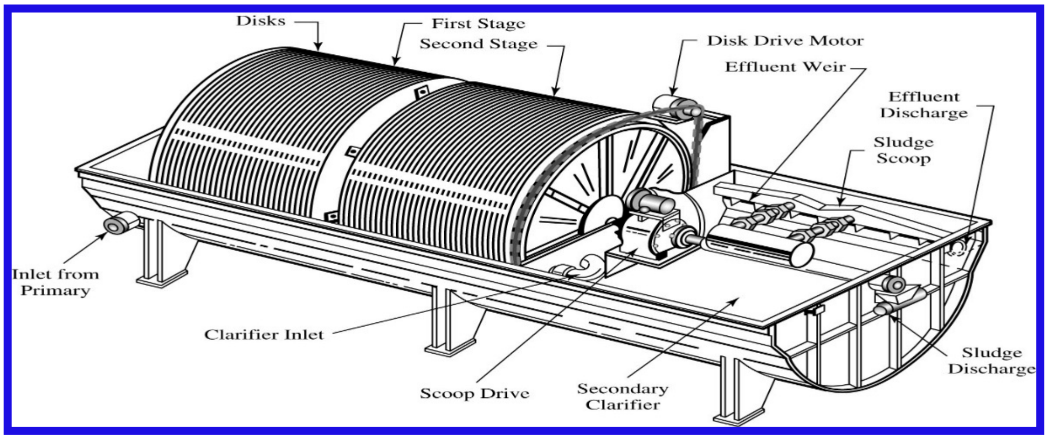


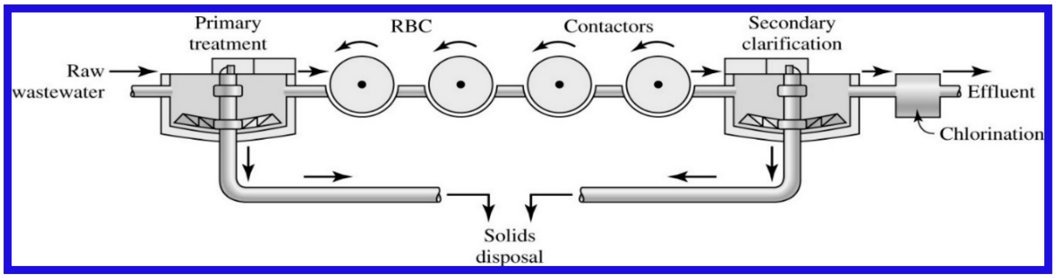
WWT SYSTEMS - Trickling Filter

Activated Sludge

Activated Sludge Aeration Tank

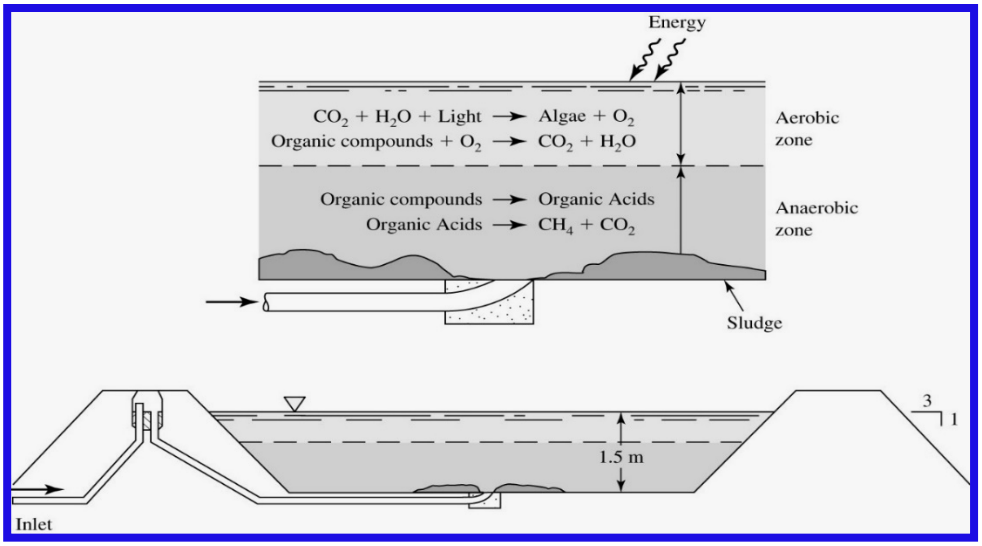
Rotating Biological Container





Rotating Biological Contactor (RBC) and Process Arrangement

Lagoons



Schematic diagram of facultative lagoon pond relationships

The alternatives for municipal wastewater treatment fall into 3 major categories:

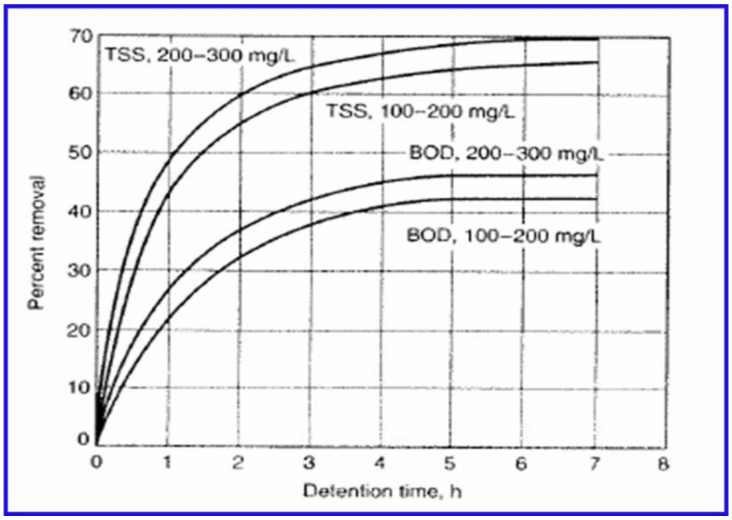
1. Primary treatment
2. Secondary treatment, and
3. Advanced treatment

It is commonly assumed that each of the "degrees of treatment" assumed to include the previous steps. For example, primary treatment is assumed to include the pretreatment processes: bar rack, grit chamber, and equalization basin. Likewise, secondary treatment is assumed to include all the processes of primary treatment: bar rack, grit chamber, equalization basin, and primary settling tank.

8.2 Primary Treatment

The purpose of primary treatment is to provide protection to the wastewater treatment plant (WWTP) equipment that follows.

- In some old municipal plants, the equalization basin step may not be included.
- The major goal of primary treatment is to remove those pollutants that will either settle or float.
- It will typically remove about 60% of SS in raw sewage and 35% of the BOD_5. Soluble pollutants are not removed.
- Typical % removal of BOD_5 and TSS can also be estimated from the graph below.



8.3 Secondary Treatment

- It may remove more than 85% of BOD_5 and SS, but it does not remove significant amount of nitrogen, phosphorous, or heavy metals, not does it completely remove pathogenic bacteria and viruses.
- When secondary treatment is not adequate, additional treatment processes are applied to the secondary effluent to provide advanced wastewater treatment (AWT).
- The four biological treatment systems, mentioned in Section 8.1, are employed as secondary treatment processes.

8.4 Advanced Treatment

- These processes may include chemical treatment and filtration of the wastewater - much like adding a typical water treatment plant to the tail end of a secondary plant.
- Some of these processes can remove as much as 99% of BOD_5, SS, phosphorous, and bacteria, and 95% of nitrogen.
- Most of the impurities are removed from the wastewater as a solid, that is sludge. Some organics are broken down into harmless CO_2 and water.

9. Unit Operations of Pretreatment

Several devices and structures are placed upstream of the primary treatment operation to provide protection to the WWTP equipment. These devices and the structures are classified as pretreatment because they have little effect in reducing BOD_5. The following are the typical devices or structures used in primary treatment.

- Bar Racks
- Grit Chambers
- Comminutors
- Equalization

9.1 Bar Racks

- Typically, the first device encountered by the wastewater entering the plant.
- The primary purpose of the rack is to remove large objects that would damage or foul pumps, valves, and other mechanical equipment.
- Rags, logs, and other objects that find their way into the sewer are removed from the wastewater on the racks.
- In modern WWTPs, the racks are cleaned mechanically.
- The materials are stored in a hopper and sent to a sanitary landfill at regular intervals.
- Bar racks (or bar screens) may be categorized as trash racks, manually cleaned racks, and mechanically cleaned racks.
- Trash racks have openings that range from 40 to 150 mm that is designed to remove such as logs.
- Manually cleaned racks have openings that range from 25 to 50 mm with a channel approach velocity range from 0.3 to 0.6 m/s.
- Mechanically cleaned racks have openings that range from 5 to 40 mm with a channel approach velocity range from 0.6 to 1.2 m/s.
- Minimum velocities of 0.3 to 0.6 m/s are necessary to prevent grit accumulation.
- Regardless of the type of rack, two channels are provided to allow one to be taken out of service for cleaning and repair.

Bar rack (right) and comminutor (left) in a wastewater Treatment Plant

9.2 Grit Chambers

- Inert dense materials, such as sand, broken glass, silt, and pebbles, are called grit, need to be removed from wastewater as they abrade pumps and other mechanical devices causing undue wear.
- In addition, they have tendency of settling in corners and bends, reducing flow capacity, and ultimately, clogging pipes and channels.
- There are 3 types of grit removal devices:
  - Velocity Controlled
  - Aerated, and
  - Constant-Level Short-Term Sedimentation Basin

Velocity Controlled Grit Chamber. Left Chamber in Service. Right Chamber Out of Service.

Grits

9.3 Comminutors

- Devices that are used to macerate wastewater solid (rags, paper, plastic, and other materials) by revolving cutting bars are called comminutors.
- These devices are placed downstream of the grit chambers to protect the cutting bars from abrasion.
- They are used as a replacement for downstream bar rack but must be installed with a hand-cleansed rack in parallel in case they fail.

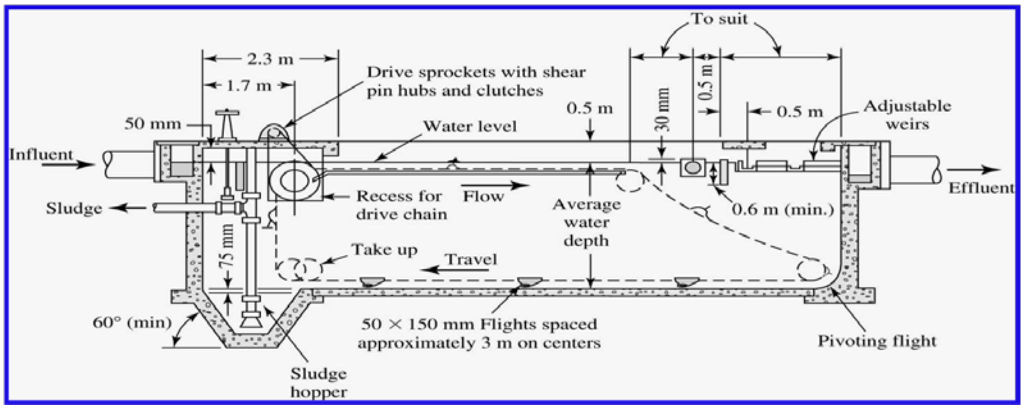
9.4 Equalization

- Flow equalization is not a treatment process, but a technique that can be used to improve the effectiveness of both secondary and advanced wastewater treatment processes.
- The purpose of flow equalization is to dampen the variations of flows in different times of the day so that the wastewater can be treated as a nearly constant flow rate.
- It can significantly improve the performance of an existing plant and increase its useful capacity.
- In new plants, flow equalization can reduce the size and cost of the treatment units.
- Flow equalization is usually achieved by construction of large basins that collect and store wastewater flow and from which the wastewater is pumped to the treatment plant at a constant rate.
- These basins are normally located near the head end of the treatment works, preferably downstream of pretreatment facilities such as bar racks, grit chambers, and comminutors.
- Adequate aeration and mixing must be provided to prevent odors and solid deposition.
- The required volume of an equalization basin is estimated from a mass balance of the flow into the treatment plant with the average flow the plant is designed to treat.
- The theoretical basis is the same as that used to size a reservoir.

10. Primary Treatment

- With the screening completed and the grit removed, the wastewater still contains light organic suspended solids, some of which can be removed from the sewage by gravity in a sedimentation tank.
- The tank can be rectangular or round. The mass of the settled solids is called raw sludge. The sludge is removed from the sedimentation tank by mechanical scrapers and pumps.
- Floating materials, such as grease and oil, rise to the surface of the tank, where they are collected by a surface skimming system and removed from the tank for further processing.
- The primary sedimentation tank is characterized by Type II flocculent settling.
- The Stokes equation is not applicable as the flocculating particles are continually changing in size, shape, and when water is entrapped in the floc.
- There is adequate mathematical relationship that can be used to describe Type II settling.
- Typical rectangular tanks range from 15 to 100 m in length and 3 to 24 m in width.
- Common length-to-width ratio for design of new facilities range from 3:1 to 5:1.

- Side water depths range from 2 to 5 m. Typically the depth is 3.5 m.
- Circular tanks have diameters from 3 to 90 m.
- Side water depths range from 2.4 to 5 m.
- Hydraulic detention time in the sedimentation tanks ranges from 1.5 to 2.5 hours under average flow conditions. A 2.0-hour detention time is typical.



Primary Settling Tank.

- Literature provides values for detention time and overflow rate that can be obtained from column experiments.

Condition	Range ($m^3/m^2{\cdot}d$)	Typical ($m^3/m^2{\cdot}d$)
Primary clarification prior to secondary treatment		
Average flow	30–50	40
Peak flow	70–130	100
Primary clarification with waste-activated sludge return[a]		
Average flow	25–35	30
Peak flow	45–80	60

[a]In many designs the waste-activated sludge is returned to the primary sedimentation basin. The objective is to concentrate the secondary sludge in the primary sedimentation basin.
Note: 1 $m^3/m^2{\cdot}d$ = 24.5424 gpd/ft^2.

- Literature – Detention time and overflow rate

	Detention Period (h)					
Overflow Rate ($m^3/m^2 \cdot d$)	2.0-m Depth	2.5-m Depth	3.0-m Depth	3.5-m Depth	4.0-m Depth	4.5-m Depth
30	1.6	2.0	2.4	2.8	3.2	3.6
40	1.2	1.5	1.8	2.1	2.4	2.7[a]
50	1.0	1.2	1.4	1.7	1.9	2.2
60	0.8	1.0	1.2	1.4	1.6	1.8
70	0.7	0.9	1.0	1.2	1.4	1.5
80	0.6	0.8	0.9	1.1	1.2	1.4

[a] A 4.5-m deep sedimentation basin having an overflow rate of 40 $m^3/m^2 \cdot d$ (982 gpd/ft^2) will provide a detention period of 2.7 h. This may be the most desirable design condition.

- Circular tanks have diameters from 3 to 90 m.
- Side water depths range from 2.4 to 5 m.
- Hydraulic detention time in the sedimentation tanks ranges from 1.5 to 2.5 hours under average flow conditions. A 2.0-hour detention time is typical.

Primary settling tank

Primary settling tanks in foreground followed by aeration tanks and circular secondary settling tanks

Example 2: Determine the surface area of a primary settling tank sized to handle a maximum hourly flow of 0.570 m^3/s at an overflow rate of 60.0 m/d. If the effective tank depth is 3.0 m, what is the effective theoretical detention time?

If the theoretical detention is increased by 50%, what would be the surface area of the settling tank?

Example 2: Solutions

Given Q = 0.570 m^3/s x 86,400 s/d = 49,248 m^3/d

Overflow velocity, v_0 = 60 m/d

Tank depth, d = 3.0 m

Surface area,

$$A_s = \frac{Q}{V_0} = \frac{49{,}248\text{m}^3/\text{d}}{60\text{m/d}} = 820.8\text{m}^2$$

The effective theoretical detention time,

$$t = \frac{\forall}{Q} = \frac{A_s d}{Q} = \frac{820.8m^2 \times 3m}{49{,}248m^3/d} = 0.05 \text{ days} = 1.2 \text{ hours} \quad \text{Ans.}$$

If the detention time is increased by 50%, i.e. the new detention time, t = 1.20 hours x 1.50 = *1.8 hours = 0.075 days.*

New volume, $\forall = Qxt = 49{,}248\ m^3/d \times 0.075\ day = 3{,}693.6\ m^3$

New surface area, $As = \forall/d = 3693.6\ m^3/3\ m = 1{,}231.2\ m^2$

$\therefore$ New surface area is *(1,231.2-820.8)/820.8x100% =50%* higher than the initial surface area.

Example 3: You are asked to ***design a water treatment plant settling tank (length, width, depth, and check for volume provided)*** after coagulation for the City of Austell. The design flow is 0.85 m^3/s and the overflow rate, and the detention time found from the column test are 35 m/d and 94.5 min, respectively. The City of Austell is asking you to keep the tank length maximum of 50 m and length-to-width ratio maximum of 4 to 1.

Example 3: Solutions

Given,							
Design flow, Q =		0.85	m^3/s x (86,400 s/ 1 day)		73,440	m^3/d	
Over flow rate, v_o =		35	m/d				
Detention time, t =		94.5	min.				
L_{max} =		50	m				
L: W ratio =		4	: 1	$\therefore$ L =	4W		
Suface area, $SA = Q/v_0$ =		(73440 m^3/d)/(35 m/d)					
	=	2,098.29	m^2				
Total volume required, $\forall_{req}$ = Q x t =			(0.85 m^3/s) x (94.5 min.) x (60 s/1 min.)				
		=	4819.5	m^3			
Depth, d =	$\forall$/SA =	4819.5 m^3 /2098.29 m^2					
	=	2.30	m plus sludge depth and freeboard				

Example 3: Solutions – cont'd

Tank Dimensions according to given parameters inputted from above:								
Assuming Number of Tank =		1	Probable number of tanks necessary = SA /(Lmax x W) =				2098.29/(50x50/4) =	8
Surface area of each tank, A =		Volume/(d x1) =		4819.5/(2.3x1) =	2098.29	m^2		
Surface Area, A = L x W =		4W x W			W = (2098.29/4)^(1/2) =	22.90	m	
					and L = 4W = 4 x 22.9 =	91.61	m	
Check for Lmax:		Lmax is NOT OK, Change the number of tank =				8		
	Surface area for each tank, A =		4819.5/(2.3x8) =		262.29	m^2		
		W = (262.29/4)^(1/2) =			8.10	m		
	and	L = 4.5 W =			32.39	m		
		Lmax is OK.						
Check: Total volume provided =			32.39 x 8.1 x x 8 =		4819.50	m^3		
					>= Volume required, OK.			
P.S.: In practice, even number of tanks are provided. In this case you can provide 8 tanks. If the number is odd, add one more tank.								

<u>Example 4</u>: The influent suspended solids (SS) concentration of a primary settling tank is 435 mg/L. The average flow rate is 0.050 m^3/s. If the suspended solids removal efficiency is 60%, how much kilograms of suspended solids are removed in the primary tank each day?

<u>Example 4</u>: Solutions

Given, Initial SS = 435 mg/L,

$Q = 0.050\ m^3/s \times 86{,}400\ s/d = 4{,}320\ m^3/d$

SS removal efficiency = 60%

Mass of SS entering the primary tank

M_{ss} entered = 435 mg/Lx(1g/m^3/1 mg/L) x (4,320 m^3/d)x(1 kg/1000 g)

= 1,879.2 kg/day

M_{ss} removed = 1,879.2 kg/d x 0.6 =**<u>1,127.52 kg/day</u>**. **Ans.**

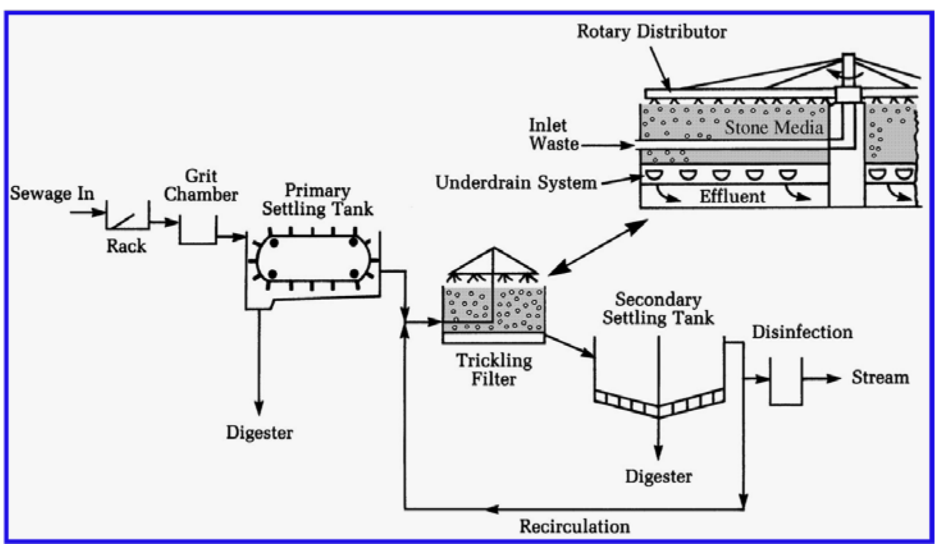
11. Unit Operations of Secondary Treatment

11.1 Overview

- The main purpose of secondary treatment is to remove the soluble BOD that escapes primary treatment and to provide further removal of SS.
- The basic ingredients needed for conventional aerobic secondary biologic treatment are the availability of many microorganisms, good contact between these organisms and the organic material, the availability of oxygen, the maintenance of favorable environment conditions (i.e., temperature, sufficient time for the organisms to work).
- The most common approaches that are used to achieve these basic needs are:
    - Trickling Filters,
    - Activated Sludge, and
    - Oxidation Ponds
- A process that does not fit precisely either in trickling filter or the activated sludge category but does employ principles of common to both is the Rotating Biological Contactors (RBCs).

11.2 Trickling Filters

- A trickling filter consists of a bed of coarse material, such as stone, slats, or plastic materials (media), over which the wastewater is applied. It is often called attached growth process.
- Trickling filters have been a popular biological treatment process. The most widely used design for many years was simply a bed of stones from 1 to 3 m deep through which the wastewater passed.
- The wastewater is typically distributed over the surface of the rocks by a rotating arm.
- Rock filter diameters may range up to 60 m.



Trickling-Filter Plant with Enlargement of Trickling Filter

- Trickling filters are classified according to the applied hydraulic and organic load.
- Hydraulic load is expressed as cubic meters of wastewater applied per day per square meter of bulk filter area ($m^3/d \cdot m^2$) or depth of water applied per unit of time (mm/s or m/d).
- Organic load is expressed as kg of BOD_5 per day per cubic meter of bulk filter volume ($kg/d.m^3$)
- An important element in trickling filter design is the provision for return of a portion of the effluent to flow through the filter that is called recirculation.
- The ratio of the return flow to the incoming flow is called the *recirculation ratio (r).*

The recirculation is practiced in stone filters for the following reasons:

- To increase contact efficiency by bringing the waste into contact more than once with active biological materials.
- To dampen variations in loadings over a 24-hr period. The strength of the circulated flow lags behind of the incoming wastewater. Thus, recirculation dilutes strong influent and supplement the weak influent.
- To raise DO of the effluent
- To improve distribution over the surface, thus reducing the tendency to clog and also to reduce filter flies.
- To prevent the biological slimes from drying out and dying during nighttime periods when flows may be too low to keep the filter wet continuously.

Trickling Filters – Design Equations

Empirical formulas have been developed to predict the efficiency of the filters based on the BOD load, the volume of filter media, and the recirculation. For a single stage filter or first stage of a two-stage filter, the efficiency is (NRC, 1946):

$$E_1 = \frac{1}{1 + 4.12\left(\frac{QC_{in}}{\forall F}\right)^{0.5}} \quad \ldots\ldots (9)$$

where, E_1 = fraction of BOD_5 removal for first stage at 20°C, including recirculation and sedimentation

Q = wastewater flow rate, m^3/s; C_{in} = influent BOD5, mg/L

$\forall$ = volume of filter media, m^3;   F = recirculation factor

The recirculation factor is

$$F = \frac{1 + R}{(1 + 0.1R)^2} \quad \ldots\ldots (10)$$

where, R = recirculation ratio = Q_r/Q.

Q_r = recirculation flow rate, m^3/s.

Q = wastewater flow rate, m^3/s

For the second stage filter, the efficiency is

$$E_2 = \frac{1}{1+\frac{4.12}{1-E_1}\left(\frac{QC_e}{\forall F}\right)^{0.5}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

where, E_2 = fraction of BOD_5 removal for second stage at 20^{o}C, including recirculation and sedimentation

E_1 = fraction of BOD_5 removed in the first stage

C_e = effluent BOD_5 from first stage, mg/L

The effect of temperature on the efficiency may be estimated from the following equation:

$$E_T = E_{20}\theta^{T-20} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

where, θ = 1.035 is used.

Example 5: Determine the concentration of the effluent BOD_5 for the two-stage trickling filter described below. The wastewater temperature is 17^{o}C. Assume the NRC equations apply.

Design flow = 0.0709 m^3/s; Influent BOD5 (after primary treatment) = 206 mg/L, Diameter of each filter = 25.0 m, Depth of each filter = 1.93 m, Recirculation flow rate for each filter = 0.0694 m3/s

Example 5: - Solutions

Given,	Q =	0.0709	m^3/s	C_{in} =	206	mg/L	diameter of each filter =	25	m
	Q_r =	0.0694	m^3/s	T =	17	°C	depth of each filter =	1.93	m

(a) Calculate the volume of each filter

$$\forall = \frac{\text{pi x 25 m^2 x 1.93 m}}{4} = \underline{947.39}\ m^3$$

(b) Calculate the recirculation ratio, R

$$R = \frac{Q_r}{Q} = \frac{0.0694\ m^3/s}{0.0709\ m^3/s} = \underline{0.979}$$

(c) Calculate the recirculation factor, F

$$F = \frac{Q_r}{Q} = \frac{1 + 0.979}{(1 + 0.1\ 0.979)^2} = \underline{1.6417}$$

(d) Calculate the efficiency of the 1st filter, E_1

$$E_1 = \frac{1}{1 + 4.12\left(\frac{QC_{in}}{\forall F}\right)^{0.5}} = \frac{1}{1 + 4.12\ (0.0709 \times 206/947.387 \times 1.6417)^{0.5}} = \underline{0.7147}$$

(e) Correct the efficiency, E_1 for temperrature

$$E_T = E_{20}(1.035)^{T-20}$$

E_{17} = 0.7147 (1.035) ^(17 - 20) = **0.6446**

(f) Calculate the effluent concentration from 1st stage

C_e = (1 - 0.6446)(206) = **73.21** mg/L

(g) Calculate the efficiency of the 2nd stage or 1st filter, E_2

$$E_2 = \frac{1}{1 + \frac{4.12}{1 - E_1}\left(\frac{QC_e}{\forall F}\right)^{0.5}} = \frac{1}{1 + [4.12/(1 - 0.6446)] \times [0.0709 \times 73.21/(947.387 \times 1.6417)]^{0.5}}$$

= **0.5989**

(h) Correct the efficiency, E_2 for temperrature

$$E_T = E_{20}(1.035)^{T-20}$$

E_{17} = 0.5989 (1.035) ^(17 - 20) = **0.5402**

(i) Calculate the fFinal effluent BOD_5

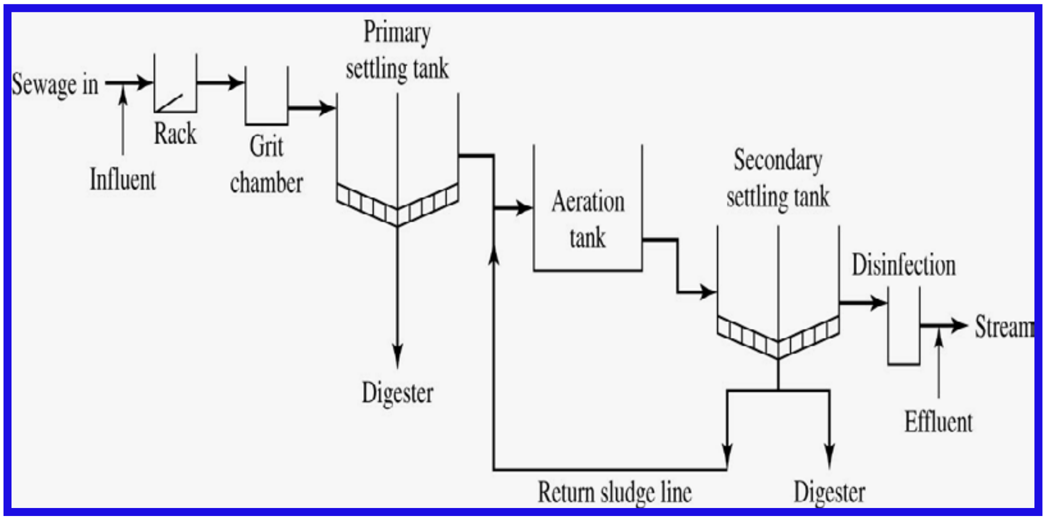
Effluent BOD_5 = (1 - 0.5402) x 73.21 = **33.67** mg/L **ANS.**

11.3 Activated Sludge

- The activated sludge process is a biological wastewater treatment technique in which a mixture of wastewater and biological sludge (microorganisms) is agitated and aerated.
- It is often called suspended growth process.
- The biological solids are subsequently separated from the treated wastewater and returned to the aeration process as needed.
- In this process, microorganisms are mixed thoroughly with the organics under conditions that simulate their growth through use of the organics as food.
- As the microorganisms grow and are mixed by the agitation of the air, the individual organisms clump together (flocculate) to form an active mass of microbes (biologic floc) called ***activated sludge***.
- In practice, wastewater flows continuously into an aeration tank where **air is injected to mix the activated sludge** with the wastewater and to supply the oxygen needed for the organisms to break down the organics.
- The mixture of activated sludge and wastewater in the aeration tank is called ***mixed liquor***.

- The mixed liquor flows from the aeration to a secondary clarifier where the activated sludge is settled out. Most of the settled sludge is returned to the aeration tank (called *return sludge*) to maintain the high population of microbes that permits rapid breakdown of the organics.

- In conventional activated sludge systems, the wastewater is typically aerated for 6–8 hrs (rectangular basins).
- About 8 m^3 of air (*1.68* m^3 *of* O_2) is provided for each m^3 of wastewater treated.
- The typical sludge age of an effective system is 20-30 days but should not be < 10 days and > 50 days.
- The biomass *mixed liquor suspended solids* (MLSS) concentration should not be <1,000 mg/L and >5,000 mg/L; a higher concentration could result in the clarifier failing.



Conventional Activated Sludge Plant

Activated Sludge Aeration Tank

End of Tank

Activated Sludge - Terms used

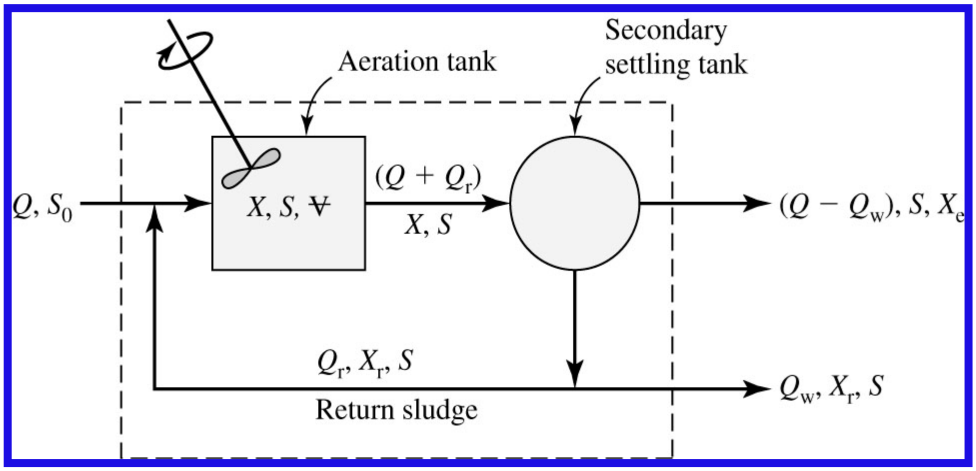
- Wasting: a portion of the microorganisms that is discarded from the process is called wasting.
- Waste Activated Sludge (WAS): The discarded microorganisms are called waste activated sludge.
- Mean Cell Residence Time (θ_c) / Solids Retention Time (SRT) / Sludge Age: the average amount of time that microorganisms are kept in the system is called SRT.
- Completely Mixed (CSTR) Activated Sludge Process:
- Plug Flow with Recycle Waste Activated Sludge Process:

Activated Sludge – Completely Mixed (CSTR) Activated Sludge Process:

– The design formula for the **CSTR** activated sludge process are a mass balance application of the equilibrium used to describe the kinetics of microbial growth.

– Two mass balances are required to define the design of the **CSTR**:
  - Biomass
  - Food (Soluble BOD_5)

- Under steady-state conditions, the mass balance for biomass is written as:
  - Biomass in influent + Net biomass growth = Biomass in effluent + Biomass wasted
- The biomass in influent is the product of the concentration of microorganisms in the influent ($\boldsymbol{X_o}$) and the flow rate ($\boldsymbol{Q}$).



Completely mixed biological reactor with solids recycle

- The concentration of microorganisms in the influent (X_o) is measured as the **SS** (mg/L).
- The biomass grows in the aeration tank is the product of the volume (∀) and the expression for growth microbial mass as given below.

$$(\forall)\left(\frac{\mu_m SX}{K_s + S} - k_d X\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

where, μ_m= maximum growth rate constant, t^{-1}

S = concentration of limiting food in solution, mg/L

Ks = half-saturation constant, mg/L

= concentration of limiting food when μ = 0.5

k_d = endogenous decay rate constant, t^{-1}

∀ = volume of aeration tank, m^3; ***X*** = **MLVSS** in the aeration tank, mg/L

- The biomass in the effluent is the product of flow rate of treated water leaving the plant (Q-Q_w) and the concentration of microorganisms that does not settle in the secondary clarifier (X_e).
- The biomass that is wasted is the product of concentration of microorganisms in the WAS flow (X_r) and the WAS flow rate (Q_r).
- The equation of the mass balance can be written as:

$$QX_o + \forall\left(\frac{\mu_m SX}{K_s + S} - k_d X\right) = (Q - Q_w)X_e + Q_w X_r \ldots\ldots\ldots\ldots (14)$$

where, $\boldsymbol{Q}$ = flow rate into the aeration tank, m^3/d

$\boldsymbol{Q_w}$ = flow rate of liquid containing microorganisms to be wasted, m^3/d

$\boldsymbol{X_e}$ = microorganisms concentration (**VSS**) in the effluent from secondary settling tank, mg/L; $\boldsymbol{X_r}$ = VSS in sludge being wasted, mg/L

- At steady-state, the mass balance equation for food (Soluble BOD_5) may be written as:

  Food in influent – Food consumed = Food in effluent + Food in WAS……….(15)

- The food in influent is the product of the concentration of Soluble BOD_5 in the influent ($\boldsymbol{S_o}$) and the flow rate ($\boldsymbol{Q}$).
- The food that is consumed in the aeration tank is the product of the volume (∀) and the expression for rate of food utilization as given below.

$$(\forall)\left(\frac{\mu_m S X}{Y(K_s + S)}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

where, $\boldsymbol{Y}$ = decimal fraction food mass converted to biomass =

$$yield\ coefficient \times \frac{mg/L biomass}{mg/L food utilized}$$

- The food in the effluent is the product of flow rate of treated water leaving the plant ($\boldsymbol{Q}$-$\boldsymbol{Q_w}$) and the concentration of Soluble BOD_5 in the influent ($\boldsymbol{S}$).
- Concentration of the Soluble BOD_5 in the influent ($\boldsymbol{S}$) is the same as the aeration tank because it is assumed to be completely mixed.
- The food in the activated sludge flow is the product of conc. that is wasted is the product of conc. of Soluble BOD_5 in the influent ($\boldsymbol{S}$) and the **WAS** flow rate ($\boldsymbol{Q_r}$).
- The equation of the mass balance for food can be written as:

$$QX_o + \forall\left(\frac{\mu_m S X}{K_s + S} - k_d X\right) = (Q - Q_w)X_e + Q_w X_r \ldots\ldots\ldots\ldots (17)$$

  Where symbols carry their usual meaning as defined earlier.

- In order to develop working design equations, the following assumptions are made:
  - The influent and effluent biomass concentrations are negligible compared to that in the reactor.
  - The influent food ($\boldsymbol{S_o}$) is immediately diluted to the reactor concentration in accordance with the definition of CSTR.
  - All reactions occur in the CSTR.

- Based on the assumptions, the **hydraulic detention time (HRT as denoted by θ)** of the reactor is defined as:

$$\theta = \frac{\forall}{Q} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

  - The mean solid or cell-residence time (SRT as denoted by θ_c) is defined as:

$$\theta_c = \frac{\forall X}{Q_w X_r} \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

  - The mean cell-residence time ($\boldsymbol{\theta_c}$) expressed in the above equation must be modified if the effluent biomass concentration is not negligible as follows:

$$\theta_c = \frac{\forall X}{Q_w X_r + (Q - Q_w) X_e} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

  - It is observed from the equations that once the $\boldsymbol{\theta_c}$ is selected, the conc. of soluble **BOD_5** in the effluent (***S***) is fixed and it can be expressed as:

$$S = \frac{K_s(1 + k_d\theta_c)}{\theta_c(\mu_m - k_d) - 1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

  - Since the suspended solids that do not settle can contribute to the BOD_5, this value must be subtracted from the total allowable BOD_5 to find the desirable effluent quality (**i.e. allowable S**) as:

S = Total BOD_5 allowed – BOD_5 in suspended solids …… (22)

  - The concentration of microorganisms, ***X*** (**MLVSS**) in the aeration tank can be expressed as:

$$X = \frac{\theta_c Y(S_o - S)}{\theta(1 + k_d\theta_c)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

  - Values of growth constants for domestic wastewater

Parameter	Basis	Value	
		Range	Typical
$\boldsymbol{K_s}$	mg/L BOD_5	25 - 100	60
$\boldsymbol{k_d}$	d^{-1}	0 - 0.30	0.1
$\boldsymbol{\mu_m}$	d^{-1}	1.0 - 8.0	3.0
Y	mg VSS/mg BOD_5	0.4 - 0.8	0.6

- The return sludge pumping may be determined from a mass balance around the settling tank.
- Assuming that the amount of sludge in the secondary settling tank remains constant (steady-state conditions) and that the effluent ***SS*** (X_e) are negligible, the balance is:

Accumulation = inflow – outflow …….…… (24)

$$0 = (Q+Q_r) X^{/} - (Q_r X_r^{/} + Q_w X_r^{/}) \text{ ………..…….(25)}$$

where, ***Q*** = flow rate into the aeration tank, m^3/d

Q_r = return sludge flow rate, m^3/d

$X^{/}$ = mixed liquor suspended solids (MLSS), g/m^3

Q_w = flow rate of liquid containing microorganisms to be wasted, m^3/d

$X_r^{/}$ = maximum return sludge concentration, g/m^3

Solving for Q_r, we get

$$Q_r = \frac{QX^{/} - Q_w X_r^{/}}{X_r^{/} - X^{/}} \text{……………………….(26)}$$

- If the assumption for the effluent suspended solids (X_e) is negligible is not valid and X_e is significant, the mass balance may be expressed as

$$0 = (Q + Q_r)X^{/} - (Q_r X_r + Q_w X_r^{/} + (Q - Q_w)X_e) \text{…………….(27)}$$

- Solving for Q_r, we get

$$Q_r = \frac{QX^{/} - Q_w X_r^{/} - (Q - Q_w)X_e}{X_r^{/} - X^{/}} \text{…………….(28)}$$

$$X_r^{/} = \frac{10^6}{SVI} mg/L \text{………..………………….(29)}$$

Activated Sludge – Plug Flow with Recycle Waste Activated Sludge Process:

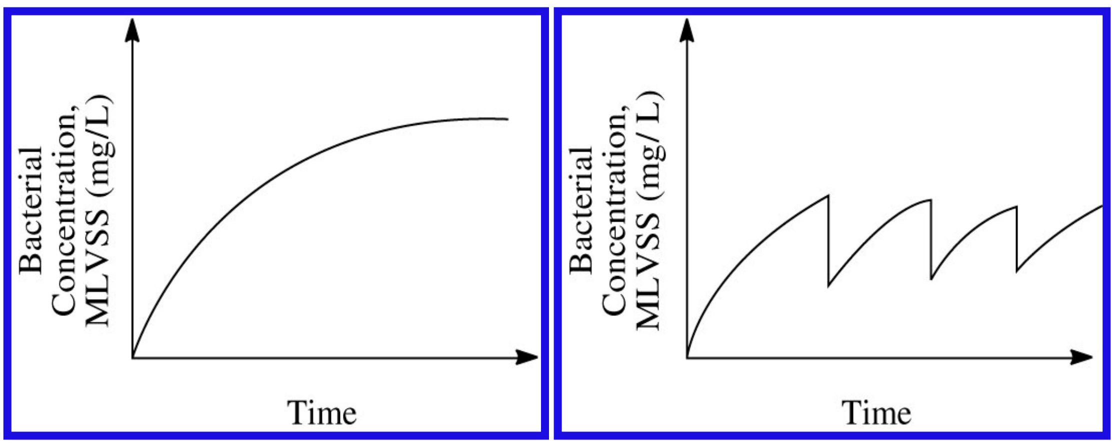
- Similar derivation of equations

- **Activated Sludge – Terms Used – Cont'd**
  - **Food to Microorganisms Ratio (F/M Ratio):** Defined as the ratio of BOD_5 (in mg/d) to MLVSS (in mg). It is usually expressed as:

$$\frac{F}{M} = \frac{QS_o}{\forall X} = \frac{BOD_5/d}{MLVSS} \qquad (30)$$

  - Hence the unit is mg/mg.d.
    - The F/M ratio is controlled by wasting part of the microbial mass, thereby reducing the MLVSS. A high rate of wasting causes a high F/M ratio.
    - A high F/M corresponds to a short θ_c and a low F/M corresponds to a long θ_c. F/M varies from 0.1 to 1.0 mg/mg.d

- **Activated Sludge – Food to Microorganisms Ratio (F/M Ratio):**



Parameter	No Wasting (A)	Wasting (B)
F/M	Low	High
θc	Long	Short
Sludge	None	Much
Oxygen Required	High	Low
Power	High	Low

  - **Mixed-Liquor Suspended Solids (MLSS):** Amount of suspended solids in mixed liquor. It is expressed in mg/L. **MLSS** be limited to **5,000 mg/L** (lower at temperatures of less than 20°C), even though *SVIs* may be very low.

– **Mixed-Liquor Volatile Suspended Solids (MLVSS):** The volatile portion of MLSS. It is also expressed in mg/L.

– **Sludge Volume Index (SVI):** It is determined from a standard laboratory test. It is defined as the volume in millimeters occupied by **1 g** of activated sludge after the aerated liquor has settled for 30 minutes. It is calculated as follows:

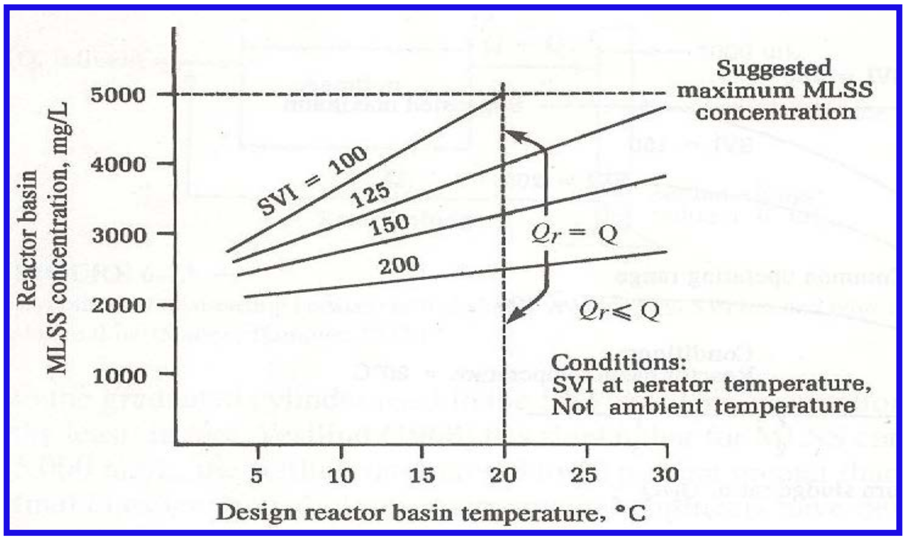
$$SVI = \frac{SV}{MLSS} \times \frac{1{,}000mg}{1g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

Where, SVI = sludge volume index, mL/g

SV = volume of settled solids in one-liter graduated cylinder after 30 minute settling, mL/L

MLSS = mixed-liquor suspended solids, mg/L

- Typical values of SVI for activated sludge plants operating with an MLSS conc. of 2,000-3,500 mg/L range from 80-150 mL/g.



- **Activated Sludge – <u>Sludge Production</u>:**

– Heukelekian (1951) and Sawyer (1956) reported that a net yield of 0.5 kg MLVSS /kg BOD_5 removed could be expected for a completely soluble organic substrate.

– Most researchers agree that, depending on the inert solids in the system and the SRT, 0.4 to 0.6 kg MLVSS/BOD_5 removed will normally be observed.

– The net activated sludge produced per day is determined by:

$$Y_{obs} = \frac{Y}{1 + k_d \theta_c} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

– And

$$P_x = Y_{obs} Q(S_0 - S) \times (10^{-3} kg/g) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

where, P_x = net waste activated sludge produced each day in terms of VSS, kg/d

Y_{obs} = observed yield, kg MLVSS / kg BOD_5 removed.

<u>Example 6</u>: The City of Atlanta has been directed to upgrade its primary WWTP to a secondary plant that can meet the effluent standard of 20 mg/L of BOD_5 and 25 mg/L of SS. They have selected completely mixed activated sludge system. Data given:

BOD_5 in SS = 60%

Primary settling tank effluent characteristics:

Flow = 0.175 m^3/s; BOD_5 = 80.0 mg/L,

Assume the values for the growth constants as K_s = 100 mg/L BOD_5, μ_m = 2.5 d^{-1}; k_d = 0.05 d^{-1}; Y = 0.50 mg VSS/mg BOD_5 removed, MLVSS (X) = 2,000 mg/L, MLSS = 1.2xMLVSS mg/L. **Estimate the volume of the aeration tank.**

<u>Example 6</u>: Solutions

Aeration Tank

(Yellow cells: Data input)

Initial Inputs:

Initial BOD_5 (S_o) =	80.0	mg/L
Effluent Standard BOD_5 =	20.0	mg/L
BOD_5 in SS =	60%	
Effluent Standard SS =	25.0	mg/L
Flowrate (Q) =	0.175	m^3/s
Half velocity constant (K_s) =	100.0	mg/L BOD_5
Decay rate of microorganism (k_d) =	0.050	d^{-1}
Maximum growth rate constant (μ_m) =	2.50	d^{-1}
Yield coefficient (Y) =	0.50	mg VSS/mg BOD_5
Mixed-Liquor volatile SS (MLVSS), X =	2,000	mg/L

TABLE 6-11
Values of growth constants for domestic wastewater[a]

Parameter	Basis	Value[b] Range	Typical
K_s	mg/L BOD_5	25–100	60
k_d	d^{-1}	0–0.30	0.10
μ_m	d^{-1}	1–8	3
Y	mg VSS/mg BOD_5	0.4–0.8	0.6

[a]Sources: Metcalf & Eddy, 2003 and Shahriari et al., 2006.
[b]Values are for 20°C.

Typical values for K_s, k_d, μ_m, and Y are listed in the table above.

The allowable soluble BOD_5 can be calculated using the equation (22) below. The equation uses the effluent standard BOD_5, effluent standard SS, and the percent assumption of BOD_5 in Suspended Solids.

$$S = BOD_5\ allowed - BOD_5\ in\ suspened\ solids$$

S =	20 - 0.6 x 25 =		5	mg/L

The mean cell-residence time can be estimated with the equation (21) below:

$$S = \frac{K_s(1+k_d\theta_c)}{\theta_c(\mu_m-k_d)-1} \Rightarrow \theta_c = \frac{K_s+S}{S(\mu_m-k_d)-K_sk_d}$$

$$\theta_c = \frac{100 + 5}{5(2.5 - 0.05) - 100 \times 0.05} = 14.48 \text{ days}$$

Hydraulic Detention time is computed by the equation (23) below, measured in units (day) and then converted for hours.

$$X = \frac{\theta_c(Y)(S_o-S)}{\theta(1+k_d\theta_c)} \Rightarrow \theta = \frac{\theta_c(Y)(S_o-S)}{X(1+k_d\theta_c)}$$

$$\theta = \frac{14.48 \times 0.5(80 - 5)}{2000\,(1 + 0.05 \times 14.48)} = 0.1575 \text{ day} = 3.78 \text{ hr}$$

The required volume of the aeration tank is then estimated using the equation (18) below:

$$\theta = \frac{\forall}{Q} \Rightarrow \forall = Q\theta$$

$\forall$ = 0.175 m^3/s x 3.78 hr x 3600 s/ 1 hr = 2381.40 m^3 **ANS.**

Example 7: The 500-bed Atlanta Hospital has a small activated sludge plat to treat wastewater from the hospital. The average daily hospital discharge is **1.5 m³** per day per bed, and the average soluble BOD_5 after primary settling is **500 mg/L**. The aeration tank effective liquid dimensions of 10 m x 10 m x 4.5 m. The plant operating parameters are as follows:

MLVSS = 2,500 mg/L; MLSS = 1.20 (MLVSS); Settling sludge volume after 30 min. = 200 mL/L. Determine aeration time, F/M ratio, Sludge volume index (SVI), and solids concentration in the return sludge.

Example 7: Solutions

Q = 1.5 m^3/d/bed x 500 bed = **750 m^3/d**

(a) Aeration period (Equation 18)

$$\boldsymbol{\theta = \frac{\forall}{Q} = \frac{(10m \times 10m \times 4.5m)}{750m^3/d} = 0.6\ d = 14.4\ \text{hours.}}$$

(b) **F/M ratio (Equation 30; Note: 1 mg/L = 1 g/m^3)**

$$\frac{F}{M} = \frac{QS_o}{\forall X} = \frac{750m^3/\text{d} \times 500\text{g/m}^3}{450m^3 \times 2{,}500\text{g/m}^3} = \mathbf{0.33 mg/mg.d}$$

which is within 0.1 to 1.0

(c) **SVI (Equation 31)**

$$SVI = \frac{SV}{MLSS} \times \frac{1{,}000\text{mg}}{1g} = \frac{200\text{mL/L}}{1.20 \times 2{,}500\text{mg/L}} \times \frac{1{,}000\text{mg}}{1g}$$
$$= 66.67\text{mL/g}$$

(d) **Solid concentration in return sludge (Equation 29)**

$$X_r^{/} = \frac{10^6}{SVI}\text{mg/L} = \frac{10^6}{66.67} = 14{,}999.25 \approx 15{,}000 \text{ mg/L}$$

- **Activated Sludge – Oxygen Demand:**
  - An estimate of the oxygen requirements may be made from the BOD_5 of the waste and the amount of activated sludge wasted every day.
  - If it is assumed that all of the BOD_5 is converted to end products, the total oxygen demand can be computed by converting BOD_5 to BOD_L .

    $C_5H_7NO_2 + 5O_2 \leftrightarrow 5CO_2 + 2H_2O + NH_3$+energy ……….. (34)
  - The ratio of gram molecular weights of O_2 to Cell = (5 x32)/113 = 1.42
  - Thus, the oxygen demand may be estimated as *1.42(*P_x*).*
  - The mass of oxygen required may be estimated as:

$$M_{O_2} = \frac{Q(S_0 - S)(10^{-3}kg/g)}{f} - 1.42(P_x)\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(35)$$

where, Q = wastewater flow rate in the aeration tank, m^3/d

S_0 = influent soluble BOD_5, mg/L

S = effluent BOD_5, mg/L

f = conversion factor for converting BOD_5 to BOD_L

P_x = waste activated sludge produced (Eq. 33)

Example 8: The City of Marietta has been directed by the City Board to upgrade its primary Wastewater Treatment Plant (WWTP) to include a secondary treatment process that can meet an effluent standard of 20 mg/L of BOD_5 and 30 mg/L of suspended solids (SS). They want to select completely mixed activated sludge system. The BOD_5 of the SS is estimated to be 50% of the SS concentration. Design the aeration tank including F/M ratio, sludge return, sludge to be wasted each day, volume of air to be supplied, keeping the L:W within 4:1 and maximum depth of 5 m and maximum length of 25 m for the tank. Assume the reasonable values of missing parameters from FE Exam Handbook or from literature.

The effluent characteristics from the primary treatment process are:

Flow = 0.150 m^3/s, BOD_5 = 95 mg/L. Assume the values for the growth constants as K_s = 100 mg/L BOD_5, μ_m = 2.5 d^{-1}; k_d = 0.05 d^{-1}; Y = 0.50 mg VSS/mg BOD_5 removed, MLVSS = 2,500 mg/L, MLSS = 1.20 (MLVSS), wastewater temperature = 25.0°C.

TABLE 6-11
Values of growth constants for domestic wastewater[a]

Parameter	Basis	Value[b] Range	Typical
K_s	mg/L BOD_5	25–100	60
k_d	d^{-1}	0–0.30	0.10
μ_m	d^{-1}	1–8	3
Y	mg VSS/mg BOD_5	0.4–0.8	0.6

[a] *Sources:* Metcalf & Eddy, 2003 and Shahriari et al., 2006.
[b] Values are for 20°C.

Example 8: Solutions

Aeration Tank

(Yellow cell) Data input

Initial Inputs:

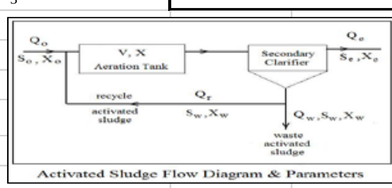
Initial BOD_5 (S_o) =	95.0	mg/L
Effluent Standard BOD_5 =	20.0	mg/L
BOD_5 in SS =	50%	
Effluent Standard SS, Xe =	30.0	mg/L
Flowrate (Q) =	0.150	m^3/s
Half velocity constant (K_s) =	100.0	mg/L BOD_5
Decay rate of microorganism (k_d) =	0.050	d^{-1}
Maximum growth rate constant (μ_m) =	2.50	d^{-1}
Yield coefficient (Y) =	0.50	mg VSS/mg BOD_5
Mixed-Liquor volatile SS (MLVSS), X =	2500.0	mg/L
Sludge Volume Index (SVI) =	175.0	mL/g at 25oC
Desirable Tank Depth, d =	5.0	m
Desirable Tank Dimenions, L : W =	4	: 1
Maximum Length, L_{max} =	25.0	m
MLSS Correction factor, C_f =	1.200	(decimal)
Temperature, T =	25	°C
Air density at std temperature (25°C) =	1.185	kg/m^3
Oxygen transfer rate from air =	10.00%	
Oxygen content in air (by mss) =	23.20%	
BOD_5 to ultimate BOD_u, f =	68.00%	

TABLE 6-11
Values of growth constants for domestic wastewater[a]

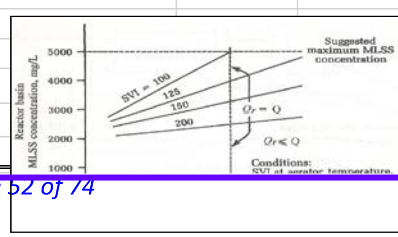
Parameter	Basis	Value[b] Range	Typical
K_s	mg/L BOD_5	25–100	60
k_d	d^{-1}	0–0.30	0.10
μ_m	d^{-1}	1–8	3
Y	mg VSS/mg BOD_5	0.4–0.8	0.6

[a] *Sources:* Metcalf & Eddy, 2003 and Shahriari et al., 2006.
[b] Values are for 20°C.

Typical values for K_s, k_d, μ_m, and Y are listed in the table above.



Activated Sludge Flow Diagram & Parameters



The allowable soluble BOD_5 can be calculated using the equation (22) below. The equation uses the effluent standard BOD_5, effluent standard SS, and the percent assumption of BOD_5 in Suspended Solids.

$$S = BOD_5\ allowed - BOD_5\ in\ suspened\ solids$$

S = 20 - 0.5 x 30 = 5 **mg/L**

The mean cell-residence time can be estimated with the equation (21) below:

$$S = \frac{K_s(1+k_d\theta_c)}{\theta_c(\mu_m - k_d) - 1} \Rightarrow \theta_c = \frac{K_s + S}{S(\mu_m - k_d) - K_s k_d}$$

$$\theta_c = \frac{100 + 5}{5(2.5 - 0.05) - 100 \times 0.05} = 14.48 \text{ days}$$

Hydraulic Detention time is computed by the equation (23) below, measured in units (day) and then converted for hours.

$$X = \frac{\theta_c(Y)(S_o - S)}{\theta(1+k_d\theta_c)} \Rightarrow \theta = \frac{\theta_c(Y)(S_o - S)}{X(1+k_d\theta_c)}$$

$$\theta = \frac{14.48 \times 0.5(95 - 5)}{2500\,(1 + 0.05 \times 14.48)} = 0.1512 \text{ day} = 3.63 \text{ hr}$$

The required volume of the aeration tank is then estimated using the equation (18) below:

$$\theta = \frac{\forall}{Q} \Rightarrow \forall = Q\theta$$

$\forall$ = 0.15 m^3/s x 3.63 hr x 3600 s/ 1 hr = 1959.55 m^3

Tank Dimensions according to given parameters inputted from above:

Assuming Number of Tank =		1	Probable number of tanks necessary = SA /(Lmax x Lamx/W) =		391.91/(25x25/4) =	3
Surface area of each tank, A =	Volume/(d x1) =		1959.55/(5x1) =	391.910	m^2	
Given, L : 4 : 25	∴ **L** =	4W				
Surface Area, A = L x W =	4Wx W		**W =** (391.91/4)^(1/2) =		9.90	**m**
			and L = 4W = 4 x 9.9 =		39.59	**m**
Check for Lmax:	Lmax is NOT OK, Change the number of tank =				3	
	A =	1959.55/(5x3) =		130.64	m^2	
	W = (130.64/4)^(1/2) =			5.71	m	
and	L = 4.5 W =			22.86	m	
	Lmax is OK.					
Check: Total volume provided =		22.86 x 5.71 x 5 x 3 =		1959.55	m^3	
				>= Volume required, OK.		

P.S.: In practice, even number of tanks are provided. In this case you can provide 4 tanks with smaller size.

Food to Microorganism Ratio (F/M)

The F/M ratio is controlled by wasting part of the microbial mass, thereby reducing the MLVSS. A high rate of wasting causes a high F/M ratio. A high F/M yields organisms that are saturated with food. The result is that efficiency of treatment is poor. A low rate of wasting causes a low F/M ratio, which yields organisms that are starved. This results in more complete degradtion of the waste. F/M values typically range from 0.1 to 1.0 mg/mg•d for the various modifications of the activated sludge process. System performs better at F/M ratio of 0.2 to 0.4.

The F/M ratio is computed by the equation (30) shown below:

$$\frac{F}{M} = \frac{QS_o}{\forall X}$$

$$\textbf{F/M} = \frac{(0.15 \times 86{,}400) \times 95}{(1959.55 \times 2500)} = \underline{0.25} \text{ mg/mg} \cdot \text{d}$$

Within the limit of 0.1 to 1.0, OK.

Sludge Return

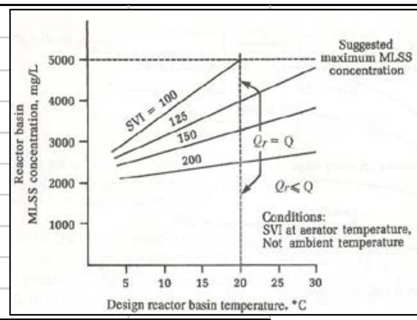
Begin by computing the anticipated concentration of the MLSS using the input for "MLSS Correction Factor":

MLSS = 1.2(2500) = **3,000** **mg/L**

Using the figure below, find the sludge volume index based on your temperature in °C as well as your MLSS. Once you have located your value, input the value below:

SVI = 175.0 mL/g

Given temperature, T = 25 °C



The estimated solid concentration in return sludge is (Eq. 5-29):

$$X'_r = \frac{10^6}{SVI}$$

$$X'_r = \frac{1{,}000{,}000}{175.00} = \underline{5{,}714} \text{ mg/L}$$

The sludge wasting flow rate may be computed by the equation below, keeping in mind that Xr = X'r/1.2:

$$Q_w = \frac{\forall X}{\theta_c X_r} \times \frac{1d}{86400s}$$

Q_w = $\frac{(1959.55 \times 2500 \times 1)}{(14.48 \times (5714.29/1.2) \times 86{,}400)}$ = **0.0008** **m^3/s**

Noting that 1 mg/L = 1 g/m^3, if we ignore the effluent suspended solids, the estimated return sludge flow rate is (Eq. 26):

$$Q_r = \frac{QX' - Q_w X'_r}{X'_r - X'}$$

Q_r = $\frac{((0.15)(3000) - (0.0008)(5714.29))}{(5714.29 - 3000)}$ = **0.164** **m^3/s**
0.170 **m^3/s** Rounded

If the effluent suspended solids are not neglected, the estimated return sludge flow rate is Q_r: (Eq. 28)

$$Q_r = \frac{QX' - Q_w X'_r - (Q - Q_w)X_e}{X'_r - X'}$$

Q_r = $\frac{((0.15)(3000) - (0.0008)(5714.29) - (0.15 - 0.0008)(30))}{(5714.29 - 3000)}$ = **0.162** **m^3/s**
0.170 **m^3/s** (rounded)

Sludge Production

Using the equation (32) below, the observed yield is computed:

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

Y_{obs} = $\frac{(0.5)}{(1 + (0.05)(14.48))}$ = **0.29** **kg VSS/kg BOD_5 removed**

With the observed yield, the net waste activated sludge produced each day is computed below (Eq. 33):

$$P_x = Y_{obs} Q(S_o - S)(86{,}400s/d)(\frac{10^{-3}kg}{g})$$

P_x = (0.29)(0.15)(95 - 5)(86,400)(10^-3) = **338.3** **kg/d of VSS**

The total mass produced includes inert materials. Using the relationship between MLSS and MLVSS:

Increase in MLSS = $C_f \; x \; P_x$ = (1.2)(338.26) = **405.9** **kg/d**

The mass of solids (both volatile and inert) lost in the effluent is:

$$SS\ lost\ in\ effluent = (Q - Q_w)(X_e)(\frac{86400s}{d})(\frac{10^{-3}kg}{g})$$

SS lost in effluent = (0.15 - 0.0008)(30)(86,400)(10^-3) = **386.67** **kg/d**

The mass to be wasted is then:

$$Mass\ to\ be\ wasted = increase\ in\ MLSS\ - SS\ lost\ in\ effluent$$

Mass to be wasted = 405.91 - 386.67 = **19.24** **kg/d**

Oxygen Demand				
Before the oxygen demand can be computed, two inputs must be entered. The first input is the conversion factor for converting BOD_5 to ultimate BOD_u. This input must be listed in percent form. The second input is the oxygen transfer rate from the air in percent.				
BOD_5 to BOD_u ratio, *f* =	**68.00%**	**Percent**	*(Assumed)*	
O_2 transfer efficiency, η =	**10.00%**	**Percent**		

The mass of oxygen required may be estimated as: (Eq. 35)

$$M_{O_2} = \frac{Q(S_o - S)(10^{-3} kg/g)}{f} - 1.42(P_x)$$

$$M_{O2} = \frac{(0.15)(95 - 5)(86{,}400)(10^{-3})}{0.68} - 1.42(338.26) = \mathbf{1234.97} \text{ kg/d of } O_2$$

Assumptions:

air density =	**1.185**	kg/m^3 at standard conditions (25°C)
By mass, air contains	**23.2%**	oxygen

Below is the comparison of 100 percent transfer efficiency, and the input, η, transfer efficiency:

η = 100% - Oxygen Transfer Efficiency

$$\forall_{air} = \frac{M_{O2}}{\rho \times \%O_2 \times \eta}$$

$$\forall_{air} = \frac{1234.97}{(1.185) \times (0.232) \times 1.00}$$

$\forall_{air}$ = **4,492** m^3/d

VS.

η = 10% - Oxygen Transfer Efficiency

$$\forall_{air} = \frac{M_{O2}}{\rho \times \%O_2 \times \eta}$$

$$\forall_{air} = \frac{1234.97}{(1.185) \times (0.232) \times 0.1}$$

$\forall_{air}$ = **44,921** m^3/d

11.4 Rotating Biological Contactors (RBCs)

The RBC process consists of closely spaced discs (3 – 3.5 m in diameter) mounted on a horizontal shaft and rotated, while about one-half of their surface are is immersed in wastewater. The discs are typically constructed of plastic. The speed of rotation of the discs is adjustable. The microbes in the wastewater begin to adhere to the rotating surfaces and grow there until the entire surface area of the discs is covered with a 1 to 3 mm layer of biological slime.

The rotating discs carry a film of wastewater into air, this wastewater trickles down the surface of the discs, absorbing oxygen. The film of water mixed with the reservoir of wastewater as the discs complete their rotation that add oxygen in the reservoir and mixing the treated and partially treated wastewater. As the attached microbes pass through the reservoir, they absorb other organics for breakdown. The excess growth of microbes is shared from the discs as they move through the reservoir. These dislodge organisms are kept in suspension by the moving discs, thus the discs serve several purposes:

- They provide media for buildup of attached microbial growth.
- They bring the growth into contact with wastewater.
- They aerate the wastewater and the suspended microbial growth in the reservoir.

11.5 Oxidation Ponds

Waste stabilization pond that refers to a pond or lagoon used to treat organic waste by biological and physical processes. There are 5 types of oxidation/stabilization ponds/lagoons:

1. Aerobic Ponds
2. Anaerobic Ponds
3. Facultative Ponds
4. Maturation or Tertiary Ponds
5. Aerated Lagoons

Oxidation Ponds = Aerobic Ponds

The aerobic pond is shallow, less than 1 m in depth, in which light penetrates to the bottom, thereby maintaining active algal photosynthesis throughout the entire system. During the daylight hours, large amounts of oxygen are supplied by the photosynthesis process; during the hours of darkness, wind mixing of the shallow water mass generally provides a high degree of surface reaeration. Stabilization of the organic material entering an aerobic pond is accomplished mainly through the action of aerobic bacteria.

Oxidation Ponds = Anaerobic Ponds

Deep ponds that receive high organic loadings such that anaerobic conditions prevail throughout the entire pond depth. Anaerobic ponds become turbid from the presence of reduced metal sulfides. This restricts light penetration to the point that algal growth becomes negligible. Anaerobic treatment of complex wastes involves **two distinct stages:**

(1) Acid fermentation:

- Complex organic materials are broken down mainly to short-chain acids and alcohols.

(2) Methane fermentation:

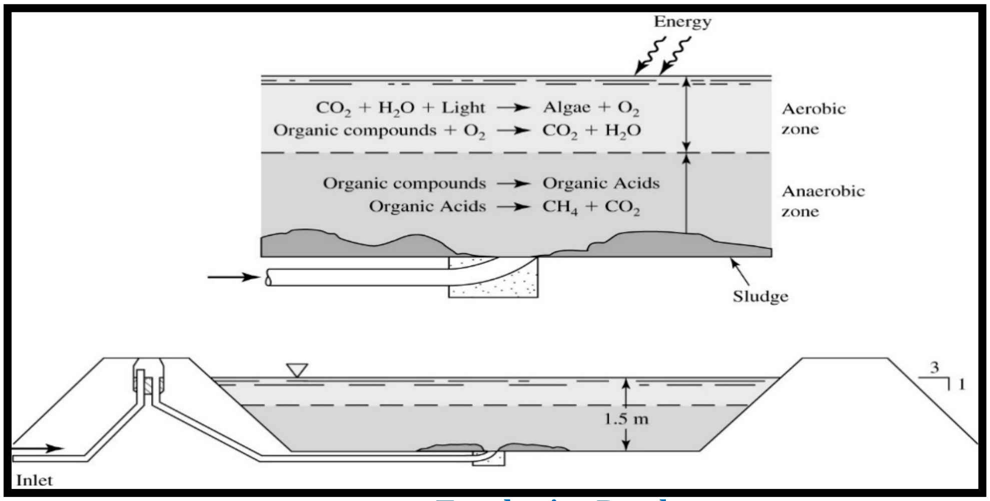
- Acids and alcohols are converted to gases, primarily methane and carbon dioxide.

The proper design of anaerobic ponds must result in environmental conditions favorable to methane fermentation. Anaerobic ponds are used primarily as a pretreatment process and particularly suited for treatment of high-temperature, high-strength wastewaters. However, they have been used successfully to treat municipal wastewater as well.

Oxidation Ponds = Facultative Ponds

Ponds depth ranges from 1 to 2.5 m, which have anaerobic lower zone, a facultative middle zone, and aerobic upper zone maintained by photosynthesis and surface aeration. Approx. 25% of the municipal wastewater treatment plants in the USA are ponds and about 90% of these ponds are located in communities of 5,000 people or fewer. Facultative ponds are popular for such treatment situations:

- Because long retention times facilitate the management of large fluctuations in wastewater flow and strength with no significant effect on effluent quality.
- Capital, operating, and maintenance costs are less than those other biological systems that provide equivalent treatment.
- The Michigan rule of thumb:
    1. The BOD_5 loading rate should not exceed 22 kg/ha . d on the smallest lagoon cell.
    2. The detention time in the lagoon (considering total volume of all cells but excluding bottom 0.6 m in the volume calculation) should be 6 months.



Facultative Ponds

Oxidation Ponds – Maturation or Tertiary Ponds

Ponds used for polishing effluents from other biological processes. Dissolved oxygen is furnished through photosynthesis and surface aeration. This type is also known as polishing pond.

Aerated Lagoons

Ponds oxygenated through the action of surface or diffused air aeration.

12.Disinfection

Disinfection is used in wastewater treatment to **reduce pathogens (disease causing microorganisms) to an acceptable level**.

Three categories of human enteric pathogens are normally of consequence: bacteria, viruses, and amebic cysts.

Water disinfectants must have the following properties:

Water disinfectants must have the following properties:

- They must destroy the kinds and numbers of pathogens that may be introduced into the water within a practicable period of time over an expected range of temperature.
- They must meet possible fluctuations in composition, concentration, and condition of the waters or wastewaters to be treated.
- They must be neither toxic to humans and domestic animals nor unpalatable or otherwise objectionable in required concentrations.
- They must be dispensable at reasonable cost and safe and easy to store, transport, handle, and apply.
- Their concentration or strength in the treated water must be determined easily, quickly, and (preferably) automatically.
- They must persist within disinfected water in a sufficient concentration to provide reasonable residual protection against its possible recontamination before use.

Disinfection kinetics

Chick's Law: Number of organisms destroyed in a unit time is proportional to the number organisms remaining:

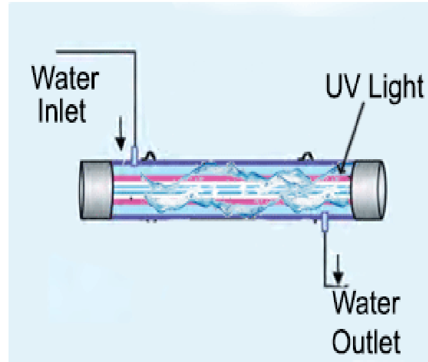
$$-\frac{dN}{dt} = kN$$

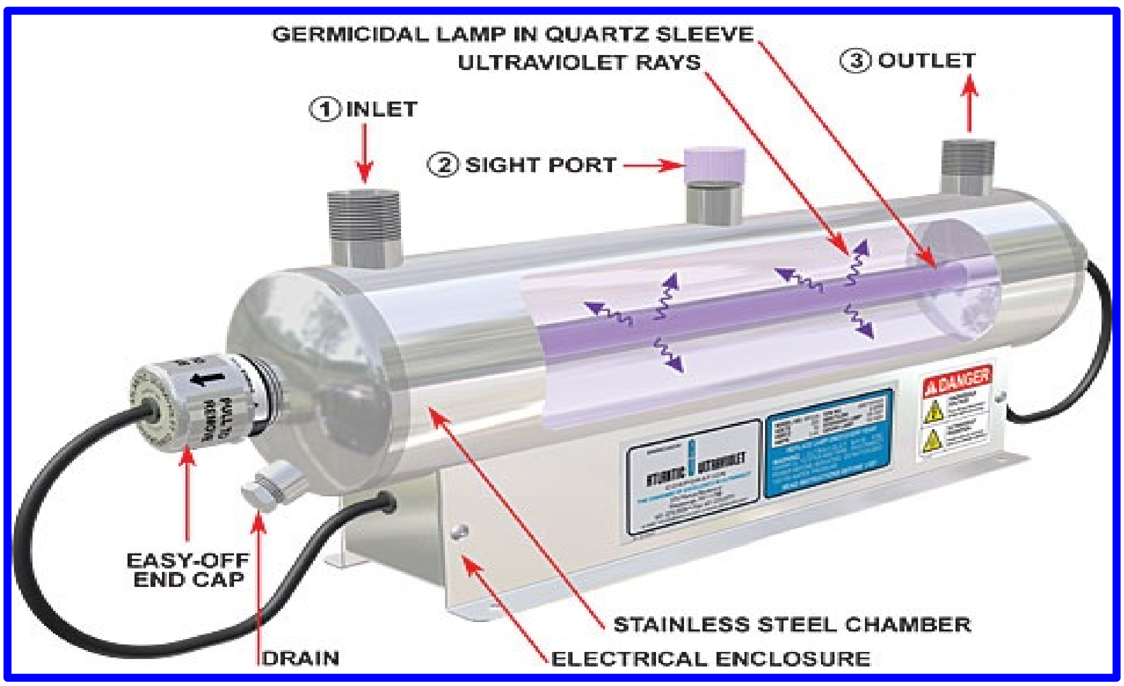
This is a first-order reaction. The solution is:

$$N = N_0 e^{-kt}$$

The last treatment step in a secondary treatment plant is the addition of disinfectant to the treated wastewater.

- Chlorine gas or some other form of chlorine is commonly used in the USA for wastewater disinfection.
- Wastewater is held into a basin for **15 minutes** to allow the chlorine to react with pathogens.
- UV radiation or ozone can also be used for wastewater disinfection.
- UV radiation is the best option for disinfection in wastewater treatment as the residual chlorine due to disinfection by chlorine could be harmful to aquatic lives.
- If disinfection by chlorine is done, treated wastewater has to be dechlorinated by potassium bisulfate before discharging to the surface water.





13. Advanced Wastewater Treatment

12.1 Filtration

- Secondary treatment processes, such as activated-sludge process, are highly efficient removal of biodegradable colloidal and soluble organics.

- Typical BOD_5 from the secondary treatment processes is 20 to 50 mg/L, this is because of secondary clarifier is not perfectly efficient at settling out the microorganisms from the biological treatment processes.
- These organisms contribute both to the suspended solids and to the BOD_5 because the process of biological decay of dead cells exerts BOD.
- By using filtration process similar to that used in water treatment plants, it is possible to remove the residual suspended solids including the unsettled microorganisms.
- Removing the microorganisms reduces the residual BOD_5.
- Typical, plain filtration can reduce activated sludge effluent suspended solids from 25 to 10 mg/L.
- Typically, filtration can achieve 80% of suspended solids reduction for activated sludge and 70% reduction for trickling filter effluent.
- **Classification of Filters:**
  - Based on type of medium used such as sand, coal (called anthracite), **dual media** (coal plus sand), or **mixed media** (coal, sand, and garnet).
  - **Based on allowable loading rate:** Loading rate is the flow rate of water applied per unit area of the filter. It is the velocity of water approaching the face of the filter.

$$v_a = \frac{Q}{A_s}$$

where, v_a = *face velocity, m/d = loading rate,* $m^3/d.m^2$.

Q = *flow rate on filter surface,* m^3/d

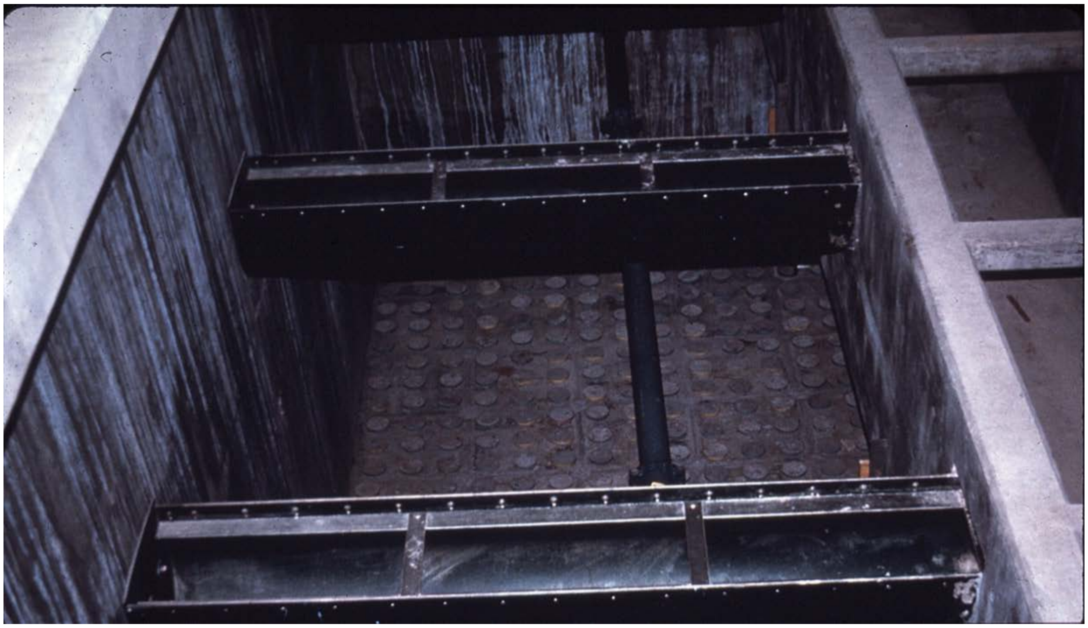
A_s = *surface area of filter,* m^2.

  - Based on loading rate, the filters are classified as ***slow sand filter***, ***rapid sand filter***, or ***high-rate sand filters.***

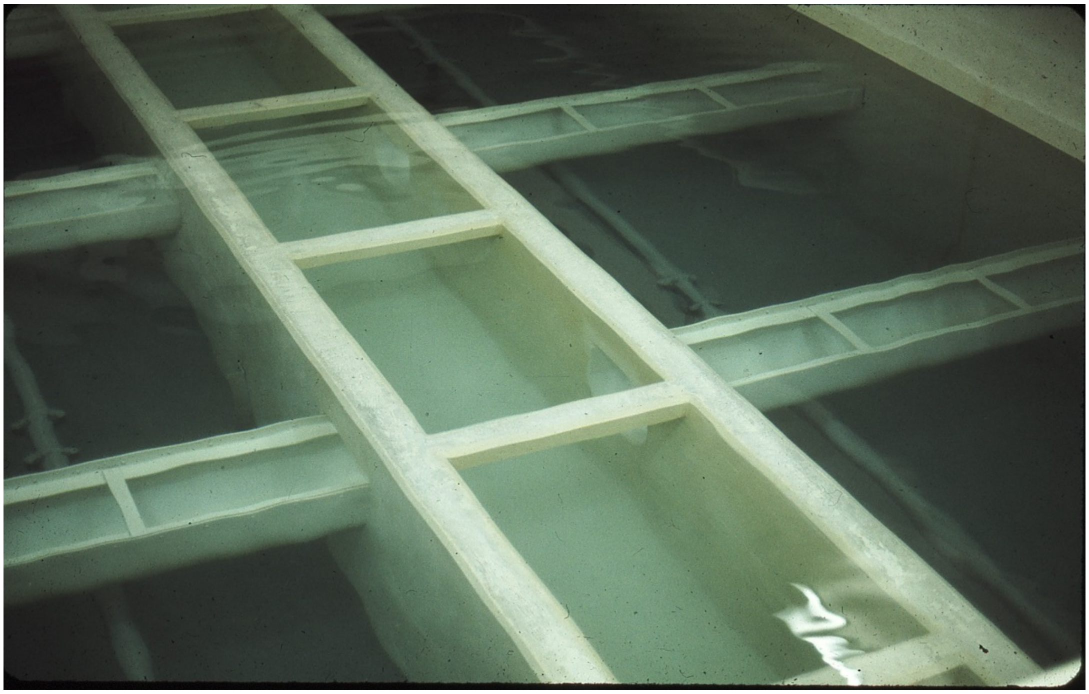
- **Slow Sand Filters:**
  - First introduced in the 1800s.
  - Loading rate = 2.9 to 7.6 $m^3/d.m^2$
  - As the suspended or colloidal material is applied to the sand, the particles begin to collect in the top 75 mm and to clog the pore spaces.
  - As the pore spaces clogged, water will no longer pass though the sand.

- Top layer of sand is scrapped off, cleaned, and replaced.
- It requires large areas of land and are operator intensive.

- **Rapid Sand Filters:**
  - First developed in the 1900s when large number of filtration systems need to be employed to prevent epidemics.
  - RSFs have ***graded*** (layered) sand in bed.
  - The sand grain size distribution is selected to optimize the passage of water while minimizing the passage of particulate matter.
  - RSFs are cleaned in place by forcing water backwards through the sand, called ***backwashing***.
  - Loading rate up to 120 - 235 $m^3/d.m^2$
  - Normally, a minimum of 2 filters are constructed to ensure redundancy.
  - For larger plants (> $0.5\ m^3/s$), a minimum of 4 filters is suggested.
  - Maximum surface area = 100 m^2



Rapid sand filter with exposed under drain block. Wash water troughs run from left into gullet on right.



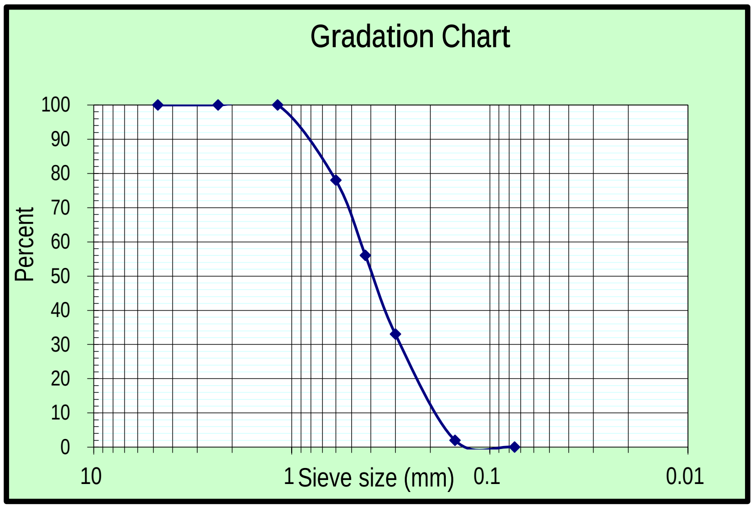
Rapid sand filter during filtration. Reflection shows water level above gullet and backwash troughs.

Dual-Media Filters:

- Developed in in the wartime era of the early 1940s.
- Utilize more of the filter depth to remove particle.
- Removal of larger particles on the top of the smaller particles that is accomplished by placing a layer of coarse coal on top of a layer of fine sand.
- Coal has lower specific gravity than sand, so after backwash, coal settles slower than sand and ends up on top.
- Loading rate up to = 300 $m^3/d.m^2$

- **Deep-Bed Monomedi Filters:**
  - Came into use in mid 1980s. Designed to achieve higher loading rate and produce lower finished water turbidities.
  - The filters consist of 1.0 - 1.5 mm diameter anthracite about 1.5 - 2.5 m deep.
  - Loading rate up to = 800 $m^3/d.m^2$

- **Grain Size Distribution:**
  - For silica sand effective size (D_{10}) = 0.35 to 0.55 mm with a max. of 1.0 mm.
  - Uniformity coefficient ($U = D_{60}/D_{10}$) = 1.3 to 1.7



$D_{10} = 0.19\ mm$
$D_{60} = 0.45\ mm$
$U = 0.45/0.19 = \underline{2.368}$

- **Filter Hydraulics:**
  - The loss of pressure (head loss) through a filter media with uniform porosity is expressed by (Rose, 1954):

$$h_L = \frac{1.067 v_a^2 D}{\phi g n^4} \sum_{i=1}^{n} \frac{C_D f}{d}$$

Where, h_L = frictional head loss through the filter, m

v_a = approach velocity, m/s

D = depth of filter sand, m

C_D = drag coefficient

f = mass friction of sand particles of diameter, d

d = diameter of sand particles, m

ϕ = shape factor; g = acceleration due to gravity, m/s^2

n = porosity

- **Filter Hydraulics:**

Initial head loses in excess of **0.6 m** indicate that either the loading rate is too high, or the sand has too large of fine grain sizes.

12.2 Carbon Adsorption

Adsorption is a mass transfer process wherein a substance is transferred from the liquid phase to the surface of a solid where it bounds by chemical or physical forces.

In water treatment, the adsorbent (solid) is activated carbon either granular activated carbon (GAC) or powdered activated carbon (PAC).

PAC is fed to the raw water in a slurry and used to **remove taste- and odor causing substances** or to provide some removal of synthetic organic chemicals (SOC).

GAC is added to the filter system by replacing the anthracite with GAC or an additional contractor is built with GAC.

GAC has been proposed to be used to remove naturally occurring organic matter that would, in turn, reduce formation of disinfection by-products (DBPs).

GAC usually lasts for 90-120 days until it losses its adsorption capacity.

12.3 Phosphorous Removal

The removal of phosphorous to prevent or reduce eutrophication is typically accomplished by chemical precipitation using one of the three compounds. The reactions for each compound are as follows:

Using ferric chloride: $FeCl_3 + HPO_4^{2-} \leftrightarrow FePO_4\downarrow + H^+ + 3Cl^-$ (effective pH = 5.5 – 7.0)

Using alum: $Al_2(SO_4)_3 + 2HPO_4^{2-} \leftrightarrow 2AlPO_4\downarrow + 3H^+ + 3SO_4^{2-}$ (effective pH = 5.5 – 7.0)

Using lime: $5Ca(OH)_2 + 3HPO_4^{2-} \leftrightarrow Ca_5(PO_4)_3OH\downarrow + 3H_2O + 6OH^-$

12.4 Nitrogen Removal

Nitrogen in any soluble form NH_3, NH_4^-, NO_2^-, NO_3^-, but not N_2 gas, is a nutrient and may need to be removed from wastewater to help control algal growth in the receiving body. Nitrogen in the form of ammonia exerts an oxygen demand (BOD) and can be toxic to fish. The nitrogen removal is accomplished by both a biological process called ***nitrification/ denitrification*** and by a chemical process called ***ammonia stripping***.

Nitrification is the biological oxidation of ammonia with oxygen into nitrite followed by the oxidation of these nitrites into nitrates. Degradation of ammonia to nitrite is usually the rate limiting step of nitrification. Nitrification is an important step in the nitrogen cycle in soil. This process was discovered by the Russian microbiologist, Sergei Winogradsky.

Nitrification is a process of nitrogen compound oxidation (effectively, loss of electrons from the nitrogen atom to the oxygen atoms):

$NH_3 + 1.5\ O_2$ + Nitrosomonas → $NO_2^- + H_2O + H^+$

$NO_2^- + 0.5\ O_2$ + Nitrobacter → NO_3^-

$NH_3 + O_2 \rightarrow NO_2^- + 3H^+ + 2e^-$

$NO_2^- + H_2O \rightarrow NO_3^- + 2H^+ + 2e^-$

Denitrification is a microbially facilitated process of nitrate reduction that may ultimately produce molecular nitrogen (N_2) through a series of intermediate gaseous nitrogen oxide products. Denitrification generally proceeds through some combination of the following intermediate forms:

$NO_3^- \rightarrow NO_2^- \rightarrow NO + N_2O \rightarrow N_2\ (g)$

The complete denitrification process can be expressed as a redox reaction:

$2\ NO_3^- + 10\ e^- + 12\ H^+ \rightarrow N_2 + 6\ H_2O$

The nitrification step is expressed in chemical terms as follows:

$NH_4^+ + 2O_2$ + (bacteria) ↔ $NO_3^- + H_2O + 2H^+$

Bacteria must be present to cause the reaction to occur and this step satisfies the oxygen demand of ammonia. If nitrogen level is of concern, the nitrification step must be followed by anoxic **denitrification** by bacteria:

NO_3^- + organic matter → $N_2 + CO_2 + H_2O$

Nitrogen in the form of ammonia can be removed chemically by **raising pH** to convert the ammonium into ammonia, which can then be stripped from the water by passing large quantities of air through the water. The reaction is as follows:

$NH_4^+ + OH^- \leftrightarrow NH_3 + H_2O$

The hydroxide is usually supplied by **adding lime**.

14. Summary

In this course we understood the principles of wastewater treatment and unit processes, such as wastewater microbiology, characteristics of domestic and industrial wastewater, on-site disposal, municipal wastewater treatment systems, unite operations of pretreatment, primary treatment (mostly sedimentation), unit processes for secondary treatment (mostly biological treatment processes such as trickling filter, activated sludge, rotating biological contactor, and oxidation lagoons), disinfection, and advanced wastewater treatment such as filtration, adsorption, and phosphorous and nitrogen removal technologies. In this course we defined several terms related to secondary treatment processes mostly activated sludge. We solved several problems to understand the extent of wastewater treatment and design some unit processes.

15. References

1. Introduction to Environmental Engineering by Mackenzie L. Davis and David A. Cornwell, McGraw-Hill Book Company.
2. Personal work experience and open source from internet.

++++++++++++++++ The End ++++++++++++++

Any questions please contact the instructor at makarim@juno.com

QUIZ for Principles of Wastewater Treatment (*Answer Key)

1. Microorganisms are organized into _________ broad groups (called kingdom) based on their structural and functional differences.
a. 2
b. 4
c. 5
d. 8

2. Carbon is a basic building block for cell synthesis. Based on carbon source microorganisms are classified as:
a. Obligate Aerobes, Obligate Anaerobes, and Facultative Aerobes
b. Heterotrophs and Autotrophs
c. Phototrophs, Chemotrops, Organotrophs, and Lithotrophs
d. Psychrophiles, Mesophiles, Thermophiles, and Stenothermophiles

3. Based on temperature range microorganisms are classified as:
a. Obligate Aerobes Obligate Anaerobes, and Facultative Aerobes
b. Heterotrophs and Autotrophs
c. Phototrophs, Chemotrophs, Organotrophs, and Lithotrophs
d. Psychrophiles, Mesophiles, Thermophiles, and Stenothermophiles

4. Based on the energy source microorganisms are classified as:
a. Obligate Aerobes Obligate Anaerobes, and Facultative Aerobes
b. Heterotrophs and Autotrophs
c. Phototrophs, Chemotrophs, Organotrophs, and Lithotrophs
d. Psychrophiles, Mesophiles, Thermophiles, and Stenothermophiles

5. Based on the oxygen utilization microorganisms are classified as:
a. Obligate Aerobes, Obligate Anaerobes, and Facultative Aerobes
b. Heterotrophs and Autotrophs
c. Phototrophs, Chemotrophs, organotrophs, and lithotrophs
d. Psychrophiles, Mesophiles, Thermophiles, and Stenothermophiles

6. The term anoxic is commonly used to describe conditions under which free oxygen is present only at very low concentrations or when oxygen is absent, and nitrite or nitrate is present.
a. False
b. True

7. The general term that describes all of the chemical activities formed by a cell is metabolism. Metabolism is divided into two parts: ______________ and ______________.
a. Catabolism and Anabolism
b. Metabolism and Catabolism
c. Metabolism and Anabolism
d. all of the above

8. In anaerobic decomposition, molecular oxygen and nitrate must be present as terminal electron acceptors. Sulfate (SO_4^{2-}), CO_2, and organic compounds that can be reduced serve as terminal electron acceptors. The reduction of sulfate results in the production of hydrogen sulfide (H_2S) and a group of equally odiferous organic sulfur compounds called mercaptans.
a. True
b. False

9. In aerobic decomposition, molecular oxygen (O_2) must be present as the terminal electron acceptor for decomposition to proceed by aerobic oxidation. In natural water bodies, oxygen is measured as DO. In this case, chemical end products are primarily **CO_2, water**, and **new cell** material.
a. True
b. False

10. The major end products of anaerobic decomposition are ___________, ____________, and ____________.
a. carbon dioxide, methane, and nitrogen
b. carbon dioxide, methane, and water
c. carbon dioxide, ammonia, and oxygen
d. carbon monoxide, methane, and water

11. The microbial population growth follows the binary fission. The population of bacteria *(P)* after the *n*th generation is given by the following expression: $P = P_o(2)^n$. After 5th generation, 1 microorganism will become ___________ microorganisms:
a. 8
b. 16
c. 32
d. 64

12. In industrial wastewater, the conventional Pollutants are:
a. BOD
b. Total Suspended Solids (TSS)
c. Oil and grease
d. pH
e. All of the above
f. None of the above

13. USEPA established a definition of secondary treatment based on 3 wastewater characteristics and these are:
a. BOD_5, SS, and pH
b. BOD_5, SS, and Turbidity
c. BOD_5, DS, and pH
d. BOD_7, SS, and pH

14. The alternatives for municipal wastewater treatment fall into 3 major categories and these are:
a. Preliminary treatment; Secondary treatment, and Advanced or tertiary treatment
b. Primary treatment; Secondary treatment, and Advanced or tertiary treatment
c. Primary treatment; Secondary treatment, and Special treatment
d. Primary treatment; Preliminary treatment, and Advanced or tertiary treatment

15. The typical devices or structures that are used in primary treatment are:
a. Bar Racks
b. Grit Chambers
c. Comminutors and Equalizations
d. all of the above
e. none of the above

16. Devices that are used to macerate wastewater solid (rags, paper, plastic, and other materials) by revolving cutting bars are called _______________.
a. grit chambers
b. comminutors
c. bar racks
d. equalization basins

17. Flow equalization is not a treatment process, but a technique that can be used to improve the effectiveness of both secondary and advanced wastewater treatment processes.
a. True
b. False

18. The purpose of flow equalization is not to dampen the variations of flows in different times of the day so that the wastewater can be treated as a nearly constant flow rate.
a. True
b. False

19. The Stokes equation is applicable in Type II settling as the flocculating particles are continually changing in size, shape, and when water is entrapped in the floc.
a. True
b. False

20. The Stokes equation is applicable in Type I (discrete) settling as the individual particles settle.
a. True
b. False

21. Nitrification is the biological oxidation of ammonia with oxygen into nitrite followed by the oxidation of these nitrites into nitrates.
a. True
b. False

22. As the microorganisms grow and are mixed by the agitation of the air, the individual organisms clump together (flocculate) to form an active mass of microbes (biologic floc) called ____________ __________.

a. mixed liquor
b. activated sludge
c. suspended solids
d. sludge volume

23. The mixture of activated sludge and wastewater in the aeration tank is called ___________ ___________.

a. mixed liquor.
b. activated sludge
c. suspended solids
d. sludge volume

24. Denitrification is a microbially facilitated process of nitrate reduction that may ultimately produce molecular ____________ through a series of intermediate gaseous nitrogen oxide products.

a. nitrogen (N_2)
b. oxygen (O_2)
c. chlorine (Cl_2)
d. iodine (I_2)

25. The typical sludge age of an effective system is ____ to ____ days but should not be < 10 days and > 50 days.

a. 10 to 20
b. 20 to 30
c. 30 to 40
d. 40 to 50.

26. The typical sludge age of an effective wastewater treatment system is 20 to 30 days but should not be < ______days and > ________ days.

a. 10   20
b. 10 50
c. 10 60
d. 20 50

27. The biomass *mixed liquor suspended solids* (**MLSS**) concentration should be < 1,000 mg/L and > 5,000 mg/L; a lower concentration could result in the clarifier failing.

a. True
b. False

28. A portion of the microorganisms that is discarded from the process is called ____________.
a. wasting
b. handling
c. discharging
d. discarding

29. The discarded microorganisms in the activated sludge process are called _________ ___________ ____________.
a. waste activated sludge
b. waste inactivated sludge
c. waste concentrated sludge
d. waste treated sludge

30. The average amount of time that microorganisms are kept in the treatment system is called ________________.
a. HRT or hydraulic retention time
b. SRT or sludge age or mean cell residence time
c. all of the above
d. none of the above

31. A high F/M ratio corresponds to a _______ θ_c and a low F/M corresponds to a _______ θ_c and F/M ratio varies from 0.1 to 1.0 mg/mg.d.
a. short; long
b. small; large
c. small; big
d. long; short

32. Two mass balances are required to define the design of the completely mixed (CSTR) activated sludge process and these are ____________ and ___________.
a. Biomass and Food (Soluble BOD_5)
b. Biomass and pH
c. Food and Turbidity
d. Biomass and Turbidity

33. The F/M ratio is controlled by wasting part of the microbial mass, thereby reducing the MLVSS. A high rate of wasting causes a high F/M ratio.
a. True
b. False

34. The typical sludge age or solid retention time of an effective wastewater treatment system is 20-30 days but should not be < 10 days and > ______ days.
a. 10
b. 25
c. 30
d. 50

35. Wastewater is held into a basin for ________ minutes to allow the chlorine to react with pathogens.
a. 10
b. 15
c. 20
d. 25

36. The available advanced or tertiary waste treatment (AWT) processes are: ___________, _____________, __________________, and _____________.
a. Sedimentation, Carbon adsorption, Phosphorous removal, and Nitrogen control
b. Filtration, Precipitation, Phosphorous removal, and Nitrogen control
c. Filtration, Carbon adsorption, Phosphorous removal, and Coagulation flocculation
d. Filtration, Carbon adsorption, Phosphorous removal, and Nitrogen control

37. The refractory materials from the wastewater can be removed by adsorbing them on _________ _____________.
a. Alumina silica
b. activated carbon
c. activated nitrogen
d. activated phosphorous

38. The removal of phosphorous to prevent or reduce eutrophication is typically accomplished by ___________ __________ using one of the three compounds such as ferric chloride, alum, and lime.
a. Coagulation flocculation
b. chemical precipitation
c. chemical oxidation
d. physical oxidation

39. The nitrogen removal is accomplished by both a biological process called nitrification/ denitrification and by a chemical process called _________ ____________.
a. air stripping
b. ammonia stripping
c. chemical precipitation
d. oxidation reduction

40. Under _____________ program, all facilities that discharge pollutants from any point source into waters of the US are required to obtain a ____________ permit.

a. SWPP
b. Air Pollution
c. NPDES
d. Solid Waste