Wastewater Treatment Engineering

110401455

Introduction Instructor: Dr. Zeyad Tarawneh

The course will cover:

• Introduction to drinking water quality: water quality in relation to wastewater quality, water quality parameters, water treatment.

• Wastewater flow quantity: sources, population estimation, flow amount, peak flow, wastewater flow variation.

• Wastewater collection system: components of sewer systems, design and performance of sewers.

• Wastewater flow characteristics: wastewater flow quality parameters.

• Wastewater treatment: primary treatment, secondary treatment, sludge handling.





Water quality in relation to wastewater:

The characteristics of wastewater flow are highly affected by the amount and the quality of domestic water used during the human daily activities. For example, the pH of the wastewater flow is related to drinking water chemical parameter that is called the alkalinity. Therefore it is an essential step to study and understand the calculations related to water quality parameters for better understanding of the wastewater characteristics in which treatment plans come after.

Water quality and treatment

Water quality parameters:

Physical water quality parameters: like suspended solids content, turbidity, water color, and odor.

Chemical water quality parameters: like dissolved solids content, alkalinity, hardness, fluoride content, metals content, and organics.

Biological water quality parameters: like pathogens including viruses, bacteria, protozoa, and helminthes (parasitic worms).

Water quality and treatment		
Drinking water standards (EPA) for selected parameters	Contaminant	Max level
	Total Coliforms (#/100mL)	5
	Turbidity (NTU)	1 – 5
	Color (Units)	15
	Odor (TON)	3
	pH	6.5 - 8.5
	Total dissolved solids (mg/L)	500
	Inorganic chemicals (mg/L)	
	Chloride	250
	Fluoride	0.7 - 2.4
	Lead	0.05
	Cadmium	0.01
	Silver	0.05
	Zinc	5
	Copper	1
	Organic chemicals (µg/L)	
	Trihalomethanes	100
	Benzene	0.05
	Carbon tetracholride	0.05

Physical parameters: Suspended solids content

Water total solids consist of suspended solids (SS) and dissolved solids (DS). The suspended solids can be either organic or inorganic. Organic solids like plant fibers, algal cells, and bacteria. Inorganic solids like polymers and other industrial materials. Generally the suspended solids can be removed from water by sedimentation. If suspended solids do not obey the traditional sedimentation process then coagulationflocculation-sedimentation process is recommended.

Filtration through sand filters is also used to remove the suspended solids that not being removed by the sedimentation process.



Water quality and treatment **Example:** A 250ml water sample is filtered through a filter paper. The dry mass of the filter paper is 54.352g. After filtration, the filter media and its filtrate were dried for 24 hours at 104°C and the dry mass of the filter paper and its non-filterable solids is 54.389g, what is the suspended solids concentration in mg/L? Soln: The mass of suspended solids = 54.389 – 54.352 = 0.037gSolids concentration = solids weight (mg)/ sample volume (L) $= 0.037 \times 1000/(250 / 1000) = 148mg/L$

Water quality and treatment **Physical parameters: Turbidity**

Turbidity in water results from the erosion of colloidal materials like the clay, silt, rock fragments, and metal oxides in the soil. Usually for water quality, the turbidity is used as indirect measure of the suspended solids content.

Water turbidity is usually measured using meters expressed in NTU (Nephelometric Turbidity Units). Such meters are based measuring the light scattered from turbid water sample. Turbid water sample



Water quality and treatment **Physical parameters: Color**

Pure water is colorless, however water from natural sources may have a color depending on the contaminants nature. If the color is produced due to suspended solids then it is an <u>apparent color</u>, while the color produced due to dissolved solids is a <u>true color</u>. Water color formed when water comes in contact with organic materials like wood and humic acids, or inorganic materials like iron oxides (red color), industrial wastes (textile, paper production ..etc).

Color in water is measured and expressed in True Color Units (TCU). Water samples are compared to standard samples using color comparison tubes.

Water quality and treatment **Physical parameters: Odor**

Pure water is odorless, however odor in water may appear due to water contact with natural substances like grass and soil, bio-reactions (algae), contact with human waste, or due to pollution from human activities like industry. Odor in water is measured using a quantitative test employing human sense and the result is expressed in units of Threshold Odor Number (TON).

$$TON = \frac{200}{A}$$

A: volume of odorous water (mL)

Water quality and treatment **Example:** A water sample is inspected by a panel of 10 persons. All persons have agreed that sample # 3 is the first where odor just detected, is this sample acceptable in terms of odor quality? Sample # 1 2 3 4 5 Volume of odorous water (mL) 20 50 80 120 150 Soln: For sample # 3, the odorous volume A = 80mL.

TON = 200/80 = 2.5 < 3 TON, therefore the water is acceptable for drinking in terms of odor.

Treatment of physical contaminants

Physical contaminant	Treatment	
Suspended solids	Sedimentation, or Coagulation-flocculation- sedimentation followed by sand filtration if needed	
Turbidity	Sedimentation	
Color, Odor and taste	Adsorption using activated carbon filters	

Water quality and treatment **Chemical parameters: definitions**

The equivalence of an element is the number of Hydrogen atoms that the element can hold in combination. It usually equals the absolute value of the valence of that element. For example the Oxygen (O) has valence of 2^- therefore its equivalence is 2.

The molecular mass of a compound is the sum of its components atomic masses. For example the molecular mass of $H_2O = 2(1) + 16 = 18g$, or 1 mole of H_2O has a mass of 18g.

An equivalent of a compound is its grams molecular mass divided by its equivalence.

Water quality and treatment **Example:**

How many grams in 1 equivalent of calcium (Ca²⁺), carbonate (CO₃²⁻), and calcium carbonate (CaCO₃)?

Soln:

1 equivalent of $CO_3^{2-} = (12+3\times16)/2 = 30g/eq$. 1 equivalent of $Ca^{2+} = 40/2 = 20g/eq$.

1 equivalent of $CaCO_3 = (40+12+16\times3)/2 = 50g/eq$

Water quality and treatment Chemical parameters: Introduction

For concentration calculations in water quality, it is more convenient to express the concentration of different materials in one standard unit. That unit is the **mg/L as CaCO₃**. For example assume water has 50mg/L of A and 20mg/L of B, and assume that the water quality parameter C = [A] + [B], then you cannot say that C = 50 + 20 = 70mg/L (you have added different concentrations of different materials).

The best way is to express [A] in **mg/L as CaCO**₃ and express [B] in **mg/L as CaCO**₃ and then find the quality parameter (C for example).

The concentration of A, [A] in mg/L as $CaCO_3 =$

 $=\frac{\left[A\right]_{mg/L}\times 50}{equivalent of A}$

Example:

Water has 55mg/L of Ca²⁺, express the [Ca²⁺] in mg/L as CaCO₃?

The equivalent of $Ca^{2+} = (40/2) = 20g/eq$.

The $[Ca^{2+}]_{mg/L \text{ as } CaCO3} = \frac{55 \times 50}{20}$ = 138mg/L as CaCO₃

Water quality and treatment Chemical parameters: Dissolved solids

Dissolved solids exist in water due to the solvent action of the water on natural materials like the soil, rocks, etc. Such solids can be organic or inorganic. Excessive amounts of the dissolved solids in drinking water may cause undesirable taste and produce color, they make the water hard, change the pH, and such excessive amounts may be toxic to humans.

The total dissolved solids content is expressed in mg/L units. While suspended solids retained as nonfilterable on a filter paper, dissolved solids pass that filter paper. The total solids in water (TS) is the sum of total suspended solids (TSS) and the total dissolved solids (TDS)

Water quality and treatment Measuring the dissolved solids

The dissolved solids content is measured experimentally in the lab. Filtering water sample of a given volume, the suspended solids retained on the filter, while the dissolved solids pass. The collected filterable solids (dissolved solids) are dried at 104°C for 24 hours and weighted. The weight of the DSS in the water sample volume is mg/L content.

Igniting the collected filterable solids at 600°C the inorganic dissolved solids remain while the organic solids evaporate as volatile materials.

The dissolved solids can be removed from water using advanced filtration (reverse osmosis).



Water quality and treatment **Example:**

100mL water sample analyzed for dissolved solids content. The evaporation dish has 327.465g net mass. After evaporation, the dish and its contents weigh 327.517g and later placed in muffle furnace at 600°C. After cooling the dish and contents weighing 327.498g, what is the total dissolved solids (TDS) content and the inorganic dissolved solids in mg/L?

Soln:

The mass of the TDS = 327.517 - 327.465 = 0.052gThe TDS concentration = 52mg/(0.1L) = 520mg/L. The mass of the inorganic dissolved solids = 327.498 - 327.465= 0.033gInorganic dissolved solids concentration = 33/0.1 = 330mg/L. Organic dissolved solids concentration = = TDS - inorganic solids = 520 - 330 = 190mg/L

Water quality and treatment Chemical parameters: Water pH

The absolute pure water has pH of 7.5, however that absolute pure water has undesirable taste to humans. Water provided from natural sources includes dissolved solids that alter the pH value and make the water taste favorable to consumers.

The desirable drinking water has pH that ranges between 6.5 - 8.5, i.e. within this range humans feel no differences in water taste.

Water with pH > 8.5 has bitter taste, while water with pH < 6.5 has acidic taste.

Water quality and treatment Chemical parameters: Alkalinity

The water alkalinity is defined as the water ability to neutralize acids, i.e. to reduce the acidic action. Studying water alkalinity is important because later drinking water will be converted to wastewater. Usually wastewater in sewer turns to be more acidic (pH<<7.5) due to anaerobic conditions as follows:

Organic matter + SO_4^{2-} + bacteria \longrightarrow H₂S + CO₂ + energy

The formation of H_2S will decrease the pH of the wastewater, therefore we prefer to have drinking water that is more alkaline (pH > 7.5) to reduce the acidic environment of the wastewater formed.

Water quality and treatment **Chemical parameters: Alkalinity** The most common compounds that form the water alkalinity are: the bicarbonate (HCO₃⁻), the carbonate (CO₃²⁻), and the hydroxide (OH⁻). Such compounds may present from origin or from CO₂ that dissolves easily in water. $CO_2 + H_2O \iff H_2CO_3$ (carbonic acid) $H_2CO_3 \iff H^+ + HCO_3^-$ (bicarbonate) $HCO_3^- \iff H^+ + CO_3^{2-}$ (carbonate) $HCO_3^- \iff H^+ + CO_3^{2-}$ (carbonate) $CO_3^{2-} + H_2O \iff HCO_3^- + OH^-$ (hydroxide)

Measuring the water alkalinity

Alkalinity is measured by titrating the water sample with standard acid (usually $0.02N \text{ H}_2\text{SO}_4$). <u>Based on</u> <u>1L water sample, each 1mL of the acid is equivalent</u> to <u>1mg of alkalinity expressed as mg/L as CaCO_3</u>.

Example:

A 1L water sample was titrated with 250mL of 0.02N H₂SO₄, what is the measured alkalinity?

Measured alkalinity = 250 mg/L as CaCO₃.



Measuring the water alkalinity

Referring to the alkalinity titration curve, it is clear that mL acid $(0.02N \text{ H}_2\text{SO}_4)$ used to bring down the water pH to a value of 4.5 equals <u>the total alkalinity</u> of the water sample that has 1L volume.

Total alkalinity = $[HCO_3^{-1}] + [CO_3^{2-1}] + [OH^{-1}]$

Referring to the alkalinity titration curve, it is clear that if the water sample pH is ≤ 8.3 , then the water total alkalinity is due to the bicarbonate (HCO₃⁻), can you see that from the alkalinity titration curve?

Water quality and treatment

Impact of water alkalinity

The impact of high alkalinity (due to high OHconcentrations, i.e. pH > 8.5) on water appears as a bitter taste which could be unfavorable to water consumers. Therefore always keep the water pH in the range 6.5 - 8.5.

Moreover, the presence of alkalinity forming compounds may react with the Ca^{2+} resulting in precipitated substances, called the scale, that may foul (clog or close) pipes in heating systems and may cause damage to boilers in electricity generating units.

Example

A 250mL water sample has an initial pH of 8. The water titrated with 22mL of $0.02N H_2SO_4$ to pH value of 4.5, what is the water total alkalinity?

Soln:

The 22mL of acid neutralized the alkalinity in the 250mL sample, then the total alkalinity is:

 $= (22mg/250mL)\times(1000mL/L) = 88mg/L$ as CaCO₃

Question: what alkalinity forming compounds exist?



Soln:

pH = -Log[H] and pOH = -Log[OH] pH + pOH = 14, then pOH = 14 - 10 = 4Then $[OH] = 10^{-4}$ mole/L $[OH]_{g/L} = 17g/mole \times 10^{-4}$ mole/L = 0.0017g/L $[OH]_{mg/L CaCO3} = 1.7 \times 50/17 = 5mg/L$ as $CaCO_3$

The [OH] alkalinity = 5 mg/L as CaCO_3 (remember that the alkalinity measured using the $0.02N \text{ H}_2\text{SO}_4$ is for 1L sample). Therefore the actual mL acid used to equalize the OH⁻ alkalinity in the 200mL sample = $5 \text{mL} \times (200/1000) = 1 \text{mL}$

Water quality and treatment

From the 11mL used to bring the pH to 8.3, 1mL used for the OH⁻ alkalinity, so 11 - 1 = 10mL used for the first 1/2 of the CO₃²⁻. The other 1/2 of the CO₃²⁻ used 10mL also, therefore in the 200mL sample the total CO₃²⁻ alkalinity was equalized by 10 + 10 = 20mL acid (10 for each 1/2), so the CO₃²⁻ alkalinity is 20mg in the 200mL.

The $[CO_3^{2-}]_{mg/L CaCO3} = (20mg/200mL) \times (1000mL/L)$

= 100mg/L as CaCO₃

The HCO₃⁻ alkalinity used 9mL (30 – 1 – 20), so its alkalinity is 9mg in 200mL, then the [HCO₃⁻] $_{mg/L CaCO3}$ =

 $= (9mg/200mL) \times (1000mL/L) = 45mg/L$ as CaCO₃

Total alkalinity = $[HCO_3^{-}]+[CO_3^{2-}]+[OH^{-}] = 150mg/L$ as CaCO₃

Water quality and treatment Chemical parameters: Hardness

Scientifically hardness is defined as the concentration of the multivalent cations, usually Ca^{2+} and Mg^{2+} . Hardness is also defined as the water ability to precipitate soap. In that case, soap compounds combine with Ca^{2+} and Mg^{2+} forming very complex compounds that do not stay in suspension and finally precipitate. Since that complex compound do not stay in suspension, the cleaning action of the soap dissipates causing consumers to add more soap. The more soap is added, the more phosphorus is added, i.e. more pollutants are added hence decreasing the quality of wastewater produced.

Water quality and treatment Chemical parameters: Hardness

Hardness causing species in water are not toxic to humans, however a lot of soap is consumed when hard water is used. Furthermore under super-saturation conditions hardness causing species react with anions in water forming a solid precipitate: $CaCO_3$ or $Mg(OH)_2$. That solid precipitate is called scale.

The formation of scale in boilers and pipes will cause economic losses, i.e. more heating will be required and will usually damage or clog pipes and boilers in water systems.

Water quality and treatment Chemical parameters: Hardness

The scale forms as follows:

 $Ca^{2+} + CO_3^{-} \xrightarrow{heat} CaCO_3$ $Mg^{2+} + 2OH \xrightarrow{-high pH} Mg(OH)_2$

Hard water contains high concentrations of Ca^{2+} and Mg^{2+} . At high temperatures the carbonate (CO_3^{2-}) reacts easily with the Calcium (Ca^{2+}) forming the scale $CaCO_3$.

At high pH levels, the OH⁻ combines with the Mg^{2+} forming the scale $Mg(OH)_2$.

Water quality and treatment

Chemical parameters: Hardness

The water total hardness = $[Ca^{2+}] + [Mg^{2+}]$ in units of mg/L as CaCO₃. The total hardness also equals the carbonate hardness + the non-carbonate hardness. The hardness that equivalent to the alkalinity is called the carbonate hardness. Non-carbonate hardness forms when Ca²⁺ or Mg²⁺ combines with compounds like SO₄²⁻ or Cl⁻.

Hardness classification:

Soft water has hardness < 50mg/L as CaCO₃. Moderately hard water has hardness 50 - 150 mg/L as CaCO₃. Hard water has hardness 150 - 300mg/L as CaCO₃. Very hard water has hardness > 300mg/L as CaCO₃.

Water quality and treatment **Example:**

Water sample was analyzed for dissolved solids as shown by the following contents:

 $Ca^{2+} = 100mg/L$

 $Mg^{2+} = 36.6mg/L$

 $HCO_{3}^{-} = 300 mg/L$

Calculate the total hardness, the carbonate hardness and the non-carbonate hardness.

Classify water in terms of hardness?

Water quality and treatment

Soln:

The total hardness = $[Ca^{2+}] + [Mg^{2+}]$

 $[Ca^{2+}] = (100 \times 50)/20 = 250 \text{ mg/L} \text{ as } CaCO_3.$

 $[Mg^{2+}] = (36.6 \times 50)/12.15 = 150 \text{ mg/L} \text{ as } CaCO_3.$

The total hardness = 400 mg/L as CaCO₃ that is > 300 mg/L as CaCO₃, therefore the water is classified as very hard.

The carbonate hardness is equivalent to the total alkalinity, therefore the carbonate hardness = $[HCO_3^{-}]$

 $= (300 \times 50)/61 = 246$ mg/L as CaCO₃.

Soln:

The carbonate hardness = 246mg/L as CaCO₃.

The non-carbonate hardness =

= total hardness – carbonate hardness

= 400 - 246 = 154 mg/L as CaCO₃.

Hard water can be softened (treated) by the partial or full removal of calcium and magnesium cations. Reverse Osmosis (RO filter) is used to remove calcium and magnesium cations from water. Water is softened by adding the lime $Ca(OH)_2$ for the calcium and magnesium carbonate hardness removal and by adding the soda Na_2CO_3 for the calcium and magnesium non-carbonate hardness removal.

Water quality and treatment

Chemical parameters: Fluoride

Fluoride originates from few types of sedimentary or igneous rocks. It is rarely found in the surface water while few ground water sources may contain fluoride at low concentrations.

Fluoride at high concentrations is toxic to humans while low fluoride concentration (around 1mg/L) is useful to prevent dental cavities.

Water quality and treatment Chemical parameters: Metals

Generally all metals are soluble in water to some degree. Some metals at low concentrations are very toxic. The non-toxic metals at high concentrations may cause health problems.

Non-toxic metals: like calcium, magnesium, iron, sodium, manganese, aluminum, copper and zinc. Excessive amount of the non-toxic metal salts like sodium salts may cause bitter taste to water and cause health problems. Manganese and iron salts can cause color problem.

Toxic metals: like arsenic, barium, cadmium, lead, mercury, and silver. (All are very toxic at low amounts)

Water quality and treatment Chemical parameters: Organics

Organics may present in surface waters naturally due to the run off or some human activities. Organics can be either biodegradable or non-biodegradable.

Biodegradable organics: found in dissolved form like starches, fats, proteins, and acids. Such compounds present a food source to microorganisms and can be degraded easily with time.

Non-biodegradable organics: found in dissolved form like cellulose and phenols. Such compounds resist biological degradation and stay dissolved in water for long time.

Water quality and treatment Biological parameters: Pathogens

For human water use and consumption, the most important biological parameters for water quality are pathogens. Such microorganisms are capable to infect humans or transmit diseases.

Waterborne pathogens include viruses, bacteria species, protozoa, and helminthes (parasitic worms). The most common diseases caused by pathogens are: Cholera, Typhoid, and fever.

Water disinfection

Chlorination:

The addition of specified amounts of chlorine to water for the purpose of disinfection is called chlorination. The whole process is about killing all microorganisms and retarding its re-growth rate if microorganisms removal is not 100%.

At low concentrations, chlorine is not harmful to humans compared to microorganisms. It penetrates the microorganism cell wall and reacts with its enzymes leading at microorganism death. At high concentrations complete oxidation of the cell wall occurs leading to microorganism death.

Water disinfection

Chlorination:

Chlorine is added to water in the form of gas (Cl_2) , $Cl_2 + H_2O \longrightarrow H^+ + HOCl (hypochlorous acid)$ or in the form of ionized solids { $Ca(OCL)_2$, NaOCl} $Ca(OCL)_2 \longrightarrow Ca^{2+} + 2OCl^-$ (hypochlorite) NaOCL \longrightarrow Na + OCl⁻ The hypochlorous acid and the hypochlorite are related as follows:

HOC1 \longleftrightarrow H⁺ + OCl⁻ (pH dependent, how??)

Water disinfection

Chlorination:

The sum of the hypochlorous acid (HOCl) and the hypochlorite (OCl⁻) are is called the free chlorine residual. The HOCl is more effective disinfectant than the OCl⁻ by two orders of magnitude. The process of water disinfection using chlorine is affected by the:

- 1- Form of chlorine
- 2- pH and temperature
- 3- Concentration
- 4- Contact time
- 5- Type of microorganism.

Water disinfection Time extension of chlorination:

The longer the time the chlorine exists the more protection against microorganisms or their re-growth is provided. The chlorine is a strong oxidizing agent that reacts easily with a lot of elements or salts exist naturally in water, ultimately the free chlorine (Cl_2) will be consumed forming compounds with no disinfection ability.

The question is: How can we extend the chlorine existence time in water systems? The answer: using Ammonia (NH_3). It can be either added or naturally exist. It reacts with the chlorine forming <u>stable</u> <u>compounds</u> of disinfecting ability although less than the free chlorine.

Water disinfection **Determine the chlorine:** The following reactions occur: $M_3 + HOCl \longrightarrow NH_2Cl \text{ (monochloramine)} + H_2O$ $M_2Cl + HOCl \longrightarrow NHCl_2 \text{ (diochloramine)} + H_2O$ $MHCl_2 + HOCl \longrightarrow NCl_3 \text{ (trichloramine)} + H_2O$ The monochloramine is persistent (durable and stable) therefore serves as disinfectant for long time. Usually the more chlorine is added at low pH values then all the monochloramine will be converted to trichloramine that causes taste and smell to water and cause irritation to eyes in swimming pools. It is preferable to prevent the formation of the trichloramine (NCl_3). At pH values (around 6.5 or more) only monochloramine is formed.



Water disinfection Time extension of chlorination:

It should be noted that chlorine (the strong oxidizing agent) can react with organic acids (humic and fulvic) forming <u>chlorinated hydrocarbons</u> that is classified as <u>carcinogenic</u> materials. Therefore such materials should be removed prior to the chlorine addition through ordinary treatment processes (adsorption by activated carbon filters). In case that prior treatments fail to remove all organic materials the addition of Ammonia will simply prevent the formation of such compounds. Because Ammonia strongly reacts with the chlorine forming NH₂Cl (monochloramine) that plays as disinfectant type of chlorine compounds.

Wastewater Treatment Engineering

11401355

Wastewater flow quantity Instructor: Dr. Zeyad Tarawneh

Wastewater flow quantity

Introduction

Drinking water consumed by humans through domestic or industrial activities will turn into wastewater that carries different pollutants. Wastewater treatment is necessary because treated wastewater is potential water source besides protecting the environment from:

- high organic and inorganic loads
- groundwater pollution
- contamination with pathogens

The degree to which the wastewater is loaded with contaminants determine the treatment technology that will be used.

Wastewater flow quantity

Wastewater sources

Wastewater is generated from three main sources: Domestic source, industrial source, and infiltrated water from rain or ground water table.

Domestic source: primarily from residential, commercial, recreational activities, and public service buildings. This source is generated at a rate of around 80% of the domestic water consumption. Such wastewater flow contains mainly organic matters, nutrients (Nitrogen & Phosphorus), and pathogens.

Wastewater flow quantity

Wastewater sources

Industrial source: wastewater from food production and paper industries mostly contains organic matter. Some industries generate wastewater that is heavily loaded with inorganic matter (heavy metals).

Infiltrated water to sewers: primarily as inflow water that inters the sewer system from ground water table or storm rain. Infiltrated water may enter the sanitary sewer from cracked pipes or manholes. Such water does not contain organic loads. The quantity of the infiltrated water depends on the sewer age, rainfall amount, and variation in the water table.

Wastewater flow quantity **Relation of quantity to population**:

For wastewater conveying and treatment, future population estimates based on scientific methods that capture population growth patterns is a key point for design. Usually the population economic level, the climate, the population density, and the degree of urbanization are factors affecting water consumption rates and hence amounts of wastewater generated. For example, high economic level, hot climate, and urbanized regions consume water at high rates, therefore huge wastewater amounts will be generated.

Wastewater flow quantity

Population estimation

For a given design period, wastewater flow determination requires reasonable estimation of the population at the end of the design period. Prediction of future population depends on past and current population that can be obtained from statistics department. The following methods are used to estimate the future population:

- 1- The arithmetic growth method
- 2- The uniform percentage method
- 3- The logistic method
- 4- The declining growth method

Population estimation

The arithmetic growth method:

The arithmetic growth assumes that the population growth rate (K_a) is constant over time. Mathematically it is expressed as:

$$\frac{dP}{dt} = K_a$$

P: population, *t*: time, and K_a : constant growth rate.

Integrating *P* over *t*, then:

$$P_t = P_0 + K_a \Delta t$$

 P_0 : the current population, P_t : the future population over Δt years.

Population estimation

Given a current population (P_0) and past population (P_1), the constant K_a is estimated as follows:

$$K_a = \frac{\Delta P}{\Delta t} = \frac{P_0 - P_1}{t_0 - t_1}$$

Question:

The population at 1990 and 2011 is 22000 and 27400 person, respectively, estimate K_a assuming arithmetic growth pattern?

Population estimation

Example:

The population of a city has been recorded in years 1990 and 2005 as 100000 and 110000, respectively. Estimate the year 2015 population assuming arithmetic growth pattern exists?

Soln: $P_0 = 110000$, $P_1 = 100000$

$$K_a = \frac{110000 - 100000}{15} = 667$$

$$P_{2015} = 110000 + (667)(10) = 116670$$

Population estimation

The uniform percentage growth method:

The uniform percentage growth method assumes a rate of increase that is proportional to the population, mathematically it is written as:

$$\frac{dP}{dt} = K_u P$$

 $K_u P$: increase in growth rate.

Integrating *P* over *t*, then:

$$\ln P_t = \ln P_0 + K_u \Delta t$$

Population estimation **Example:**

The population of a city has been recorded in years 1990 and 2005 as 100000 and 110000, respectively. Estimate the year 2015 population assuming uniform percentage growth pattern exists?

Soln: P₀ = 110000, P₁ = 100000

$$K_u = \frac{ln110000 - ln100000}{15} = 0.0064$$

 $ln P_{2015} = ln 110000 + (0.0064)(10) = 11.672$

$$P_{2015} = 117216$$





Population estimation The logistic growth method: Mathematically, the logistic growth population estimate over Δt is: $P_t = \frac{P_{sat}}{1 + e^{a+b\Delta t}}$ $P_{sat} = \frac{2P_0P_1P_2 - P_1^2(P_0 + P_2)}{P_0P_2 - P_1^2}$ $a = ln \frac{P_{sat} - P_0}{P_0} \qquad b = \frac{1}{n}ln \frac{P_2(P_{sat} - P_1)}{P_1(P_{sat} - P_2)}$ *n*: time interval between the 3 succeeding population data (P_0, P_1 and P_2)

Population estimation **Example:**

The population of a city has been recorded in years 1990, 2000, and 2010 as 20000, 28000, and 33000 respectively. Estimate the 2020 population assuming logistic growth pattern exists?

Soln: P₀=33000, P₁=28000, P₂=20000 $P_{sat} = \frac{2 \times 33000 \times 28000 \times 20000 - (28000)^2 (33000 + 20000)}{33000 \times 20000 - (28000)^2}$ $P_{sat} = 37032 \qquad a = -2.1022 \qquad b = -0.0971$ $P_{2020} = \frac{37032}{1 + e^{-2.1022 + (-0.0971)(10)}} = 35380$

Population estimation

The declining growth method:

Like the logistic growth, it assumes the city has limiting saturation population. Given P_{sat} (from statistics department), mathematically it is expressed as:

$$\frac{dP}{dt} = K_d \left(P_{sat} - P \right)$$
$$P_t = P_0 + \left(P_{sat} - P_0 \right) \left(1 - e^{K_d \Delta t} \right)$$
$$K_d = -\frac{1}{n} ln \frac{P_{sat} - P_1}{P_{sat} - P_0}$$

n: time interval between succeeding population data (P_0 and P_1)

Population estimation

Example:

The population of a city has been recorded in years 2000 and 2010 as 28000 and 33000, respectively. Given that P_{sat} as 39500, estimate the 2020 population assuming declining growth pattern exists?

Soln: P₀=33000, P₁=28000

$$K_d = -\frac{1}{10} ln \frac{39500 - 28000}{39500 - 33000} = -0.057$$

 $P_{2020} = 33000 + (39500 - 33000)(1 - e^{-0.057 \times 10}) = 35820$

Population estimation

What type of growth pattern?

What type of growth pattern shall be used to estimate the population if it is required to calculate the wastewater flow?

The answer:

Check graphically what form of growth does the population in the city take (arithmetic, uniform, logistic, etc...). Moreover, it is required to refer to information provided by the statistics department in relation to the existence of limiting saturation population.

Population estimation

In the case that the information provided by the statistic department in relation to the population data does not guide towards a certain type of growth, what can be done?

The following equation requires only an annual average growth percentage (for example 3%) that can be easily obtained (simulation can be used in this case). The relation is:

$$P_t = P_0(1 + K_p t_d)$$

Where K_p is the annual growth percentage, t_d : design period.

Population estimation

Question:

Using the arithmetic growth method the estimated population for the year 2025 is 24280 capita based on a current population of 17995 capita for the year 2010. Use the assumption of uniform percentage growth pattern to re-estimate the year 2025 population using time interval (Δt) of 15 years?
Wastewater flow quantity Daily variations in wastewater flow:

In designing wastewater systems, the determination of design flows is an essential step. During the day, the wastewater generated is related to users activities. In morning hours (6 – 8am), users consume water at peak rate generating wastewater at peak flow. During the period 8 - 12am, the water consumption is usually below average generating non-peak wastewater flow when most of users are at work or schools. Another peak wastewater flow appears during noon hours (3 – 5pm) when all users are at home conducting daily activities (cooking, cleaning, etc).



Wastewater flow quantity **Variation in wastewater flow:**

The variation in flow occurs during the day from minimum to maximum value. Furthermore, during the week, peak days and non-peak days occur, and so during the year there are peak months and non-peak months in terms of generated wastewater flows.

Such peak values can be obtained from records (water company), however in the absence of data the **Goodrich formula** can be used to estimate the peak factor (P_f) for desig purposes given the annual average rate of wastewater flow:

$$P_f = 1.8t^{-0.1}$$

Wastewater flow quantity

 $P_f = 1.8 t^{-0.1}$

In the Goodrich formula P_f is the peak factor and t is the time in days. For example the peak day flow factor is obtained by placing t = 1 day, therefore $P_f = 1.8$, while the peak factor in weekly basis is 1.48, and the monthly peak factor is 1.28

From experience, the peak hour flow is 1.5 of the average wastewater flow for that day, and the minimum hour flow rate is 0.5 of the average wastewater flow for that day.

Wastewater flow quantity

Example:

Calculate the wastewater flow that can be used for design purposes for a community generates an average annual wastewater flow at a rate of 100L/c/day?

Soln:

The peak day flow = $100 \times 1.8 \times 1.5 =$ = 270L/c/day

The value of 270L/day is the peak flow for design purposes.

Wastewater flow quantity

When data is unavailable, then around 80% of the domestic water consumption rate is returned as wastewater to the sanitary sewers.

Average Wastewater Flows from Residential Sources			
Source Unit Flow, L/unit-day			
		Range	Typical
Apartment	Person	200 - 340	260
Hotel, resident	Resident	150 - 220	190
Individual Dwellings		•	
Average Home	Person	190 - 350	280
Better Home	Person	250 - 400	310
Luxury Home	Person	300 - 550	380
Semimodern Home	Person	100 - 250	200
Summer Cottage	Person	100-240	190

Average Wastewater Flows from Commercial Sources			Average Wastewater Flows from Commercial Sources (cont.)					
Source	Unit	Flow, L/unit-day]				
		Range	Typical	Source	τ	Juit	Flow, L/u	uit-day
Airport	Passenger	8-15	10				Range	Typical
Automobile Service	Vehicle served	30 - 50	40	Laundromat		fachine	1800 - 2600	2200
Station	Employee	35 - 60	50		v	Wash	180 - 200	190
Bar	Customer	5-20	8	Motel	P	erson	90 - 150	120
	Employee	40 - 60	50	Motel with	Kitchen P	erson	190-220	200
Hotel	Guest	150 - 220	190	Office	E	Imployee	30-65	55
To Acatal Deciding	Empioyee	30-50	40	Restaurant	N	deal	8-15	10
(excluding industry & café)	Empioyee	30 - 03	- 22	Rooming H	louse R	lesident	90 - 190	150
	Source		Sou	rces (con	t.)	mit-dav		
			-	Range	Typical			
	Store D	Store, Department Toil Emp		let room	1600 - 2400	2000		
				nlowee	30 - 50	40		
				pioyee	30-30			
	Shopping Center Parl		king space	2-8	4			
	onopping							





Wastewater flow quantity variation in wastewater characteristics

The characteristics of wastewater vary from place to place depending on amount of the freshwater consumed, and from time to time due to biological and chemical reactions that usually take place as time proceeds. For example, as sewage is transported in the collection system, bacteria will degrade the organic material changing its state from solid to soluble as time proceeds.

Ultimately wastewater characteristics change with time and place and that should be considered when test samples are obtained.

Wastewater Treatment Engineering

110401455

Wastewater Sewers Instructor: Dr. Zeyad Tarawneh

Wastewater sewers Introduction

Sanitary sewers are open channel conduits that convey the wastewater to the treatment plant or to an open disposal site. The sewer system conveys the wastewater benefiting of the gravitational force, so it is called gravity sewer systems. Sanitary sewer systems are usually designed as separate systems from storm sewers.

The open channel hydraulics is used to design and evaluate the performance of sanitary sewers. The Manning equation is commonly used to design and evaluate the performance of sanitary sewers.



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Problem related to sewers: corrosion

Corrosion in sewer pipes usually occurs when conveying wastewater of low pH. The acidity of the low pH wastewater attacks the pipe material (cement structure) causing severe damage to sewer pipes. Such process is known as the sulfide attack.

To retard (delay) the problem of sewers corrosion, the pH of the wastewater flow should be kept a little high (> 7.5). How can that be achieved?











Design of sewers

To design sewer system for given region, the design requires a map that shows the elevations (contours), the streets layout, and the land use.

Design requirements:

• Pipes are usually placed underneath minor and major streets ensuring that every single house or public service building is served by sewer pipeline.

- The minimum pipe cover is 2m.
- Buildings are connected directly to the nearby pipe.

Design of sewers

Design criteria:

• The minimum flow velocity in any pipe is 0.6m/s to ensure pipe self cleaning, while the maximum velocity is 3.5m/s to avoid pipe damage (erosion).

• The pipe bed slope should be between maximum slope of 0.05 and minimum slope according to the pipe flow. The minimum bed slope calculated based on minimum tractive shear stress of 1pa is given as:

$$S_{o-min} = 0.000233 \ Q^{-6/13}$$

Q: pipe design flow (m^3/s) .

 S_{o-min} not less than 0.5%.

Wastewater sewers

Sewer manholes

Given the region map with sewer pipes being placed, manholes are usually introduced for inspecting and cleaning sewer pipes.

A manholes is placed when there is:

- change in the horizontal direction of the sewer line.

- change in the vertical direction of the sewer line (change in the bed slope).

- change in the sewer size, and

- at intervals of 100m when the sewer has no change in direction.







Design steps:

First:

For the sewer pipe connecting two manholes, estimate the initial sewer bed slope (S_o) as follows:

$$S_o = (GE_U - GE_L)/H.$$

 GE_U : the ground elevation at the upper manhole,

 GE_L : the ground elevation at the lower manhole,

H: the pipe horizontal projected distance (on the map). It equals the distance between the edges of the upper and the lower manholes.

Compare S_o with S_{o-min}

Design steps:

Second:

For the sewer pipe connecting two manholes, estimate the average wastewater flow (m³/s) according to the number of units being served. The flow of the peak hour in the peak day is selected as the sewer design flow (Q_d).

$$Q_d = Q_{average} \times 1.5 \times 1.8$$

Wastewater sewers **Design steps:** <u>Third</u>: For the sewer pipe connecting two manholes, the initial pipe diameter (*d*) can be obtained using the Manning equation assuming the pipe is just full to carry the design flow (*Q_d*). $Q_d = \frac{1}{n} \frac{\pi}{4} d^2 \left(\frac{d}{4}\right)^{2/3} \sqrt{S_o}$ $d = \left(\frac{3.21nQ_d}{\sqrt{S_o}}\right)^{3/8}$

Wastewater sewers		
Design steps:	Size (in)	Size (mm)
Commercial pipe sizes	8	200
	10	255
	12	305
	15	380
	18	460
	21	530
	24	615
	27	685
h	30	780
ne minimum pipe size is 8	33	840
	36	915
	42	1070
-	48	1228
	54	1370



Wastewater sewers **Design steps:**

Fourth:

If *d* is less than 8" then the pipe final diameter *D* is 8". If *d* is not among the listed commercial sizes, then *D* is selected as the next larger size. For example, if the calculated initial pipe diameter (*d*) is 403mm, then the next larger size of commercial pipes is the final sewer diameter (D = 460mm). Since *d* is always less than *D*, then Q_d is always different from Q_f (flow rate at the full pipe) when using pipe of size *D*.

$$Q_f = \frac{1}{n} \frac{\pi}{4} D^2 \left(\frac{D}{4}\right)^{2/3} \sqrt{S_o}$$

Wastewater sewers **Design steps:**

Fifth:

Use the partial flow diagram to estimate the partially full flow velocity V to check for the minimum flow velocity. V_{min} is the minimum flow velocity (usually 0.6 m/s) required such that the sewer is kept under self cleaning conditions (the flow flushes the accumulated solids by traction).

If V is less than 0.6m/s, then one solution is to increase the sewer bed slope (S_o) if that is possible.







Wastewater sewers **Design example:**

The initial bed slope is 0.02, $Q_d = 5000 \text{m}^3/\text{day} = 0.0579 \text{m}^3/\text{s}$, then the initial pipe size *d* is:

$$d = \left(\frac{3.21 \times 0.015 \times 0.0579}{\sqrt{0.02}}\right)^{3/8} = 0.229m$$

Since the initial diameter d = 229mm is not available commercially, then the next larger commercial pipe size (255mm) is selected as the sewer final diameter.

D = 255mm.







The performance during the min flow period:

The minimum flow occurs during the non-peak hours within an average day.

 $Q_{average} = Q_{non-peak} / (1.5 \times 1.8) = 0.0214 \text{ m}^3/\text{s}.$

$$Q_{non-peak} = 0.5 \times Q_{average} = 0.5 \times 0.0214 = 0.0107 \text{ m}^3/\text{s}.$$

$$Q_{non-peak}/Q_f = 0.0107/0.0768 = 0.14$$

From partial flow diagram, $V_{non-peak}/V_f = 0.7$

Therefore, $V_{non-peak} = 0.7 \times 1.5 = 1.05$ m/s



Wastewater sewers **Question:**

Design a sewer pipe that links an upper manhole (GE_U = 871m AMSL) to a lower manhole (GE_L = 871m AMSL). The sewer pipe is concrete (n = 0.015) and will convey peak flow of 5000m³/day. The horizontal distance between the two manholes is 100m.

Wastewater Treatment Engineering

110401455

Wastewater characteristics Instructor: Dr. Zeyad Tarawneh

Wastewater characteristics

Introduction

In general about 99% of the sewage mass is pure water. The little remaining part (1%) has a very significant effect on the nature of the sewage. Therefore studying the characteristics of the wastewater is an essential step for a proper selection of the wastewater treatment technology. For example, raw wastewater with high suspended solids content can be treated initially by removing considerable of the suspended solids using plain amount sedimentation process.

The characteristics of the wastewater are classified into: physical characteristics, chemical characteristics, and biological characteristics.

Physical characteristics: Appearance

The fresh (newly produced) domestic sewage has a cloudy color (light gray) and has slightly soapy odor. Usually the solids in the fresh sewage is characterized by large sizes.

Later due to biological and chemical reactions that occur as the time elapses, the aged sewage becomes dark gray in color with the odor of hydrogen sulfide (rotten egg), while the solids size becomes smaller compared to the fresh sewage solids size due to degradation

Wastewater characteristics

Physical characteristics: Appearance

The physical change from fresh to aged sewage takes usually 2 - 6 hours at temperature of 20°C depending on the concentration of the organic materials.

The concentration of the organic material is a function of the water consumption. When the water consumption rate is relatively low, the wastewater is strongly concentrated with organic materials, and that could be the case of wastewater in countries with low water consumption rates, for example Jordan.

Physical characteristics: Solids

The total solids in the wastewater are a mix of suspended (>1 μ m), colloidal (0.001 - 1 μ m) and dissolved (<0.001 μ m), however as time proceeds the microbial activities will reduce the size of solids, for example some of the suspended and colloidal solids will be changed to dissolved solids.

The total solids in the wastewater is the sum of the suspended, colloidal, and dissolved solids. Such solids can be volatile (organic origin) or non-volatile (inorganic solids). The concentration of the volatile solids is a rough measure of the organic material concentration.



Example:

A city discharges wastewater flow at a rate of 5000 m^3 /day. The total solids content in that wastewater is 1200mg/L, calculate roughly the per day mass of the settleable solids.

Soln:

The concentration of the settleable solids = = $(22.2/100)\times(1200) = 266.4 \text{ mg/L}$

The per day mass = $5000 \times 1000 \times 266.4 = 1332 \times 10^{6} \text{ mg}$ = 1332 Kg

Wastewater characteristics Chemical characteristics: BOD and COD

Wastewater contains both organic and inorganic materials. Usually the inorganic matter like the nitrogen compounds (ammonia) and the phosphorus can be removed from the wastewater using advanced treatment techniques (called the tertiary treatment).

The organic matter in the wastewater usually, like proteins and fats, is related to human activities. The quantity of the organic matter is measured indirectly by measuring the amount of the Oxygen required to convert (oxidize) all the organic matter to H_2O and CO_2 . The Biological Oxygen Demand (BOD) and the Chemical Oxygen Demand (COD) are techniques used to measure such Oxygen amount.

Wastewater characteristics Chemical characteristics: pH and alkalinity

Microorganisms consume oxygen in wastewater to oxidize the organic material to H_2O and CO_2 for their growth, however when the oxygen is totally consumed then microorganisms anaerobically oxidize the organic matter releasing acids that lower the pH, hence as sewage ages, its pH reduces more. Most of the microorganisms prefer low pH environment for their work.

The presence of alkalinity in the wastewater is preferable since it provides a buffering action against acids produced by the microbial action. The presence of alkalinity reduces the anaerobic microbial activities in the wastewater.

Wastewater characteristics Biological characteristics: Pathogens

Wastewater should be treated for the complete removal of pathogen that may cause waterborne diseases like dysentery, cholera, typhoid, and fever.

The total Coliform and Faecal Coliform microorganisms usually indicate the faecal pollution as a result of human excreta. Such microorganisms are counted in the treated wastewater and the count is used as an indicator for heath risk (pathogens probably present). Guidelines for re-using the treated wastewater requires less than 1000 FC/100mL of treated wastewater to be used in unrestricted irrigation (safe to eat crops, sport fields and public parks).

Constituent	Concentration mg/l			
	Strong	Medium	Weak	
Total solids	1200	700	350	
Dissolved solids (TDS)	850	500	250	
Suspended solids	350	200	100	
Nitrogen (as N)	85	40	20	
Phosphorus (as P)	20	10	6	
Chloride	100	50	30	
Alkalinity (as CaCO ₃)	200	100	50	
Grease	150	100	50	
BOD ₅	300	200	100	

Wastewater characteristics Wastewater in Jordan

Constituent	Concentration mg/
Dissolved solids (TDS)	1170
Suspended solids	900
Nitrogen (as N)	150
Phosphorus (as P)	25
Alkalinity (as CaCO ₃)	850
Sulphate (as SO ₄)	90
BOD ₅	770
COD	1830

Wastewater characteristics The Biological Oxygen Demand (BOD)

The BOD is defined as the amount of Oxygen needed by microbial activities to fully degrade (consume) just the organic material to convert it to H_2O and CO_2 .

The following reaction occurs in wastewater when oxygen presents (called aerobic condition),

Organic matter + Bacteria + $O_2 \longrightarrow CO_2 + H_2O$ + Energy

Wastewater characteristics **The Biological Oxygen Demand (BOD)**

The BOD is used to indicate how much the wastewater is loaded with organic materials (indicates the organic pollution degree). High BOD values indicate high loads of organic material in the wastewater. The BOD is a measure used to determine the proper wastewater treatment technique and is used to design treatment plants and also to evaluate the quality of the treated wastewater.

The BOD for wastewater is measured experimentally in the lab (usually for wastewater sample of 5 days age at 20°C).

Wastewater characteristics **BOD exertion**

The BOD exerted (Oxygen used) after (t) days can be calculated theoretically assuming that the organic material is consumed by microbes following a first order reaction of constant rate (k). The BOD_t is calculated as follows:

 $BOD_t = BOD_{\infty}(1 - e^{-kt})$

k: reaction rate constant (usually = 0.23/day at 20°C) BOD_{∞}: the ultimate amount of Oxygen (mg/L) needed to fully convert (consume) all the organic material (just the organic material).

Wastewater characteristics **BOD plot**

Plot the BOD at temperature of 20°C versus time for 20 days assuming that BOD_{∞} is 439mg/L.

Using $BOD_t = BOD_{\infty}(1 - e^{-kt})$ with k = 0.23/day at 20°C then the following table shows the results for the first 5days (finish the rest)

t (days)	BOD _t (mg/L)
0	0
1	90
2	162
3	219
4	264
5	300
	·



Wastewater characteristics The BOD₅

It is a standard measure (Universal measure) of the Oxygen amount needed to convert the organic matter over continuous 5 days at 20°C.

From the BOD_t equation it is clear that the BOD₅ represents about 68% of the BOD_{∞}.

The reaction rate constant (k) is a function of the temperature (T). Given k_{20} , the reaction rate constant (k_T) at any temperature T is obtained as follows:

$$k_{\rm T} = k_{20} [1.047^{(\rm T-20)}]$$

Wastewater characteristics **Example**

Calculate the BOD at 30°C after 3 days given that the BOD₅ is 400mg/L? Soln: The constant k at 30°C is: $k_{30} = 0.23 \times [1.047^{(30-20)}] = 0.364/day$ The BOD_∞is: $400 = BOD_{\infty}(1 - e^{-0.23 \times 5})$ so $BOD_{\infty} = 585.3 \text{ mg/L}$ and $BOD_{(3days,30^{\circ}C)} = 585.3 \times (1 - e^{-0.364 \times 3}) = 388.9 \text{mg/L}$ What do you observe?

Wastewater characteristics **Problem related to the BOD**

To control the treatment processes in wastewater treatment plants, the BOD test may not be an appropriate indicator for the quality of treated wastewater specially if a decision is required to be taken within few hours, recall that useful BOD values needs minimum 5 days.

Alternatively, the Chemical Oxygen Demand (COD) is used to indicate the treated wastewater quality. It measures chemically the Oxygen amount needed to oxidize (convert) the organic materials and some inorganic materials. The COD test requires less than 1 hour to estimate the amount of oxygen needed compared to the 5 days test (BOD).

Wastewater characteristics **Problem related to the COD**

The COD determination test uses usually a strong acid to oxidize the organic material (that is the idea), however the strong acid will oxidize also part of the inorganic materials exist naturally in the wastewater like Sulfur (S_2), therefore the COD test usually gives higher Oxygen demand values compared to the more accurate but time consuming test like the BOD.

Generally the BOD measure is used in designing treatment plant and for the long run effluents control, while the COD is used mainly for the daily control on the treated wastewater.

Wastewater characteristics Nutrients effect

Municipal wastewater contains usually considerable amounts of nutrients like Nitrogen (N) and Phosphorus (P). The presence of such nutrients affect significantly the aquatic life causing eutrophication (algal growth) in surface water bodies. Eutrophication will eventually cause fish death due to lowering the dissolved Oxygen content, and cause odor nuisance (H₂S) at night due to anaerobic conditions.



Wastewater characteristics Nutrients effect

All microorganisms requires nutrients, Nitrogen (N) and Phosphorus (P), for their growth (cells buildup). Therefore to prevent microbial growth, the treated wastewater shall contain lesser amounts of such nutrients, i.e. the removal of such nutrients is an essential treatment process that is called the tertiary treatment.

For treated wastewater effluents, the standard guidelines require contents of less than 1mg P/L and less than 10mg N/L.

Wastewater characteristics Nitrification

The presence of nitrogen compounds like ammonia in the wastewater requires an extra oxygen demand besides the amount required to oxidize the organic materials.

$$NH_4^+ + 2O_2 \longrightarrow NO_3^- + 2H^+ + H_2O$$

The oxidation of the nitrogen compounds is called the nitrification. Nitrification requires 2 moles of O_2 per oxidizing 1 mole of N in the compound NH_4 , and produces 2 moles of H⁺ that <u>reduces the pH</u>.

2 moles of $O_2 = 2 \times (32) = 64g O_2$ and 1 mole N = 14g N, therefore nitrification requires $64g O_2/14g N$

$$= 4.6 \text{g O}_2/\text{g N}.$$

Calculating O₂ amount

For nitrification, 1g N requires 4.6g O₂.

In Jordan, the average COD is 1800mg/L and [N] is 150mg/L. Assuming wastewater generated at a rate of 80% of the average daily water consumption that is assumed to be 100L/day/person, then the 1 person in Jordan requires:

For COD: (1800mg/L)×(1g/1000mg)×80L/day = 144g/day

For nitrification: $4.6 \times (150 \text{ mg/L}) \times (1 \text{g}/1000 \text{ mg}) \times 80 = 55 \text{g/day}$

The per person O_2 amount needed assuming wastewater generated as 80% of the 100L/c/day water consumption is $\approx 200 \text{ g } O_2/\text{day}$

Wastewater characteristics The population equivalent (PE)

World wide, the unit that is used to quantify the amount of wastewater generated for treatment purposes (calculations like the O_2 amount needed) is the population equivalent (PE). The PE is a very useful tool to convert the wastewater of different characteristics generated by different industries to population units (usually of similar wastewater characteristics).

1 PE represents the daily wastewater production of 1 person expressed in the total amount of Oxygen consuming substances.

Wastewater characteristics The population equivalent (PE)

If an industrial activity discharges flow rate Q_i , then for treatment purposes that industrial wastewater can be converted to persons as follows:

 $PE = Q_i \times (COD_i + 4.6 \times [N]_i)/per person O_2 amount$

Q_i: the industrial wastewater flow (m³/day),
COD_i: COD of the industrial wastewater,
[N]_i: nitrogen concentration in the industrial wastewater, mg/L.

For Jordan, the per person O_2 amount ≈ 200 g/day.

Wastewater characteristics **Example**

A Jordanian city of 15000 inhabitants generates wastewater at a rate of 80L/day/person. If the city industrial wastewater flow is 85m³/day with COD of 3000mg/L and [N] of 72mg/L, what is the average total wastewater flow produced for treatment purposes.

Soln:

 $PE = 85 \times (3000 + 4.6 \times 72)/200 = 1416 \text{ person}$

Wastewater flow = $(15000 + 1416) \times 0.08 = 1313 \text{m}^3/\text{day}$
Wastewater Treatment Engineering

110401455

Wastewater treatment Instructor: Dr. Zeyad Tarawneh

Wastewater treatment Treatment definition

Wastewater treatment refers to the process of removing pollutants from used water after being contaminated with pollutants of concentrations that exceed the limits of standards for certain use. For example, wastewater of high BOD and TSS loads should be treated before discharging that wastewater to any surface water source that requires BOD load below 25mg/L and TSS load below 35mg/L.

Wastewater treatment can be achieved through consecutive removal stages called primary, secondary, tertiary, and quartiary (advanced) treatment.







Wastewater treatment Wastewater treatment stages

Stage 1:

The primary treatment targets the removal of coarse, suspended and floating solids, and to some degree nutrients. It consists of screens, grit chamber, and primary sedimentation tanks.

Stage 2:

The secondary treatment targets the removal of dissolved organic materials and suspended solids to large degree through biodegradation. Phosphorus removal reaches high levels. It consists of aeration tanks and final sedimentation tanks.

Wastewater treatment Wastewater treatment stages

Stage 3:

The tertiary treatment targets the elimination of pollutants not removed by primary and secondary treatment processes. In such stage high percentages of nutrients (N and P) is removed. It consists of carbon adsorption filters and disinfection units.

Stage 4:

The quartiary treatment targets high quality treated water that can be used for portable and non-portable water usages. It consists of membrane filtration and RO (Reverse Osmosis) units.

Flow equalization Equalization tank

Wastewater flow varies from time to time (non-peak flows at some hours while peak flows at others). For better managing and controlling the treatment processes, the wastewater inflow should inter the treatment plant at a constant rate. The purpose of the equalization tank is to maintain a constant inflow rate to the treatment plant.



Flow equalization **Design of the equalization tank**

The mass diagram is used to find the proper volume of the equalization tank required to ensure constant wastewater inflow rate:

- Obtain cumulative variable inflows and cumulative average (constant) inflows over a period of 24 hrs.
- Plot cumulative variable inflows and cumulative average inflows versus time.

• Construct tangents of the cumulative constant inflows at the curve of the cumulative variable inflows where periods of low and high inflows occur.

• Estimate the equalization tank volume.

Flow equalization **Design example**

Design an equalization tank to provide the treatment plant with an average inflow for a community discharging daily wastewater flow as follows:

Hr	Domestic flow (m ³ /day)	Hr	Domestic flow (m3/day)
1am	1200	1pm	2400
2	1200	2	3360
3	1440	3	3840
4	1680	4	3600
5	2160	5	3240
6	3000	6	3000
7	4080	7	2400
8	3600	8	2280
9	3120	9	2160
10	2640	10	1920
11	2040	11	1680
12	2160	12	1440



Hr	Cumulative domestic flow (m ³ /d)	Cumulative plant inflow (m ³ /d)	Hr	Cumulative domestic flow (m ³ /d)	Cumulative plant inflow (m ³ /d)
1am	1200	2485	1pm	30720	32305
2	2400	4970	2	34080	34790
3	3840	7455	3	37920	37275
4	5520	9940	4	41520	39760
5	7680	12425	5	44760	42245
6	10680	14910	6	47760	44730
7	14760	17395	7	50160	47215
8	18360	19880	8	52440	49700
9	21480	22365	9	54600	52185
10	24120	24850	10	56520	54670
11	26160	27335	11	58200	57155
12	28320	29820	12	59640	59640



Wastewater Treatment Engineering

110401455

Primary treatment Instructor: Dr. Zeyad Tarawneh

Primary treatment **Purpose**

The purpose of the primary treatment is to remove the coarse, suspended solids, and to some degree nutrients. The primary treatment consists of the following steps: screening of coarse solids (large size solids), removing of sand and grit, and finally sedimentation of suspended solids. Usually the primary treatment is called the mechanical treatment.

The following schematic plot shows the primary treatment steps.



Primary treatment Screens

Screens are used to remove large objects like paper, plastic, and wood pieces. Such solids, if not removed will cause damage to pumps and sludge removal equipments, hang over weirs, block valves, nozzles, channels, and pipelines. Screens are made of bars (usually 10 mm) with clear distance (CD) between bars. The bars are laid at 30 - 60° from the horizontal. The screening process starts usually with coarse screening (screens with CD = 20 - 50 mm) followed by fine screening (screens with CD < 20 mm).

The cleaning of screens can be performed either manually for small plants or mechanically for large treatment plants.



Primary treatment

Design of screens

The design of screens depends on the flow approach velocity (usually 0.5 - 0.8m/s), the slope of screen bars, and the fraction of area between bars that is clogged. The design is optimum when the channel approach area (*A*) is equal to the effective area between bars. Usually the flow depth (D) is taken as 0.5 of the flow width (W).

The effective area between bars =

$$= \frac{A}{\sin\alpha} f_a (1 - f_c)$$



The channel approach area = Q/V

Primary treatment Design of screens

Equalizing both areas, then:

$$\frac{A}{\sin\alpha}f_a(1-f_c) = \frac{Q}{V}$$

$$A = \frac{Q}{V} \frac{\sin \alpha}{f_a (1 - f_c)}$$

V: average approach velocity (m/s) α : bars slope angle. *A* = approach channel area f_a : area factor = T/(T + CD), T: bar thickness f_c : fraction of area between bars that is clogged. When no clogging exists $f_c = 0$, complete clogging $f_c = 1$.

Primary treatment Design example

Design a fine screening unit to accommodate peak flow of $40000m^3/d$. Allow 10% clogging and use average flow velocity.

Soln:

Using average flow velocity = (0.5 + 0.8)/2 = 0.65m/s $f_c = 0.1, Q = 40000/(24 \times 3600) = 0.46$ m³/s. Using bars of T = 10mm and α as 30°, then: A = Q/V = 0.46/0.65 = 0.71m² = W×D = W×(0.5W) Therefore W = 1.2m Take channel width as 1.2m or 1200mm.



Primary treatment Grit chamber

A chamber that is used to remove particles of diameter 0.01 - 0.2mm like sand, dust, seeds, eggshells, or any other material that passes the screens but heavier than the suspended organic matter. The removal of grit and sand protect pumps, the secondary treatment units, and

pipes from damage.



Primary treatment **Design of grit chambers**

Grit chambers are usually designed considering the flow velocity and the settling velocity of grit particles. Rectangular settling chambers may have flow depth to channel width ratio (D/W) of 1.5

Particle diameter (mm)	Settling velocity (m/hr)
0.01	0.3
0.05	6.1
0.1	24
0.2	82

Primary treatment **Design example**

Design rectangular grit chambers to remove particles of 0.2mm diameter in treatment plant of 10000m³/d wastewater flow. The flow velocity is 0.3m/s.

Soln: The chamber area $A = Q/V = W \times D = 1.5W^2$ $= [10000/(24 \times 3600)]/0.3 = 0.39m^2.$ The chamber area = $1.5W^2 = 0.39$, so W = 0.5mFlow depth D = 1.5W = 0.75mAt settling velocity of 82m/hr and flow depth of 0.75, the detention time t_d = flow depth / settling velocity = 0.75/(82/3600) = 33 sec

Primary treatment Design example: The detention time (t_d) is defined as the time at which grit particles stay in the chamber. With t_d of 33 sec, the chamber length can be found as: $L = t_d \times Flow \ velocity$ $= 33 \times 0.3$ = 10mThe chamber dimensions are: Width = 0.5m Height = D + 0.2D = 0.9m (use 20% of D as free board) Length = 10m

Primary treatment Sedimentation tanks

The purpose of the primary sedimentation is to remove the suspended organic solids, consequently reducing BOD loads. The sedimentation process depends on the flow surface loading rate (SLR), the hydraulic detention time (t_d), and the concentration of the TSS in the wastewater inflow (influents).





Primary treatment **Design example:**

Design a primary rectangular sedimentation tank for wastewater flow of 5000m³/d such that 50% of the TSS is removed?

Soln:

At 50% TSS removal, the SLR = 50 m³/ m²d, The tank surface area = $5000/50 = 100m^2 = L \times W = 5W^2$ $W^2 = 100 / 5$, so W = 4.47m, take W = 4.5m. The tank length L = 5W = 22.5m. The tank volume = $4.5 \times 22.5 \times 3.5 = 355m^3$

Primary treatment **Design example:**

The detention time $t_d =$

= 355/5000 = 0.0709d

= 1.7 hrs

The designed detention time = 1.7 hrs > the typical detention time = 1.1 hrs (from the design chart), the design is OK.

Use rectangular sedimentation tank of: Width = 4.5m Length = 22.5m Depth = 3.5m

Primary treatment

Tank outflow: V-notch weirs Outlet weirs are placed to maintain outlet flow velocity suitable for settling in the basin and to minimize short-circuiting.





Wastewater Treatment Engineering

110401455

Secondary treatment Instructor: Dr. Zeyad Tarawneh

Secondary treatment **Purpose**

The effluent from the primary treatment still contains 40 - 50% of the original suspended solids and high content of the dissolved organic solids as BOD (70 - 80%) and inorganic solids like N (85%) and P (85%). To meet standards for discharging treated wastewater, the dissolved organic matter (BOD) must be reduced.

The removal of the suspended and dissolved organic matter is called the secondary treatment. Such treatment stage may consists of chemical-physical processes (coagulation, filtration, carbon adsorption) or biological processes. Compared to the primary treatment, the secondary treatment costs much more.

Secondary treatment Biological treatment

The biological treatment philosophy depends on bacteria that use organics in the wastewater as food source for energy production and biomass buildup. Such process may occur in nature without control on the biomass production, compared to a well designed treatment units, called reactors, where the control on the biomass production is achieved.

The work mechanism of the biological treatment will be detailed for the purpose of designing treatment reactors. The most common biological treatment reactor in use is called the activated sludge system.







Secondary treatment Biological treatment: biomass growth

$$k = \frac{k_o S}{K_s + S}$$

When there is excess of food, $S >> K_s$, then the growth rate constant *k* is approximately equal to the maximum growth rate k_o , so the growth is biomass limited,

$$\frac{dX}{dt} = r_x = k_o X$$

When there is shortage in the limiting food, $S \ll K_s$, then the growth is food limited, *k* is very small,

 $r_x = \text{constant}$

Secondary treatment Biological treatment: biomass growth

The general growth equation becomes:

$$r_x = \frac{dX}{dt} = \frac{k_o S}{K_s + S} X$$

The complete growth equation should account for the biomass endogenous decay (with time part of the biomass is newly produced, other part decays). Given the endogenous decay rate k_d , then

$$r_x = \frac{dX}{dt} = \frac{k_o S}{K_s + S} X - k_d X$$

Secondary treatment Biological treatment: biomass growth

Defining $r_s = dS/dt$ as the rate of food utilization, then if all of the food is converted to biomass, then the rate of food utilization equals the rate of biomass production (dS/dt = dX/dt), however in reality part of the food will be converted to biomass while the rest is waste material due to energy production. Therefore the rate of food utilization is always greater than the rate of biomass production.

If *Y* is the % of food converted to biomass, then:

$$r_{x} = -Y \frac{dS}{dt} = -Yr_{s}$$
$$r_{s} = -\frac{r_{x}}{Y} = -\frac{k_{o}S X}{Y(K_{s} + S)}$$

Secondary treatment Biological treatment: activated sludge

The activated sludge process is a suspended-culture system that has been in use since the early 1900s. The suspended-culture system means that the biomass flocks (gathers) in the reactor and stay in suspension (mixed with the wastewater). The process takes its name from the fact that part of the settled sludge containing active bacteria is returned to the reactor for the purpose of increasing the available biomass and hence speeding up the reaction (the conversion of the organic material). The process may be either completely mixed or plug-flow process with Oxygen being supplied through injecting air. Therefore the process is completely aerobic.







Secondary treatment Activated sludge: completely mixed reactor

Solving the mass balance equations considering the assumption 1 to 3, then:

$$\frac{Q_w X_u}{V X} = \frac{Q_o}{V} \frac{Y}{X} (S_o - S) - k_d$$

Defining the hydraulic detention time $\theta = V/Q_o$, and defining the mean cell residence time $\theta_c = VX/Q_wX_u$ as the average time in which the sludge (bacteria) stays in the reactor, then:

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

Secondary treatment Activated sludge: completely mixed reactor

The concentration of the biomass or the Mixed-Liquor Suspended Solids (MLSS) is related to the hydraulic detention time θ and the mean cell residence time θ_c as follows:

$$X = \frac{\Theta_c Y(S_o - S)}{\Theta(1 + k_d \Theta_c)}$$

The equation above states that as θ decreases then X increases, that is true up to certain limit. When θ approaches the bacteria generation time, then there is no time for biomass to feed and the cells will be washed out from the reactor before the growth occurs, consequently X decreases and S approaches S_o meaning that no treatment is occurring.

Secondary treatment Example:

A completely mixed activated sludge reactor of 400m³ volume is used to treat wastewater flow of 2500m³/d from primary treatment with BOD of 200mg/L. The cell mean residence time to be kept at 10d with endogenous decay rate of 0.045/d and underflow sludge concentration of 10000mg/L. The MLSS concentration is 4000mg/L with food amount converted to biomass as 0.5, calculate the effluent flow BOD and the sludge recirculation ratio?

$$X = \frac{\Theta_c Y(S_o - S)}{\Theta(1 + k_d \Theta_c)}$$

Secondary treatment **Example:**

The mixed-liquor suspended solids $X = 4000 \text{ mg/L} = 4 \text{ kg/m}^3$ The concentration of the food $S_o = \text{influent BOD} = 0.2 \text{ kg/m}^3$ $\theta = V/Q_o = 400/2500 = 0.16d$ Y = 0.5, and $\theta_c = 10d$ $X = \frac{\theta_c Y(S_o - S)}{\theta(1 + k_d \theta_c)}$ $4 = \frac{10 \times 0.5 \times (0.2 - S)}{0.16 \times (1 + 0.045 \times 10)}$ $S = 0.0144 \text{ kg/m}^3 = \text{BOD}_{\text{eff}} = 14.4 \text{ mg/L}$

Secondary treatment **Example:**

The sludge recycle ratio is defined as $\alpha = Q_i/Q_o$. Conducting mass balance over the secondary clarifier then:

$$(Q_o + Q_r)X = (Q_o - Q_w)X_e + Q_uX_u$$

Noting that X_e is negligible and that $Q_u = Q_r + Q_w$

$$Q_o X + Q_r X = Q_r X_u + Q_w X_u$$

$$Q_r = \frac{Q_w X_u - Q_o X}{X - X_u}$$

From the mean residence time

$$Q_w X_u = \frac{VX}{\theta_c} = \frac{400 \times 4}{10} = 160 \text{ kg/d}$$

Secondary treatment **Example:**

Given the secondary clarifier underflow sludge concentration (X_u) of 10000mg/L, then the recycle flow

$$Q_r = \frac{160 - 2500 \times 4}{4 - 10} = 1640 \,\mathrm{m^3/d}$$

Therefore the recirculation ratio α is

$$\alpha = \frac{Q_r}{Q_o} = \frac{1640}{2500} = 0.656$$



Secondary treatment Activated sludge: plug-flow reactor

Applying biomass and food rates balance over the reactor, then the average MLSS is given as:

$$\overline{X} = \frac{\Theta_c Y(S_o - S)}{\Theta(1 + k_d \Theta_c)}$$

The above equation applies when $\theta_c/\theta \ge 5$. and

$$\frac{1}{\theta_c} = \frac{k_o(S_o - S)}{(S_o - S) + (1 - \alpha)K_s \ln\left[\frac{S_o + \alpha S}{S + \alpha S}\right]} - k_d$$

Secondary treatment **Example:**

A plug-flow activated sludge reactor of 400m³ volume is used to treat wastewater flow of 2000m³/d from primary treatment with BOD of 170mg/L. The cell mean residence time to be kept at 8d with endogenous decay rate of 0.06/d. The average mixed-liquor suspended solids concentration is 2000mg/L with food amount converted to biomass as 0.5, calculate the effluent flow BOD?

$$\overline{X} = \frac{\Theta_c Y(S_o - S)}{\Theta(1 + k_d \Theta_c)}$$

Secondary treatment **Example:**

The average mixed-liquor suspended solids $\overline{X} = 2000 \text{ mg/L} = 2 \text{ kg/m}^3$. The concentration of the food $S_o = \text{ influent BOD} = 0.17 \text{ kg/m}^3$. $\theta = V/Q_o = 400/2000 = 0.2 \text{ d}$ Y = 0.5, and $\theta_c = 8 \text{ d} \quad (\theta_c / \theta \ge 5)$ $\overline{X} = \frac{\theta_c Y(S_o - S)}{\theta(1 + k_d \theta_c)}$ $2 = \frac{8 \times 0.5 \times (0.17 - S)}{0.2 \times (1 + 0.06 \times 8)}$

 $S = 0.022 \text{kg/m}^3 = \text{BOD}_{\text{eff}} = 22 \text{mg/L}$

Secondary treatment Completely mixed reactor vs. plug-flow

The completely mixed activated sludge reactor is superior to plug-flow reactor when wastewater flow loading rate is widely fluctuating. In the CM reactors instantaneous dilution to wastewater loads occurs due to mixing and that would reduce the effect of shock loads compared to the PF reactors where no mixing occurs.

When the flow loading rate is reasonably constant, PF systems produce more mature sludge with excellent settling characteristics. PF reactors is less energy demanding when compared to CM reactors.

Secondary treatment **Design criteria**:

The most important design variables for activated sludge reactors are:

-Volumetric loading rate (V_L) : mass of influent BOD divided by the reactor volume.

$$V_L = \frac{Q_o S_o}{V}$$

- Food to Mass ratio (F/M): mass of BOD removed divided by the biomass in the reactor.

$$F / M = \frac{Q_o(S_o - S)}{V X}$$

- Mean cell residence time (θ_c)

_	-		_		
Process type	Θ_{c}	F/N	1	V _L	
	(day)	(kg BOD/ł	kg MLSS)	(kg BOD/m ³ .	.d)
Conventional PF	4 – 15	0.2 –	0.4	0.3 – 0.6	
Completely mixe	d 4–15	0.2 –	0.4	0.8 - 2	
Operation	al paran	neters:			
Operation Process type	al param	neters: α	BOD	Air supplied	θ
Operation Process type	al param MLSS (mg/L)	eters: α (ratio)	BOD removal %	Air supplied (m ³ /kg BOD)	θ (hr)
Operation Process type Conventional PF	al param MLSS (mg/L) 1500 – 3000	neters: α (ratio) 0.25 – 0.5	BOD removal % 85 – 95	Air supplied (m ³ /kg BOD) 45 – 90	θ (hr) 4 - 8

Secondary treatment **Design example:**

Design a CM activated sludge aeration tank to treat wastewater flow of 1500 m³/d. The influent BOD is 220mg/L while the effluent BOD must meet standards of 10mg/L. The pilot plant analysis shows that Y = 0.5 and $k_d = 0.06/d$. For operational purposes the underflow sludge concentration is 10000mg/L Soln:

The F/M ratio is selected as design parameter, while MLSS is selected as operation parameter.

Select F/M = 0.3, and MLSS = 4000 mg/L.

$$F / M = \frac{Q_o(S_o - S)}{V X} = 0.3 = \frac{1500 \times (0.22 - 0.01)}{V \times 4}$$
$$V = 262.5 \text{m}^3$$

Secondary treatment **Design example:**

Check on the volumetric loading rate V_L

$$V_L = \frac{1500 \times 0.22}{262.5} = 1.257 \text{ kg BOD/m}^3.\text{d} \text{ (acceptable)}$$

 $\theta = 262.5/1500 = 0.175d = 4.2$ hrs (acceptable) Check on the mean cell residence time θ_c

$$\frac{1}{\theta_c} = \frac{Y(S_o - S)}{\theta X} - k_d = \frac{0.5 \times (0.22 - 0.01)}{0.175 \times 4} - 0.06$$

$$\theta_{\rm c} = 10 \, {\rm day} \, ({\rm ok})$$

Secondary treatment **Design example:**

Conducting mass balance over the secondary clarifier then:

$$(Q_o + Q_r)X = (Q_o - Q_w)X_e + Q_uX_u$$

Noting that X_e is negligible and that $Q_u = Q_r + Q_w$

$$Q_o X + Q_r X = Q_r X_u + Q_w X_u$$
$$Q_r = \frac{Q_w X_u - Q_o X}{X - X_u}$$

From the mean residence time

$$Q_w X_u = \frac{VX}{\theta_c} = \frac{262.5 \times 4}{10} = 105 \text{ kg/d}$$

Secondary treatment **Design example:**

Given the secondary clarifier underflow sludge concentration of 10000mg/L, then the recycled flow

$$Q_r = \frac{105 - 1500 \times 4}{4 - 10} = 736.9 \text{ m}^3/\text{d}$$

Therefore the recirculation ratio α is

$$\alpha = \frac{Q_r}{Q_o} = \frac{736.9}{1500} = 0.5$$
 (acceptable)

Secondary treatment **Design example:**

Select dimensions of the aeration tank as follows: -No. of units 2 or more. -Depth: 3 – 10m. -Length – Width: 5 – 1.

Using 2 tanks, each has 131.25 m^3 capacity. Each tank has depth of 3m, Tank area = $131.25/3 = 43.75\text{m}^2$ $43.75 = \text{L} \times \text{W} = 5\text{W}^2$, therefore W = 2.95m Use 2 tanks each of 3m depth, 3m width, and 15 length.

Secondary treatment Oxygen required

The amount of Oxygen required to aerate the activated sludge is given as:

Req O₂ = $1.47Q_o(S_o - S) - 1.15(VX/\theta_c)$

The air flow is $Q_{air} = \text{Req O}_2/0.278$

For the past design example, Req $O_2 = 342.3 \text{ kg/d}$

 $Q_{\rm air} = 342.3/0.278 = 1231.3 {\rm m}^3/{\rm d}$



Secondary treatment Secondary clarifier

Secondary clarifiers in secondary treatment must accomplish two objectives:

1- to produce effluent flow of sufficient quality to meet standards,

2- to concentrate the sludge for the returned sludge and to handle the waste sludge.

Secondary clarifiers are usually designed as circular or rectangular. The circular settling tanks are preferable due to the low operation and maintenance costs, however circular tanks might become more sensitive to short circuiting due to wind.









Secondary treatment Secondary clarifier: Design criteria

The design of the secondary clarifier depends on the solids surface loading rate (SLR), the concentration of MLSS, and the flow rate. The surface area of the clarifier is given as:

$$A_{basin} = \frac{MLSS \times (Q + Q_r)}{SLR}$$

Typical SLR value is 2.5 kg/m².hr at an average flow.

Secondary treatment **Design example:**

Design a circular secondary clarifier to accommodate flow from the aeration designed previously.

Soln:

SLR = 2.5kg/m².h, Q = 1500m³/d, $Q_r = 736.9$ m³/d, MLSS = 4000mg/L.

$$A_{basin} = \frac{MLSS \times (Q + Q_r)}{SLR} = \frac{4 \times (1500 + 736.9)}{2.5 \times 24} = 150m^2$$

Tank radius = 7m
Secondary treatment Operational problem: rising sludge

In secondary settling tanks, sludge rises after short settling period. Anaerobic condition occurs at tank bottom leading to denitrification and later sludge rises due to the uplift force of N_2 gas.

 NO_3^- +biomass $\xrightarrow{anaerobic}$ N_2^+ +energy





Wastewater Treatment Engineering

110401455

Waste stabilization ponds Instructor: Dr. Zeyad Tarawneh

Waste stabilization ponds **Purpose**

The purpose of WSP is to treat municipal wastewater at relatively low cost compared to other biological treatment systems like the activated sludge. The major processes occur in such ponds are:

- Buffering of fluctuating hydraulic and organic loads,
- Primary settling of settleable solids,
- Algal photosynthesis to generate Oxygen,
- Biodegradation of organics (anaerobic and aerobic),
- Pathogens destruction (Ultraviolet from Sunlight).

The largest WSP in the world is El-Samra treatment plant in Jordan.

Waste stabilization ponds Advantages and disadvantages

Advantages:

- simple to construct, operate, and maintain,
- low investment and operation cost,
- absorb shock hydraulic and organic loads,
- sludge produced easily handed and used.

Disadvantages:

- require large areas,
- high algae content in the effluent,
- high evaporation losses,
- potential odor and mosquito nuisance.



Waste stabilization ponds Anaerobic ponds

Are unmixed basins designed to enhance the settling and biodegradation of particulate organic solids by anaerobic digestion. The main purpose is to reduce the strength of the raw sewage.

During summer conditions, such ponds reduce the BOD and enhance solids settling, while during winter conditions anaerobic ponds work as settling basins.

Usually the anaerobic ponds are designed and placed in serial arrangement ahead of the facultative ponds, i.e. anaerobic ponds followed by facultative ponds.

Waste stabilization ponds **Design of anaerobic ponds**

Anaerobic ponds are usually designed to have depth of 3 - 5m. The hydraulic retention time in such ponds is commonly taken as 1 - 3 days. The pond surface area is given as:

$$A_{surface} = \frac{Q_o S_o}{V_L D}$$

Where

 Q_o : flow rate, m³/d, S_o : raw sewage BOD kg/m³, V_L : volumetric loading rate, kg BOD/m³.d,

D: pond depth

Desludging of the anaerobic pond is required when the pond is $\frac{1}{2}$ full with sludge.

Waste stabilization ponds **Design of anaerobic ponds**

 V_L and BOD removal % versus the temperature.

Temperature (° C)	Volumetric loading (g/m³.day)	BOD removal (%)
< 10	100	40
10 - 20	20T - 100	2T+20
20-25	10T+100	2T+20
>25	350	70

To reduce odor releases from anaerobic ponds due to the formation of H_2S , lime is added to rise the pH level to around 8. Also Sodium nitrate is added to enhance denitrification (when denitrification occurs, the pH increases).



Waste stabilization ponds **Design example:**

at critical conditions (1/2 full sludge) the pond effective depth is 2m. At 20°C, $V_L = 10 \text{ T} + 100 = 300 \text{ g/m}^3.\text{d} = 0.3 \text{kg/m}^3.\text{d}$ For each pond:

$$A_{surface} = \frac{Q_o S_o}{V_I D} = \frac{2500 \times 0.25}{0.3 \times 2} = 1042 \,\mathrm{m}^2.$$

Pond volume = $2 \times 1042 = 2084 \text{ m}^3$,

The hydraulic retention time $\theta_a = 2084/2500 = 0.83d < 1$ day. The minimum retention time is 1 day, therefore the pond volume is:

Volume = $2500 \times 1 = 2500 \text{m}^3$,

Waste stabilization ponds **Design example:**

Pond surface area = 2500/effective depth = $2500/2 = 1250m^2$, Use Length : Width (L:W) as 3 : 1, then: Pond area = L×W = $3W^2 = 1250m^2$, therefore W = 20.5mUse 2 parallel ponds each of 4m depth, 20.5m width, and 61.5m length.

The BOD removal % from anaerobic ponds = 2T + 20= $2 \times 20 + 20 = 60\%$

The initial BOD = 250 mg/L, 60% removed in the anaerobic pond, then the anaerobic pond effluent BOD = the facultative influent BOD = 40% of the initial BOD that is $0.4\times250 = 100 \text{mg/L}$.



Waste stabilization ponds Facultative ponds

Are ponds with aerobic conditions occurring in the top layer while anaerobic degradation occurring in the deeper layers containing sludge deposits. In the top layer, algae consumes CO_2 during the photosynthesis and produces O_2 by which the aerobic bacteria converts the organic acids (soluble BOD) and oxidizes compounds of C, N, S, P. The biomass produced in the top layer settled to form the sludge blanket.

In the deeper layer, Oxygen does not exist, therefore anaerobic bacteria reduces (converts) the organic material and produces reduced compounds of C, N, S, P.

Waste stabilization ponds **Design of facultative ponds**

Facultative ponds are usually designed as 1 - 2m deep. Commonly the design is based on the surface BOD loading rate λ . The minimum retention time (θ_m) is 4 days.

$$A_{surface} = \frac{Q_o S_i}{\lambda}$$

Where

 Q_o : flow rate, m³/d,

 S_i : facultative pond influent BOD kg/m³,

 λ : surface BOD loading rate, kg BOD/m².d,

Usually facultative ponds are designed as ponds in series.

Waste stabilization ponds **Design of facultative ponds**

The surface BOD loading rate λ is a function of the temperature *T* as follows:

 $\lambda = 0.035 (1.107 - 0.002T)^{T-25}$

Design example:

For the previous example, after the anaerobic ponds design facultative ponds.

Soln:

 $S_i = 100$ mg/L = anaerobic ponds effluent BOD, $Q_o = 2500$ m³/d.

 $\lambda = 0.035(1.107 - 0.002 \times 20)^{20-25} = 0.025 \text{ kg/m}^2.\text{d}$



Waste stabilization ponds **Maturation ponds**

Maturation ponds represent the disinfection treatment unit after the removal of TSS, BOD, and nutrients through anaerobic and aerobic processes that occurred in the anaerobic and facultative ponds.

In maturation ponds, ultraviolet spectrum in the regular sunlight beam do the disinfection job for the treated wastewater. The ultraviolet beams kill all bacteria and viruses if the pond is operated at minimum retention time of 3 days.

Waste stabilization ponds **Design of maturation ponds**

The # of microorganisms (bacteria or viruses) is measured indirectly through measuring the Faecal Coliforms. Standards require maximum # of FC in treated wastewater as 1000FC/100ml for wastewater re-use purposes.

$$N_e = \frac{N_i}{\left(1 + k_b \theta_f\right) \left(1 + k_b \theta_m\right)^n}$$

Where

 N_e : # of FC/100ml in the effluent of facultative ponds,

 N_i : # of FC in the influent/100ml, max $N_i = 1 \times 10^8$ FC/100ml,

 k_b : FC growth rate constant, d⁻¹. $k_b = 2.6 \times (1.19)^{\text{T-}20}$

 θ_{f}, θ_{m} : retention time of facultative and maturation ponds,

n: # of maturation ponds in series.

Waste stabilization ponds **Design of maturation ponds**

The retention time of the maturation ponds (θ_m) is minimum 3 days. Moreover it is required for operational purposes that the retention time of the maturation ponds (θ_m) not to exceed the retention time of the facultative ponds (θ_f) , why????

The depth of maturation ponds is usually 1.2m maximum. The surface area of maturation ponds is:

$$A_{surface} = \frac{Q_o \theta_m}{D}$$

Waste stabilization ponds **Design example**

Design maturation ponds using data from pervious example.

Soln:

Use n = 3 ponds each 1m deep. $\theta_f = 6$ days, select $\theta_m = 6$ days (the minimum is 3 days)

At T = 20°C, $k_b = 2.6 \times (1.19)^{\text{T-}20} = 2.6/\text{day}$.

$$N_e = \frac{N_i}{(1 + k_b \theta_f)(1 + k_b \theta_m)^n} = \frac{1 \times 10^8}{(1 + 2.6 \times 6)(1 + 2.6 \times 6)^3}$$

 $N_e = 1317 \text{ FC}/100 \text{ml} > 1000 \text{ FC}/100 \text{ml} \text{ (standards), not OK}$ Use n = 4 ponds, then: $N_e = 79 \text{ FC}/100 \text{ml} < 1000 \text{ FC}/100 \text{ml} \text{ (standards), OK}$

Waste stabilization ponds **Design example**

Ponds total surface area =

$$A_{surface} = \frac{Q_o \theta_m}{D} = \frac{2500 \times 6}{1} = 15000 \,\mathrm{m}^2.$$

Surface area for each pond = $15000/4 = 3750m^2$. Using L:W as 3:1, then W = 35.5m and L = 106.5m.



