SEWER

Sewage systems – Information

• Sewage is divided into four classifications: **Domestic** sewage comes from residences, institutions, and business buildings and is a priority due to its sanitation threat. **Industrial** waste is the liquid resulting from manufacturing or industrial processes - treatment of these wastes is usually collocated with the plant from which they originate. **Storm** sewage is the runoff during or immediately after storms. **Combined** sewage systems carry any combination of domestic, industrial, and/or storm sewage. Sewage systems consist of two major types of facilities: **collection** and **treatment**.

• The major components of collection are **pipes**, **lift stations**, and **manholes**. Pipes can vary in size, of course, and include collectors, trunk lines, and interceptors. Lift stations pump waste to a higher point for continued gravity flow or initiate forced flow when gravity flow isn't adequate. A lift station is comprised of a pump with vertical piping that is usually located in a small building. Manholes provide access into the Sewage system for inspection, preventive maintenance and repair; they are also the likely site of **junction boxes** and **cleanouts**, which marry systems and provide access to stoppages.

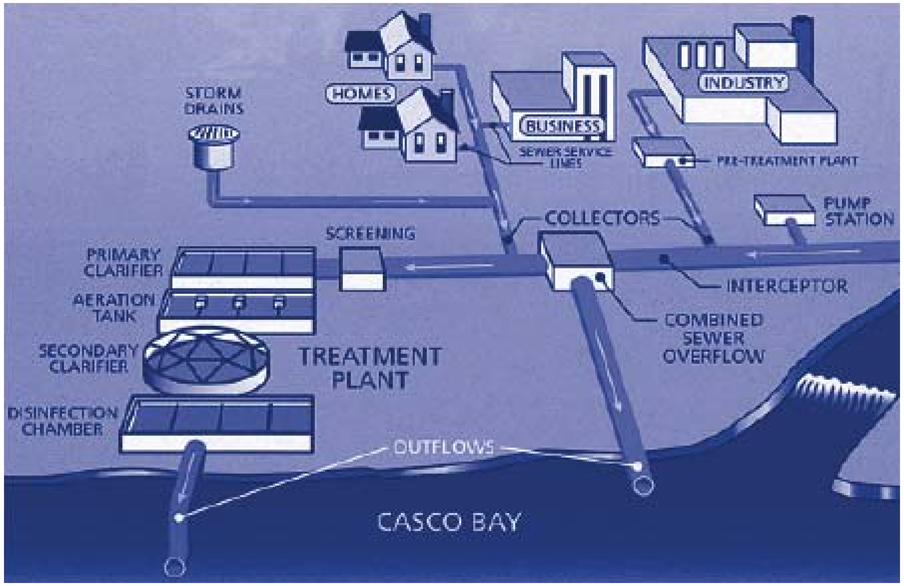
• There are three stages of treatment that may be present in a wastewater treatment plant: **primary**, **secondary** and **tertiary**. Wastewater may be completely untreated or treated up to the first, second, or all three levels.

The primary treatment method is the physical process (screening and sedimentation) that separates solids from liquids. **Racks** and **screens** are used to remove the largest solids and then **sedimentation tanks and clarifiers** are used to settle and remove smaller solids. The solids are sent to a **sludge digester** to stabilize the waste and subsequently dry in the **drying bed.** The solid waste should be disposed of off site. If the plant does no more than primary treatment, then the water is sent to a **final clarifier**, then **chlorinated (disinfected)** and finally **discharged**.

The secondary treatment method is additional to the primary. Typical layout is waste is combined with micro-organisms that break down the waste organic material in large **aerated tanks**. The wastewater then flows to settling tanks called **secondary clarifiers** where the bacteria settle out. A-typical layout has a **trickling filter** used instead, which contains granular media and bacteria that break down the organic material as it flows through the filter. Also, before discharge, a **sand filter** may be used.

The tertiary treatment method is where nutrients are removed from wastewater. Very little tertiary treatment is done.

Sewage system diagram



Sewage systems – Measurements

▪ It is obvious when a sewage collection system is failing, but a treatment plant might not show direct signs of malfunction. The following metrics are used to measure discharge in order of severity top to bottom:

Coliform bacteria count – this is the measure of fecal bacteria remaining in the water. Ideally this number would be zero. Note that the water in the environment is not totally free of fecal bacteria – wildlife do introduce some.

Chlorine – the chlorine used to kill harmful bacteria needs to be removed so it does not kill beneficial bacteria in the environment. Ideally, chlorine should not be detectable.

BOD (**B**iological **O**xygen **D**emand) – is a measure of how much oxygen in the water will be required to finish digesting the organic material. It should be zero, because organic material should not be discharged.

Dissolved oxygen – is the amount of oxygen in the water as it leaves the plant. If the water contains no oxygen, it will kill any aquatic life that comes into contact with it, therefore, dissolved oxygen should be as high as possible and needs to at least cover the BOD, if there is any.

pH – this is the measure if the water's acidity once it leaves the plant. The water's pH should match the pH of the receiving body of water.

Suspended solids – this is the measure of the solids remaining in the water after treatment. Ideally, suspended solids would be zero.

Phosphorus and **nitrogen** – is the measure of the nutrients remaining in the water. This should be close to zero.

Sewage systems-Collection systems

Lift station

Junction box and cleanout

Sewage systems - Emergency situations

- Stoppages caused by debris being blown and washed into sewers can be expected. Deliberate demolition by the enemy is usually limited to junction manholes or large mains. The enemy may destroy pumping stations deliberately because they are key points, are more accessible, and are most difficult to repair.

- Highly populated urban areas (such as apartment and house districts) depend almost entirely on waterborne sewage disposal systems, smaller communities can use temporary latrines.

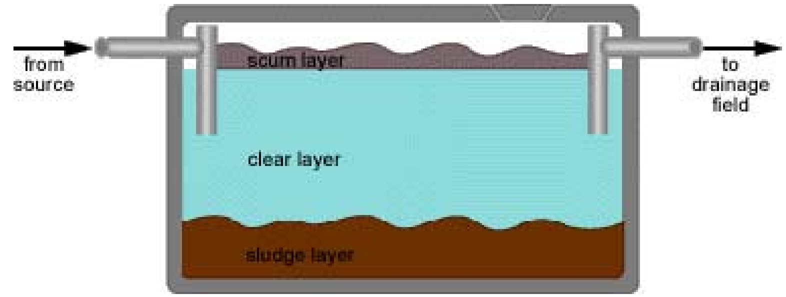
- Sewers are the most essential item in a sewage disposal system. Service can be restored temporarily by pumping from an upstream manhole, around the damaged section, and into a downstream manhole. If the sewer is completely stopped or badly damaged, an open channel can be built. Where storm and sanitary sewers are separate, it may be possible to divert sanitary sewage through a storm sewer to a suitable outlet.

- Portable, skid-mounted, centrifugal, gasoline engine pumps are the most suitable type for use in the hasty rehabilitation of Sewage systems. They must be of non-clog (open impeller) design and capable of handling unscreened sewage. Pumps with four inch intake and discharge are the most adaptable, since they can be used for draining craters, pumping around blocked sections of sewers, and temporarily replacing damaged pumping stations. Pipe less than four inches in diameter should never be used for sewage. Army system only has six inch diameter, closed impeller pumps that were originally designed for oil. Can work but not the good solution.

Septic systems – Information

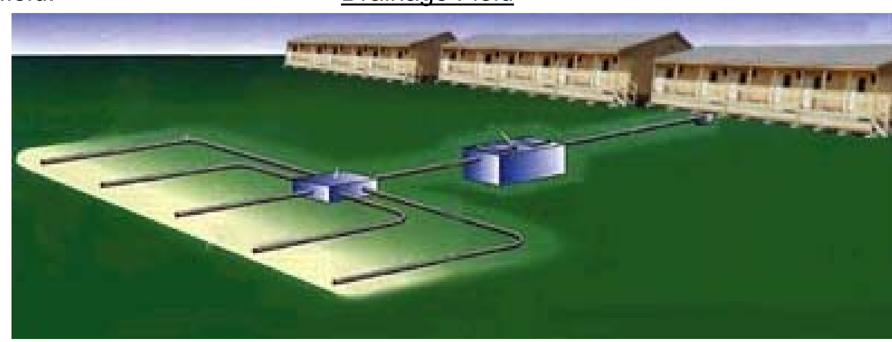
- The primary components of a septic system are its **tank** and its **drainage field**. A tank is simply a big concrete or steel tank that is buried. A drainage field is made of perforated pipes buried in trenches filled with gravel. Wastewater moves through the tank and then flows into the drain field, where it is slowly absorbed and filtered by the ground.

Septic tank



A septic tank is: wastewater flows into the tank at one end and leaves the tank on the other. There are three layers in a tank. Anything that floats rises to the top and forms a layer called a **scum layer**. Anything heavier than water sinks to the bottom and forms the **sludge layer**. In the middle is fairly **clear layer** of water, which contains bacteria and chemicals like nitrogen and phosphorus that act as fertilizers. As new water enters the tank, it displaces the water that is already there. Larger systems are more likely to have multiple tanks that separate solids and/or a sand filter between the tank and drainage field.

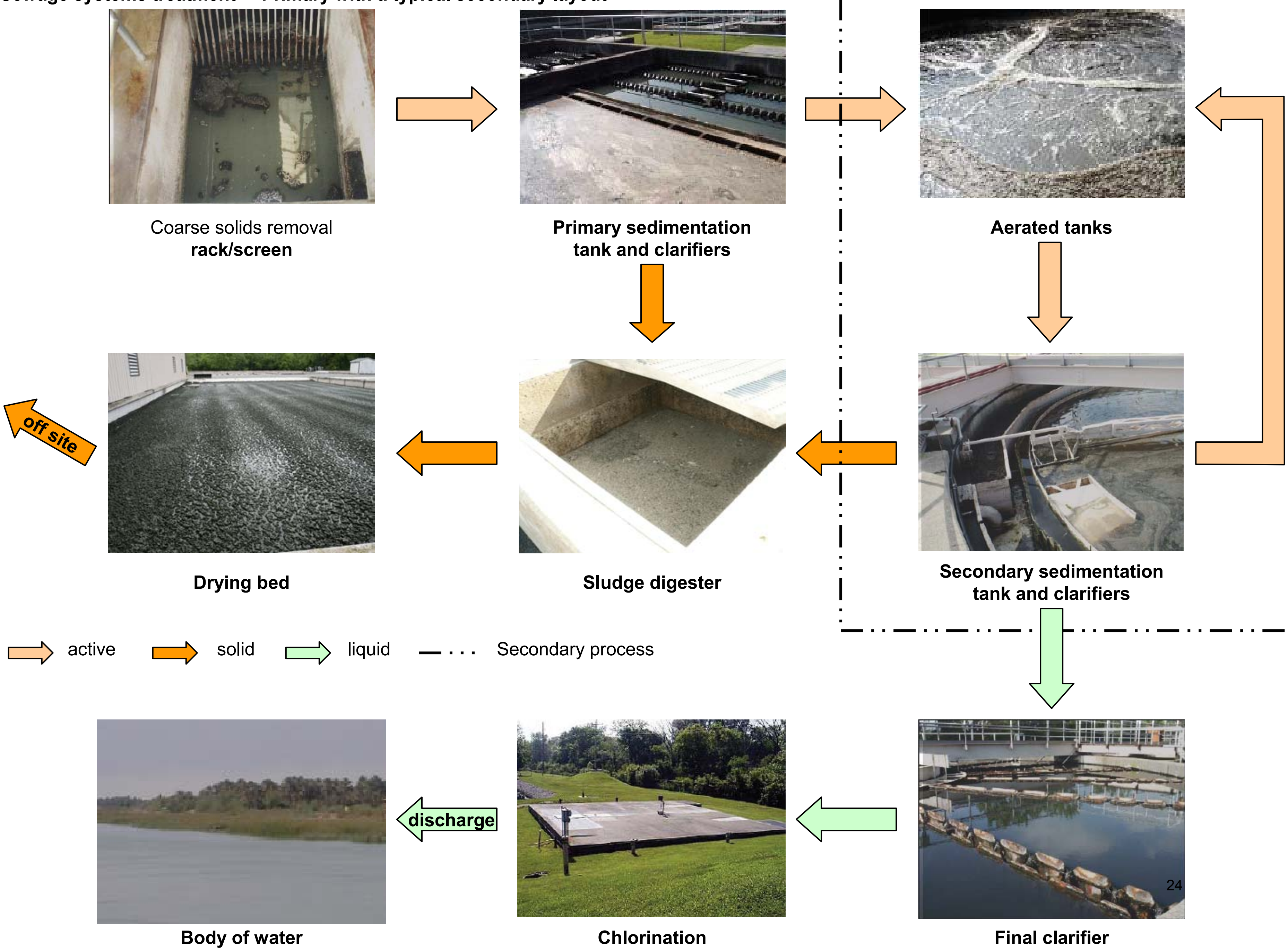
Drainage Field



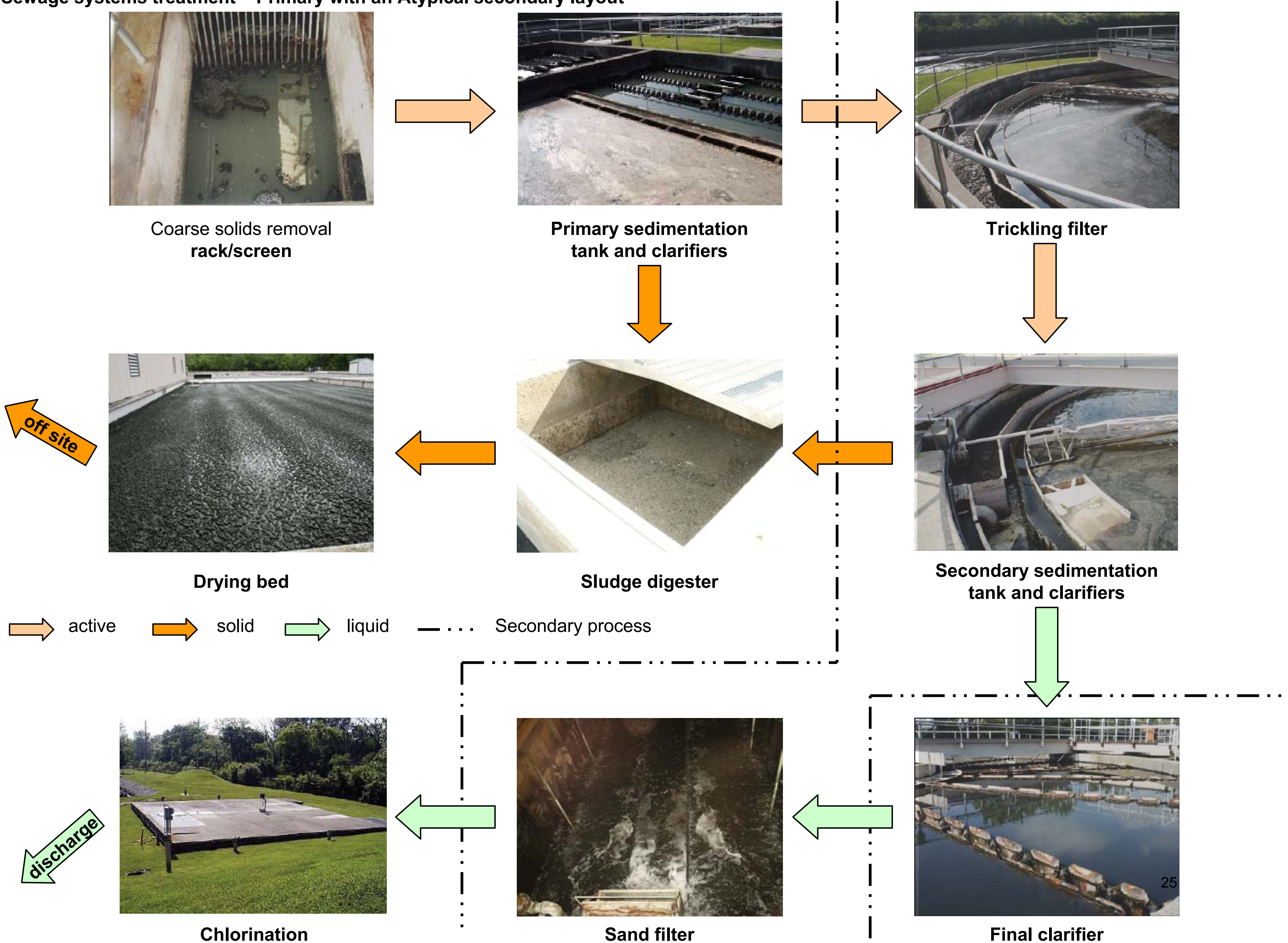
Septic systems - Measurement

- The primary metric for assessing septic tanks is their ability to accept additional wastewater. Scum must be periodically removed from the tank to avoid leaching into the drainage field and clogging the soil. Solids visible above the water level would also indicate impending problems. If possible, use a long pole to assess the thickness of the scum layer, liquid layer, and sediment.

Sewage systems treatment – Primary with a typical secondary layout



Sewage systems treatment – Primary with an Atypical secondary layout



Wastewater Treatment

Review Class Notes

Donald J. Burger

Wastewater Treatment

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Wastewater Treatment

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Wastewater flow rates

Wastewater flow rates are used to design collection system pipes, lift stations, and treatment facilities. Since wastewater is produced as water is used its collection occurs as it is used. The flow generated by water use then is related to water demand. However, not all water used is collected as wastewater; some water is used for irrigation, some is lost to evaporation as steam. Flow rates are influenced by a number of factors, but the most common is population. Thus, we often express the flow rates in terms of population: gallons per capita per day, for example.

We generally refer to treatment facilities by their capacity to treat average flow. For example, a 5 MGD wastewater treatment plant is designed to treat 5 million gallons per day on average throughout the year. Since flows are not constant, the plant must be designed to handle flows both higher and lower than the average. The higher flows often dictate the hydraulic design of a treatment plant, so the peak flow is used. Our 5 MGD plant may be designed for a peak flow of 15 or 20 MGD.

Municipal flows include flows from homes and businesses and are typically called domestic sewage or domestic wastewater. Municipal flows may contain an Industrial component as well. **Domestic Sewage** is wastewater from bathrooms, laundry and food preparation typical of dwellings and other places frequented by people and exclusive of industrial and manufacturing processes.

Industrial flows are wastewaters created by manufacturing processes and sometimes include a hazardous component. Industrial flows do not include domestic sewage from bathrooms and kitchen facilities present in the industrial facilities.

Commercial flows are typically from retail and office establishments and are generally composed of domestic sewage. Some exceptions exist in the case of certain establishments such as photolabs, dental labs, medical offices, and other businesses that contain some small-scale industrial process.

Average flows

The average is flow is determined by the dividing the total flow for a year by the number of days in the year (365). This yields the average daily flow:

$$Q_{Average} = \frac{Annual\ flow\ volume}{365\ days}$$

The average daily flow may then be divided by the population that produced the flow to determine the average daily flow per person:

$$Q_{Per\ Capita} = \frac{Q_{Average}}{Population}$$

Future flows can be determined by multiplying the average daily flow by the future population:

$$Q_{Design\ average} = Q_{Per\ Capita} \times Population_{Design\ Year}$$

Peak Flows

Peak flows are calculated by multiplying average flows by a peaking factor:

$$Q_{Peak} = Q_{Average} \times F_P$$

Research has shown that as population increases the ratio of peak flow to average flow decreases. Thus peaking factors are often calculated as a function of population. For example, Harmon's equation is a relationship that can be used to compute peaking factors:

$$F_P = 1 + \frac{14}{4 + \sqrt{P}}$$

Where:

F_p = peaking factor or ratio of peak flow to average flow
P = population in thousands

Harmon's equation is just one equation commonly used to determine peaking factors. Other equations and charts can be used to determine peak flows and peaking factors.

Infiltration and Inflow

Infiltration and inflow is the intrusion of storm water runoff and groundwater into sanitary sewer systems.

Infiltration: groundwater intrusion through pipe and manhole defects.

Inflow: surface water intrusion through openings in the collection system and direct connections to the system.

<u>Collection System Design flows</u>

Allowances for Inflow and Infiltration are added to the peak flow to determine the design flow for the collection system component.

$$Q_{Design} = Q_{Peak} + Allowance_{I/I}$$

The allowance can be expressed as a quantity per unit area served or as a quantity per unit upstream pipe size and length. Typical allowances include 500 gpd/acre served and 10 gpm/in·mile of upstream pipe.

$$Allowance_{I/I} = Area\ allowance \times Service\ Area$$

Or

$$Allowance_{I/I} = Pipe\ allowance \times \sum Pipe\ diameter \times Pipe\ length$$

Example Problem No. 1

A community needs a new wastewater treatment plant. The population is expected to grow to 35,000 people in the design year. The average wastewater flow in the community is about 100 gallons per person per day.

A. The design flow of the new plant should be:

   a) 3.5 MGD b) 7.0 MGD c) 350,000 GPD d) 850,000 GPD

B. If Harmon's equation is sufficient for design and we can neglect inflow and infiltration, then the plant must be designed for a peak flow most nearly:

   a) 3.5 MGD b) 7.0 MGD c) 8.5 MGD d) 10 MGD

C. The existing wastewater treatment plant currently serves a population of 22,000 people and has a service area of 3.4 square miles. Flow records from the existing wastewater treatment plant show an increase in the peak flow when it rains of about 642,000 gallons per day. Collection system components in this community should be designed for an increase in peak flow of about

a) 100 GPD/Acre b) 295 GPD/Acre c) 10 GPM/IN-mile d) 15 GPM/IN-mile

Collection Systems

Wastewater is collected and transported from homes, businesses and industry to wastewater treatment facilities by networks of pipes called a collection system.

Sanitary Sewer System: A collection system that collects only wastewater from homes and businesses and is designed to keep the wastewater flow separated from storm water flows. A sanitary sewer system is separate sewer system from the storm sewer system.

Combined Sewer System: A collection system that collects wastewater and storm water from homes and businesses. New systems of this type are not constructed because the expense of treating the combined flow at the treatment plant outweighs the cost savings associated with combining the systems. These are commonly found in older cities that were heavily developed prior to wastewater treatment systems.

Sewer Networks

Wastewater collection systems are composed of a network of gravity-flow pipes with access points called manholes. **Manholes** provide access to the sewer pipes for cleaning and maintenance. Collection systems may also include pumping stations (**lift stations**) and pressure piping (**force mains**). Wastewater flows from building drain systems to service lines and laterals that connect to collector sewers.

Collectors are gravity pipes that carry flows from buildings to Interceptor sewers and Trunk mains.

Interceptor sewers intercept flows from multiple collector sewers and transport the flow to trunk mains or to the treatment facilities.

Trunk mains transport flow to treatment facilities and are called trunk mains because they are analogous to the trunk of a tree with interceptors and branches and collectors and branches from the interceptors.

Lift stations are pumping stations designed to pump wastewater. They include special pumps that can pass solid material; usually a 2 ½-inch or 3-inch diameter sphere is used for design.

Force mains are pipes that operate under pressure produced by the lift station to convey the wastewater from the lift station to its discharge point. Force mains may be as small as 4-inch diameter and are sized to maintain minimum velocities greater than 2 ½ FT/s. Force mains often discharge into a manhole in a gravity sewer system or into the headworks of a wastewater treatment plant.

Collection System Design

Sewer pipes are sized to carry the design flow flowing full. The design flow may or may not be the peak flow plus an allowance for infiltration and inflow. Other considerations include minimum velocity to maintain solids in suspension and maximum flow velocity to prevent erosive damage to the sewer system facilities. Typical minimum velocity is 2.0 FT/s and typical maximum is 10 FT/s when flowing full.

Manning's equation is usually used to size pipes hydraulically. The pipe slope and flow is used to select the appropriate pipe size. The minimum size normally used for gravity sewers is 6-inch or 8 –inch pipe. Sizes smaller that 6 inches need steeper slopes to maintain velocity, clog easily with large debris and are more difficult to clean. Service pipes from individual buildings are usually 4-inch or 6-inch in diameter.

Lift stations

Lift Stations are sewage pumping stations used to lift sewage over hills in hilly terrain or to solve extreme sewer depth problems in flat terrain. These pump stations require special pumps, a structure to store wastewater between pump run cycles and controls to operate the pumps automatically. Redundant pumps are required to ensure reliability and many are equipped with back-up power supplies and other facilities to allow operation during power outages and other unusual operating conditions. Lift stations must operate automatically since they are needed 24 hours a day, 7 days a week.

Design Redundancy is necessary since lift stations operate unattended and must be reliable to prevent overflows of the collection system. This is accomplished by the installation of a redundant pump.

Installed Capacity is the total capacity of all installed pumps in a lift station operating together.

Firm Pumping Capacity is the reliable capacity of the lift station and is defined as the capacity of the installed pumps with the largest pump out of service.

Sewage pumps Types

Common types of pumps used in lift stations include: End Suction, Self-priming, and submersible.

End suction pumps are centrifugal pumps with the suction centered on the center of the pump impeller and the driver situated opposite the pump suction. End suction pumps must be operated with a flooded suction or include a means to maintain or restore the pump prime.

Vacuum Priming systems use vacuum pumps to draw water up a suction pipe to fill the pump and effect priming of the pump which allows the pump to be located above the level of the water in the lift station.

Self-priming pumps are a modification of the end suction design with an enlarged volute and suction pipe connection above the impeller. The suction connection typically includes a check gate to prevent reverse flow in the suction piping and loss of prime. The enlarged pump volute is designed to provide sufficient water storage to allow the pump to generate its own vacuum and thus re-prime itself in the event of prime loss.

Submersible pumps are similar to end suction pumps, but typically situated with the pump shaft vertical and the pump bolted directly to a specially manufactured motor that is sealed to operate completely submerged in water. Submersible pumps often require a minimum fluid height above the pump to effect sufficient motor cooling.

Lift Station Configurations

Many station configurations have been used for lift stations but most fall into one of three categories: Wet well/Dry well, wet well with pumps at surface, and submersible.

Wet Well: The structure that stores water between pump run cycles. Gravity collection system pipes connect to the wet well to deliver the flow to the lift station.

Wet well/dry well configurations are normally used with end suction pumps to provide the needed flooded suction. In this configuration sewage flows into the wet well which is separated from the dry well by a solid wall. Suction pipes for the pumps extend through the wall from the wet well to the pumps in the dry well. Pump discharge piping and valves are contained within the dry well. Electrical equipment and controls may also be contained within the dry well.

Wet well with pumps at the surface configurations have the pumps can be located above the water in the wet well, usually at ground level, but sometimes in a below ground structure over the wet well. The pumps are connected to the wet well by a suction pipe that extends vertically into the wet well through the top to a point just above the bottom of the wet well. Self-priming pumps or end suction pumps with a vacuum pump priming system are used in this configuration.

Submersible configurations use submersible pumps within the wet well. The pumps are mounted to a special discharge base bend which is anchored to the floor of the wet well. The bend supports the pump above the floor of the wet well and directs the pump discharge into a vertical discharge pipe. The pump is attached to the bend by gravity and can be removed by hoisting it up off the bend and along guide rails to the top of the wet well. This allows the pump to be serviced without the need for personnel entry into the wet well. The guide rails are attached to the bend and anchored to the wall of the wet well and to the top near an access hatch. The discharge piping can then penetrate the top of the wet well to valves or can pass through the side of the wet well to a below ground vault that contains the valves.

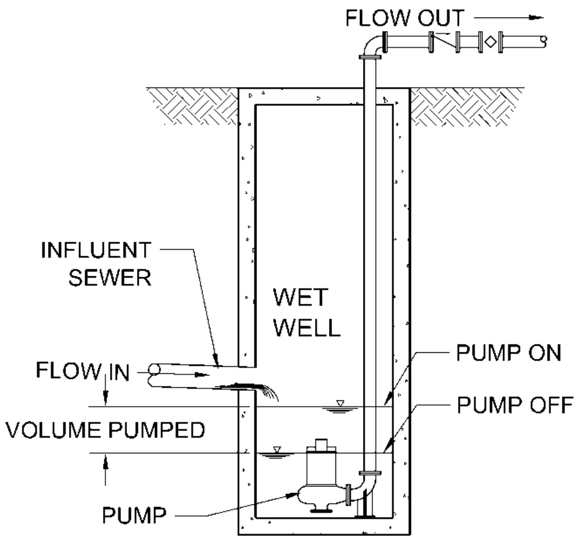

Figure 1 - Typical Submersible Lift Station

Wet Well design

The design of the wet well affects the operation of the pumps. Pumps are driven by electrical motors that are typically designed to run virtually continuously without failure. However, the stress on the electric motor is highest during pump starts and thus motors come with a maximum starting frequency rating. This is usually about six starts per hour. In other words, at six starts per hour the motor needs ten minutes between starts to stabilize winding temperatures to avoid burning up. Higher horsepower motors are typically rated for fewer starts per hour and some specially designed motors may be rated for as many as ten starts per hour. The time the motor need to cool between starts dictates the minimum cycle time for the lift station. The minimum cycle time is a function of the pump capacity and the wet well volume. Thus, wet well volume is selected to satisfy the minimum cycle time of the pump installed. The volume between the pump on and pump off elevation is determined by the following equation:

$$V = \frac{T_{Min} q_P}{4}$$

Where:

V = volume of the wet well between pump on and pump off, gallons.
T_{Min} = Minimum cycle time, minutes.
q_P = the pump's average flow rate, gallons per minute.

Lift Station Control systems

Control systems are used to operate the pumps and include a water level sensing system in the wet well with electronic controls that turn the pumps on when the level in the wet well is high and turn the pumps off when the level is restored to a minimum level in the wet well. Ultrasonic level sensors and submersible level transducers have been used successfully to transmit the water level in the wet well to a computerized controller that is program to operate the pumps based on the level in the wet well. Systems without computerized controllers use float switches that were set in the wet well at specific levels to sense high and low levels in the wet well. Other systems use a pressure transducer on an air pipe that bubbles air into the wet well at the bottom to sense the well level.

Example Problem No. 2

The sanitary sewer system shown in Figure 2 serves the neighborhoods of Dover Heights and East End Hills with the service area and population of each neighborhood shown. The average wastewater flow for each neighborhood is about 100 GPCD. Use Figure 3 for peaking factors.

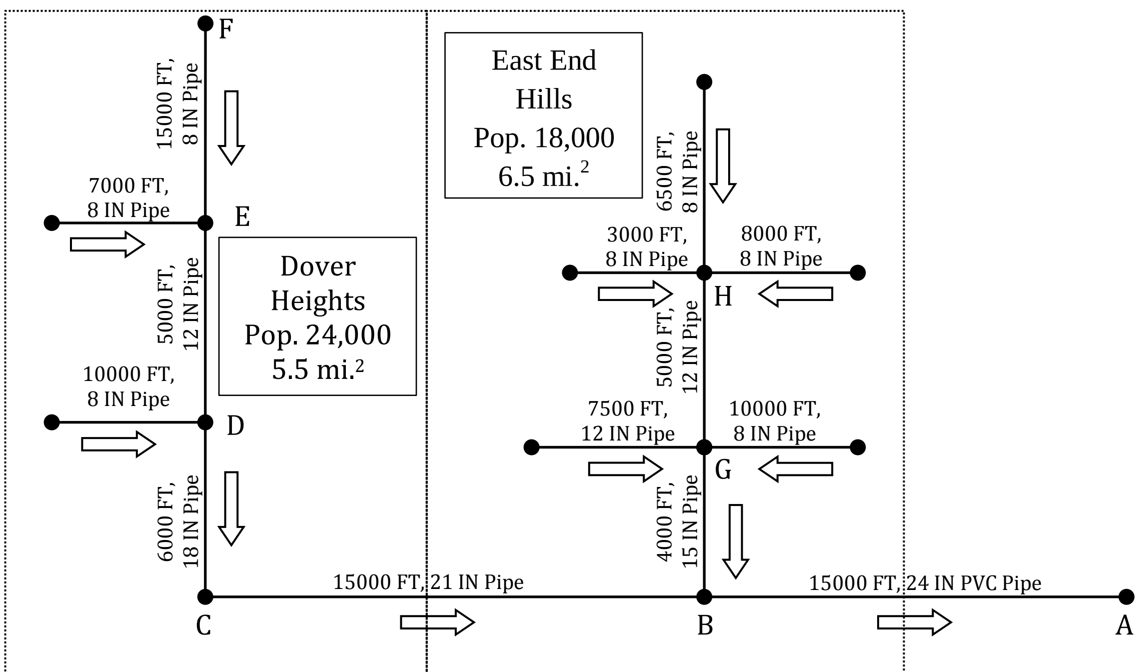

Figure 2 - Diagram for Example Problem No. 2

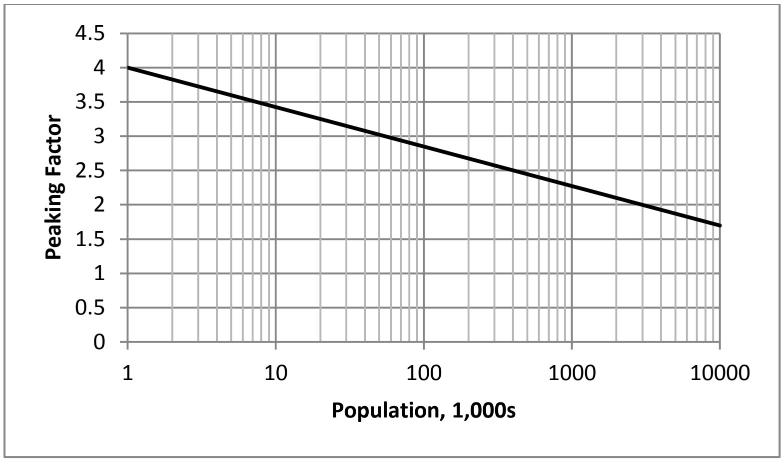

Figure 3 - Peaking for Example Problem No. 2

A. Pipe B-C in Figure 2 is in need of replacement. If inflow and infiltration is about 12 GPM/IN-mile. The design flow in FT^3/s for pipe B-C is most nearly:

a) 20.5 b)15.6 c) 11.9 d) 5.1

B. If the new pipe is 18 IN in diameter with a Manning's friction coefficient of 0.011, the minimum slope of pipe B-C in FT/100FT is most nearly:

a) 1.6 b) 0.52 c) 1.02 d) 0.011

C. A triplex lift station is needed to serve both neighborhoods shown in Figure 2. If inflow and infiltration is about 520 GPD/Acre, the capacity of each lift station pump in GPM must be most nearly:

a)7,127 b) 9,042 c) 5,910 d) 2,773

D. A new neighborhood is being planned near the north side of East End Hills. The development includes 350 single-family residential lots on about 88 acres. The developer expects an average of 2.5 persons to occupy each residence when complete. To connect to the existing sanitary sewer system, a lift station must be built to pump the wastewater from the new development to point H in Figure 2. The sewer influent pipe to the lift station has its flow line about 8.70 FT below ground. The lift station design is for two submersible pumps with a minimum submergence of 1.5 FT from the bottom of the wet well. A six foot diameter precast concrete wet well will be used. The motors on the pumps are rated for 10 starts per hour. Use 520 GPD/Acre for inflow and infiltration and 4.0 for a peaking factor. The depth in FT below ground to the bottom of the wet well will be most nearly:

a) 8.7 b) 12.2 c) 13.5 d) 10.7

Unit Operations and Processes

Wastewater treatment processes are divided into three main categories physical and chemical and biological.

Physical processes include:

- Screening.
- Grinding.
- Aeration.
- Settling.
- Drying.

Chemical processes include:

- The addition of settling aids such as Alum or iron salts.
- Disinfection with oxidants such as chlorine.
- Oxidation/reduction to remove nutrients or other contaminants.

Biological processes include:

- Activated sludge.
- Trickling filters.
- Extended aeration.

- Step aeration.
- Rotating biological contactors.
- Biological nutrient removal.
- Enhanced biological nutrient removal.
- Aerated lagoons.

Biological processes are processes that rely on a culture of bacteria and other small organisms to oxidize or reduce contaminants in the wastewater by converting soluble contaminants into biomass that forms a sludge that can be disposed of separately from the water.

Typical Unit Process

A typical unit process has three main flows, influent, effluent and waste as shown in Figure 4. The unit process is intended to remove at least one contaminant from the influent. Some processes achieve removal of multiple contaminants.

Influent is the flow into the unit process to be treated.

Effluent is the product flow out of the unit process that has been treated.

Waste is the flow from the process that contains the contaminant removed from the influent.

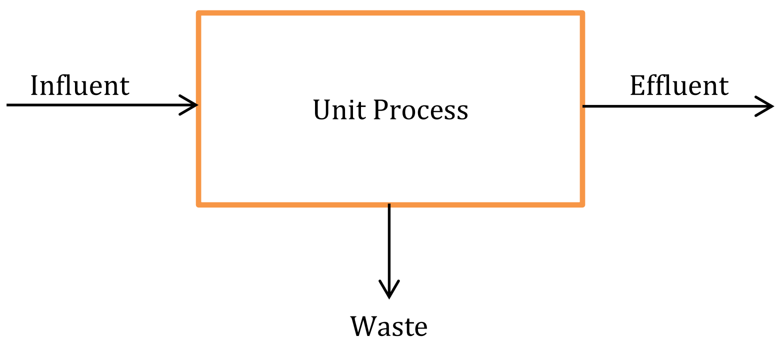

Figure 4 - Typical Unit Process Diagram

A mass balance around a unit process will follow the laws of conservation of mass:

$$Influent\ Mass = Effluent\ Mass + Waste\ Mass$$

Likewise, the flows around a unit process will be conserved as well:

$$Influent\ Flow = Effluent\ Flow + Waste\ Flow$$

Removal Efficiency

The effectiveness of a unit process is described by its removal efficiency:

$$E = \frac{S_{in} - S_{out}}{S_{in}} \times 100\%$$

Where:

E = removal efficiency of unit process in percent.
S_{in} = mass of contaminant entering unit process, Influent mass.
S_{out} = mass of contaminant leaving unit process, Effluent mass.

The same equation can be used to describe the overall removal efficiency of a whole treatment process that includes multiple unit processes. Note that :

$$S_{in} - S_{out} = Waste\ Mass$$

Treatment Trains

Unit processes can be designed to treat liquids or solids. Wastewater treatment plants typically include a liquids process train and a solids process train. The removal of solids, BOD and other contaminants from the liquid produces a solid waste that is processed for disposal in the solids train. A typical plant treatment train is shown in Figure 5.

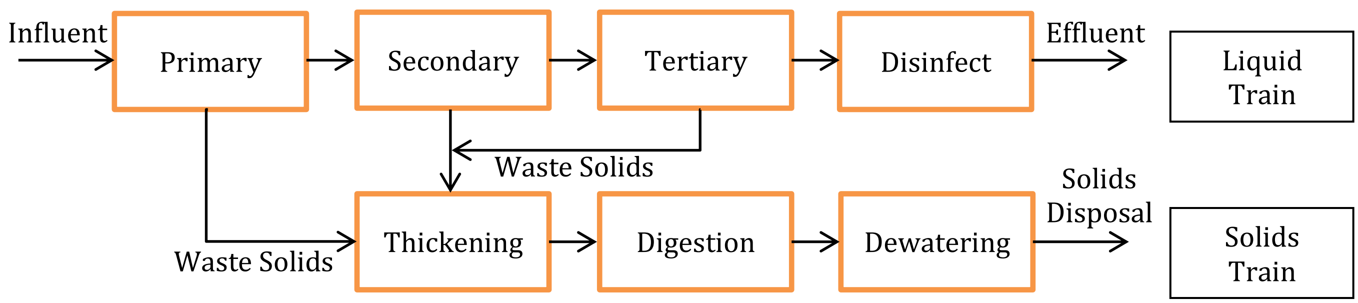

Figure 5 - Typical Treatment Plant Flow Diagram

The liquids treatment train is often divided into categories that describe the level of treatment provided. These categories include primary, secondary, and tertiary. Primary treatment processes are solids removal processes such as screening, grit removal, settling and dissolved air flotation. Secondary processes remove dissolved or soluble contaminants. Tertiary processes remove solids and soluble contaminants to a greater degree than primary and secondary processes and additional contaminants not targeted by primary and secondary processes. A typical wastewater treatment plant will include many or all of the above categories.

Virtually all wastewater treatment plants have a solids treatment train. The exceptions are facilities that use wetlands, stabilization ponds or other natural processes that can store solids for many years before removal is required. Solids treatment processes can include thickening, stabilization, dewatering, drying, composting, incineration, pasteurization, and pelletization to name a few.

Example Problem No. 3

A bar screen at a wastewater treatment plant removes about 15% of the suspended solids in the influent to the plant. The plant treats 15 MGD average flow.

A. The bar screen produces about 4,222 LB/d of solid waste. The influent suspended solids concentration in mg/L is most nearly:

a) 34 b) 150 c) 195 d) 225

B. Which of the following is not a unit process used in the liquid train of a wastewater treatment plant:

a) Primary clarifier b) Cloth disk filters

c) Belt filter press d) Chlorine contact chamber

C. Which of the following is not a unit process used in the solids train of a wastewater treatment plant:

a) Auto-thermophilic aerobic digester b) In-vessel composter

c) Heat pasteurizer d) Cloth disk filters

Primary treatment

These processes are typically used to remove solids and prepare the wastewater for subsequent processing.

Screening

Screening is separated into two categories, coarse and fine. Screens can be bar racks set in channels that are manually raked periodically to remove material. Mechanical screens have self-cleaning rakes or other means to remove the screenings for disposal. Screens come in several types and configurations. Drum screens introduce wastewater on the inside or outside of a rotating drum with fine perforations or wires spaced closely together. As the drum rotates screening are scraped or washed of the drum and collected in slurry. Screens can include washers and compactors that ease handling of the screenings. Screenings are usually disposed of by land filling.

Design factors include the quantity of screenings produced and the hydraulic head loss through the screen. The larger the screen openings the fewer screenings are captured and the lower the head loss through the screen. Head loss through a screen increases as material collects on the screen. Many screens are equipped with devices such as floats or ultrasonic level sensors that sense the water level ahead of the screen and trigger a cleaning cycle when the level exceeds a preset point.

Grinding
Grinding is accomplished with a variety of machines. Grinders and Comminutors are electrically driven or hydraulically driven machines that chop, grind or comminute large solids into small pieces. Design features include safety guards, ability to handle rags and other stringy material and submergibility. Some pumps have grinding or chopping impellers. Hydraulic head loss is the primary design constraint.

Grit Removal
Grit is typically a sandy mixture of hard solids with high specific gravity. When large quantities of grit are pumped in conventional pumps, the grit scores the impellers and volutes. Grit also has a tendency to find its way into the bearings and close tolerances parts of moving equipment and cause excessive wear. Grit can settle in pipes, junction boxes, basins and other places causing clogging and loss of capacity.

Grit has a specific gravity around 2.6 and varies in size from very fine to coarse. Grit is usually fine enough to pass through a #100 mesh screen. A #100 or #150 mesh screen size is usually used in design of grit removal facilities. Grit removed from wastewater flows is usually disposed in a landfill. Various means are available for removing grit including: settling basins, settling channels, aerated grit basins, forced vortex grit basins, and fine screens.

Gravity settling of grit can be accomplished in small settling basins or channels designed for the settling velocity of the grit. However, organic material tends to settle with the grit resulting in the need for thorough grit washing. These facilities can only be designed for very narrow flow ranges.

Aerated grit basins are design by creating a rolling flow pattern with diffused air. The design is similar to a settling basin except the rolling action helps keep the lighter organic material in suspension. An advantage of aerated grit basins is that some pre-aeration is achieved. However, these basins tend to be difficult to control and require adjustment of the airflow to achieve optimum performance.

Forced vortex grit basins are cylindrical chambers with sufficient surface area to settle the design grit. The chambers have a smaller cylindrical grit hopper below the main chamber. Suspended above the grit hopper is an impeller that creates a constant vortex in the chamber that prevents lighter material from settling in the grit hopper. These are very effective at separating grit from wastewater and function well at a wide variety of flow rates.

Fine screens that develop a mat of screened material on their surface have been shown to trap quantities of grit within the screenings. These machines remove screenings constantly through a slow climbing action that raises the screenings above the water surface before dumping them into a container.

Grit separated from the wastewater is usually concentrated and cleaned in a cyclone and classifier combination. Special pumps made with high hardness metals are used to pump the grit slurry from the grit chamber to a cyclone. The cyclone induces a circular velocity to the grit slurry helping to concentrate the grit particles that are thrown to the outside of the circle. This slurry then falls into a small tank where the grit settles to the bottom and is lifted with a screw pump. The screw pump is sized to allow most of the water to drain from the grit as it is pumped to another washer, compactor or hopper.

Primary Sedimentation (Clarification)

Primary sedimentation as the only form of wastewater treatment is being phased out. Today, primary sedimentation is used to reduce the loading on secondary, biological treatment units. Primary sedimentation can reduce influent suspended solids by 50 to 70 percent and influent BOD by 25 to 40 percent.

Sedimentation tanks are designed based primarily on Type 1 discrete particle settling. Tanks are typically designed with:

- a hydraulic retention time of 1.5 to 2.5 hours,
- a surface loading rate of 800 to 1,200 gpd/ft^2
- weir loading rates of 10,000 to 40,000 gpd/ft^2

To design primary sedimentation tanks the surface area required is determined first followed by the depth required to provide the minimum hydraulic retention time. Weirs are sized based on loading rates. Tanks can be rectangular or circular. Rectangular tanks typically have a minimum 2:1 length to width ratio. Various types of raking equipment and withdrawal equipment are available for the removal of the sludge that settles in the tanks.

Dissolved Air Flotation

Dissolved air flotation (DAF) is a solids separation process that floats solids to the surface of a basin for removal. DAF can be effective for a solid that is difficult to settle and is sometimes used in conjunction with chemical processes that produce solids. DAF systems include a high-pressure aeration system followed by a floatation tank with skimmers. Under high pressure the influent is saturated with air. When the saturated mixture is released to the low pressure of the floatation tank the dissolved air off-gases similar to the bubbles created when opening a bottle of club soda. The millions of tiny bubbles created actually lift solids to the surface. At the surface a skimmer skims the solids off into a hopper for further processing.

Design is based on hydraulic loading rates, solids loading rates and on the ratio of the volume of air to the mass of solids, A/S. The A/S ratio can vary from about 0.005 to 0.060 mL(air)/mg(solids). The following equation can be used to calculate the A/S ratio for systems that pressurize the entire influent flow:

$$A/S = \frac{1.3 s_a (fP - 1)}{S_c}$$

Where:

A/S = air to solids ratio, mL/mg
s_a = air solubility, mL/L
f = fraction of air dissolved at pressure P, usually 0.5

P = pressure, atm
p = gage pressure, lb/in^2
S_c = sludge solids, mg/L

Note that:

$$P = \frac{p + 14.7}{14.7}$$

And that:

Temp °C	0	10	20	30
s_a, mL/L	29.2	22.8	18.7	15.7

For systems that pressurize a recycle flow to cause flotation the following equation may be used:

$$A/S = \frac{1.3s_a(fP - 1)R}{S_cQ}$$

Where:

R = pressurized recycle, MGD
Q = influent flow, MGD.

Example Problem No. 4

A wastewater treatment plant treats an average flow of 20 MGD with a peak flow of 35 MGD. The influent has a BOD_5 of 220 mg/L and Total Suspended Solids (TSS) of 250 mg/L. The plant includes fine screens followed by primary clarifiers.

A. If 30% removal of suspended solids is achieved on the fine screens the effluent concentration of TSS in mg/L is most nearly:

a) 70   b) 175   c) 195   d) 225

B. If 30% removal of suspended solids is achieved on the fine screens and 60% removal is achieved in the primary clarifiers then the primary effluent concentration of TSS in mg/L is most nearly:

a) 70   b) 75   c) 105   d) 175

C. The design hydraulic loading surface rate (HLR) for primary clarification is 1,000 GPD/FT^2 for average flow and 2,000 GPD/FT^2 for peak flow. The plant has four primary clarifiers. The minimum diameter of each primary clarifier in FT is most nearly:

a) 80 b) 75 c) 105 d) 160

Secondary Clarification

Secondary Clarification is the separation of biological mass from water following a biological treatment process. Suspended solids from activated sludge processes are called Mixed Liquor Suspended Solids (MLSS) and settled in the secondary clarifiers. Also, suspended solids from trickling filters are settled in secondary clarifiers. This settling provides the final step in the secondary treatment process, so it is sometimes called Final clarification.

Secondary clarification is designed around hindered settling for activated sludge systems and flocculant settling for trickling filter systems. The solids-flux approach is often used for secondary clarifier analysis. Column settling tests are also used. Design parameters such as surface overflow rate determine required surface area and detention time determines volume. Solids loading rates may also control the design. Typical design values are:

Table 1 - Typical Design Values for Secondary Clarifiers

Type of Treatment	Overflow Rate gpd/ft^2		Solids load $Lb/ft^2 \cdot d$		Depth Ft.
	Avg.flow	Peak	Avg.	Peak	
Air activated sludge	400 – 800	1000 – 1200	20 – 30	48	12 – 20
Extended Aeration	200 – 400	600 – 800	5 – 20	34	12 - 20
Trickling Filters	400 – 600	1000 – 1200	15 – 20	38	10 – 15
Rotating Biological Contactors	400 - 800	1000 - 1200	20 - 30	48	10 - 15

Clarifiers can be circular or rectangular. Circular clarifiers are most common because of their greater efficiency and ease with accommodating sludge thickening equipment. Typically the radius of a circular tank should not exceed 5 times the depth.

Example Problem No. 5

Design secondary clarifiers for an activated sludge wastewater treatment plant that treats an average flow of 2 MGD with a peak flow of 6 MGD. It will have two secondary clarifiers designed for a HLR of 800 GPD/FT^2 at average flow and 1,600 GPD/FT^2 at peak flow. The solids loading cannot exceed 14 $LB/d/FT^2$ at average flow. The minimum HRT is 2 HR at peak flow.

A. The maximum suspended solids concentration from the activated sludge reactor is 4,000 mg/L. The minimum diameter of each secondary clarifier in FT is most nearly:

a) 50 b) 56 c) 60 d) 78

B. The minimum depth of each secondary clarifier in FT is most nearly:

a) 8 b) 10 c) 12 d) 14

Chemical Treatment

Chemical treatment of wastewater includes precipitation, adsorption and disinfection.

Chemical precipitation is usually used for removal of phosphorus and for the enhancement of solids removal in primary clarifiers.

Adsorption is used to remove organics that are not removed by conventional physical and biological processes. Adsorption may also be used for dechlorination.

Disinfection is the destruction of disease-causing organisms. Disinfection is discussed in another section of the notes.

Precipitation

Chemical precipitation in wastewater uses chemical addition to alter the state of the dissolved solids and to coagulate suspended solids. The most commonly used chemicals are those used in water treatment for coagulation and softening and include:

- Alum $Al_2(SO_4)_3$
- Ferric Chloride $FeCl_3$
- Ferric Sulfate $Fe_2(SO_4)_3$
- Ferrous Sulfate $FeSO_4$
- Lime $Ca(OH)_2$

The reactions in wastewater treatment are the same as in drinking water treatment, but there are some important considerations when using some chemicals in wastewater. For example, when using ferrous sulfate, lime must also be used and dissolved oxygen is required. The need for dissolved oxygen is present in applications in drinking water treatment as well as in wastewater treatment. However, wastewater is often devoid of or low in dissolved oxygen when drinking water will usually have more than enough.

Phosphorus removal is a common application of chemicals in wastewater treatment. This process is discussed in more detail in the Advanced Treatment section of the notes under Nutrient Removal.

Adsorption

Adsorption is the attraction and accumulation of one substance on the surface of another. Activated Carbon uses adsorption to remove natural and synthetic organic compounds, and disinfectants from wastewater. Organic compounds are attracted to the surface of activated carbon, making activated carbon effective at removing organic solvents, pesticides and herbicides.

Analysis of the adsorptive capacity of activated carbon is based on isotherms determined experimentally. The equation for the isotherm is:

$$\frac{x}{m} = kC^{1/n}$$

Where:

x = the mass of compound adsorbed onto the carbon, lb.
m = the mass of activated carbon, lb.
C = the residual concentration of compound, mg/L.
k and n = isotherm constants determined experimentally for the specific compound and for the specific activated carbon.

For design values, a safety factor is included:

$$\left(\frac{x}{m}\right)_d = kC_0^{-1/n} * SF$$

Where:

$\left(\frac{x}{m}\right)_d$ = the design capacity of the GAC to retain the compound to be removed in lb of compound per lb of GAC.
C_0 = Influent Concentration, mg/L
SF = Safety factor for design, usually between 0.75 and 0.90.
k and n = isotherm constants.

Activated carbon is discussed in more detail in Water Treatment as it is more commonly used to remove specific contaminants from drinking water.

Example Problem No. 6

Chemical treatment can also be used in wastewater treatment plants. A plant feeds alum to remove phosphorous from the wastewater and uses activated carbon to remove odors.

A. The chemical process that effects the removal of phosphorous with alum is referred to as:

a) sedimentation

b) adsorption

c) oxidation

d) precipitation

B. The chemical process that effects the removal of odor causing compounds by feeding foul air through activated carbon is called:

a) sedimentation

b) adsorption

c) oxidation

d) precipitation

National Pollution Discharge Elimination System (NPDES) permitting

The National Pollutant Discharge Elimination System (NPDES) Program has achieved significant reductions in pollutant discharges since it was established by the Federal Water Pollution Control Act Amendments of 1972. The development of this permitting program has, in turn, resulted in tremendous improvement to the quality of this country's water resources.

Under NPDES, all facilities which discharge pollutants from any point source into waters of the United States are required to obtain a permit. The permit provides two levels of control: technology-based limits (based on the ability of dischargers in the same industrial category to treat wastewater) and water quality-based limits (if technology-based limits are not sufficient to provide protection of the water body).

Section 304(a)(4) of the Clean Water Act of 1977 (CWA) designated the following as *conventional pollutants*:

- Five day biochemical oxygen demand (BOD_5),
- Total suspended solids (TSS),
- pH,
- Fecal coliform, and
- Oil and grease (O&G) (added in 1979 in 40 CFR §401.16).

Section 307(a)(1) of the CWA required the establishment of a published list of toxic pollutants or combination of pollutants often called the priority pollutants (listed in 40 CFR §401.15). Originally 65 toxic pollutants and classes of pollutants were identified; later this list was expanded to 126 pollutants and classes of pollutants. Substances, such as chlorine or ammonia that are not specifically listed as conventional or toxic pollutants, are called non-conventional pollutants.

Non-conventional pollutants are those which do not fall under either of the above categories, and include such parameters as ammonia, nitrogen, phosphorus, chemical oxygen demand (COD), and whole effluent toxicity (WET).

Pollutants can enter waters of the United States from a variety of pathways including agricultural, domestic, and industrial sources**.** For regulatory purposes these sources are generally categorized as either *point sources* or *non-point sources*. Typical *point source* discharges include discharges from publicly owned treatment works (POTWs), discharges from industrial facilities, and discharges associated with urban runoff. While provisions of the NPDES Program do address certain specific types of agricultural activities (i.e., concentrated animal feeding operations), the majority of agricultural facilities are defined as *non-point sources* and are exempt from NPDES regulation.

Pollutant contributions to waters of the United States may come from both *direct* and *indirect* sources. *Direct* sources discharge wastewater directly into the receiving water body, whereas *indirect* sources discharge wastewater to a POTW, which in turn discharges into the receiving water body. Under the national program, NPDES permits are issued only to direct point source discharges. Industrial and commercial indirect dischargers are addressed by the National Pretreatment Program.

As indicated above, the primary focus of the NPDES permitting program is municipal and non-municipal (industrial) direct dischargers. Within these major categories of dischargers, however, there are a number of more specific types of discharges that are regulated under the NPDES Program.

Municipal treatment works and other facilities are regulated by NPDES permits and affected by other NPDES programs including:

- the National Pretreatment Program,
- the Municipal Sewage Sludge Program,
- Combined Sewer Overflows (CSOs), and
- the Municipal Storm Water Program.

Non-municipal dischargers are regulated under industrial programs including:

- Process Wastewater Discharges,
- Non-process Wastewater Discharges, and
- the Industrial Storm Water Program.

Effluent Limits

Effluent limits by law must address the conventionalal pollutants as a minimum. Thus all wastewater discharges regulated under the NPDES program include limits for BOD_5, TSS, pH, fecal coliform, and oil and grease. Effluent limitations serve as the primary mechanism in NPDES permits for controlling discharges of pollutants to receiving waters. When developing effluent limitations for an NPDES permit, a permit writer must consider limits based on both the technology available to control the pollutants (i.e., technology-based effluent limits) and limits that are protective of the water quality standards of the receiving water (i.e., water quality-based effluent limits). [i]

Technology-Based Limits

The intent of technology-based effluent limits in NPDES permits is to require a minimum level of pollutant removal for point source discharges based on available treatment technologies, while allowing the discharger to use any available control technique to meet the limits.[ii]

Industrial (and other non-municipal) facility effluent limits are set based upon the national effluent limitations guidelines established by EPA, or in the absence of established national guidelines, by using best professional judgment on a case-by-case basis.

Municipal and other Publicly Owned Treatment Works (POTW) effluent limits are based on the National Secondary Treatment Standards. Since municipal and other POTWs treat mostly domestic wastewater, biological processes are very successful and form the standard for secondary treatment. The secondary treatment standards as promulgated in 40 CFR Part 133 are as follows:

Table 2 - National Secondary Treatment Standards

Parameter	30-Day Average	7-Day Average
5-Day BOD	30 mg/L	45 mg/L
5-Day CBOD (may be used instead of BOD)	25 mg/L	40 mg/L
Total Suspended Solids (TSS)	30 mg/L	45 mg/L
pH	6-9 s.u. (instantaneous)	-
Minimum Removal	85% BOD_5 and TSS	-

In addition to the limits specified as concentrations as shown in Table 2, the NPDES requires permits to include mass-based limits calculated using the concentrations with the plant design flow. The $CBOD_5$ limits shown in Table 2 may be substituted for BOD_5 limits to avoid erroneous measurements caused by nitrification in the effluent.

Water Quality-Based Limits

NPDES requires that permits include limits to protect water quality and ensure that receiving water quality standards are not violated. When technology-based limits are not sufficient to ensure that water quality standards will be attained, more stringent water quality-based limits must be imposed.

- **Water quality standards** set under Section 303 of the Clean Water Act require that appropriate effluent limits be included in the NPDES permits to effect attainment of the standards.
- **Watershed-based permitting** is used to take into account the broader effects of the entire watershed in which the discharge is located.
- Impaired watersheds may have **Total Maximum Daily Loads (TMDL)** set that dictate effluent limits of discharges to the stream.
- **Whole effluent toxicity** testing is required in NPDES permits to protect the receiving water quality from the aggregate toxic effect of a mixture of pollutants in an effluent discharge.

Example Problem No. 7

A publicly owned treatment works (POTW) with a design flow rate of 4.0 MGD must meet National Secondary Treatment Standards as shown in Table 2.

A. The 30-day average $CBOD_5$ mass limit in the NPDES permit will be most nearly:

a) 25 LB/d b) 100 LB/d c) 834 LB/d d) 1501 Lb/d

B. The 7-day average TSS mass limit in the NPDES permit will be most nearly:

a) 25 LB/d b) 100 LB/d c) 834 LB/d d) 1501 Lb/d

Biological Treatment

Biological treatment is the culture and use of microorganisms (biomass) to consume or convert dissolved and suspended compounds in the influent to remove them from the water. Carbonaceous compounds that are consumed by the biomass are converted to more microorganisms and energy. Thus, the dissolved compounds become solid in the form of the biomass. Biomass is separated from the water by settling. Some compounds are converted to other forms such as the conversion of ammonia to nitrite and nitrate by the nitrification process. Nitrification is discussed in another section. Thus, the primary purpose of biological treatment is to reduce the biochemical oxygen demand of the incoming wastewater.

Biochemical Oxygen Demand

Biochemical oxygen demand (BOD) is the quantity of oxygen utilized by a mixed population of microorganisms in aerobic oxidation at a temperature of 20°C. This is considered a measure of the biodegradable organic material in the water since the demand for oxygen by the microorganisms is related to the quantity of organic material available for energy and reproduction. A high BOD indicates a large quantity of biodegradable material in the water while a low BOD indicates a smaller quantity. The BOD does not indicate the form, type or quality of biodegradable material, only the approximate oxygen required to biodegrade the material.

Testing for the concentration of BOD is performed with samples that are diluted into various test dilutions and placed into **300 mL BOD bottles**. The dissolved oxygen content is measured at the beginning of the test. The test dilutions are incubated at 20°C for a specified number of days, standard is five, and the dissolved oxygen content is again measured. The BOD for the test is calculated as follows:

Unseeded Test:

$$\mathrm{BOD}_t = \frac{D_i - D_t}{p}$$

Seeded Test:

$$\mathrm{BOD}_t = \frac{(D_i - D_t) - (B_i - B_t)f}{p}$$

Where:

BOD_t = the biochemical oxygen demand at time t, (mg/L)
D_i = the initial dissolved oxygen concentration, (mg/L)
D_t = the dissolved oxygen concentration at time t, (mg/L)
p = the decimal fraction of sample water in test
B_i = the initial dissolved oxygen concentration of the seed control, (mg/L)
B_t = the dissolved oxygen concentration of the seed control at time t, (mg/L)
f = the ratio of seed in sample to seed in control

BOD increases with the length of time used for the test as shown in Figure 6. If the time used in the test is infinite, then the ultimate BOD would be found. The ultimate BOD is the greatest quantity of oxygen required to fully biodegrade the biodegradable material in the sample.

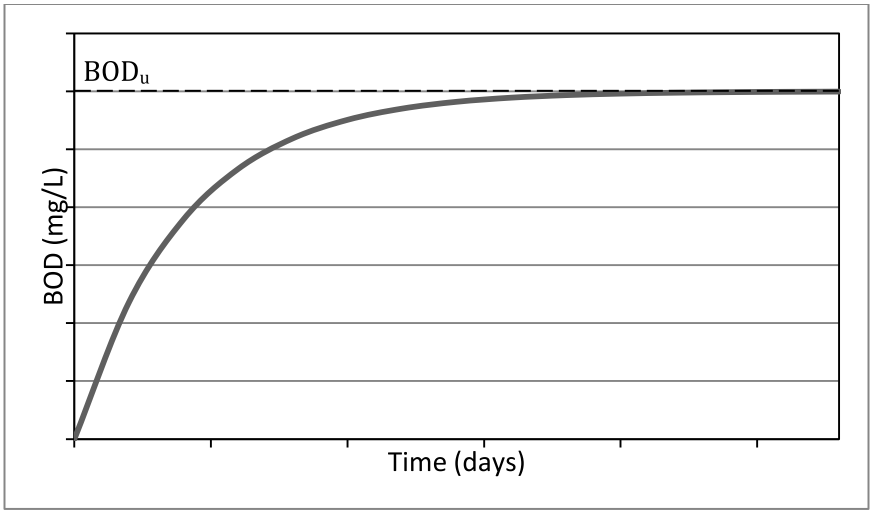

Figure 6 - Biochemical Oxygen Demand as Function of Time

The ultimate BOD is related to the BOD test for a specified period of time by the following relation:

$$\mathrm{BOD}_t = \mathrm{BOD}_u\left(1 - e^{-kt}\right)$$

Where:

BOD_u = the ultimate BOD, (mg/L)
k = a decay constant determined experimentally, (d^{-1})
t = time, days

Values for k vary from 0.10 to 0.70 depended on the character of the material tested.

Carbonaceous BOD (CBOD) is the oxygen exerted by carbonaceous materials in the sample. The measure of CBOD is obtained by inhibiting the growth of nitrifying bacteria with an inhibitory agent.

Example Problem No. 8

Samples of wastewater are taken from the influent of a wastewater treatment plant. Lab analysis for BOD is performed to determine the ultimate BOD. If the BOD_5 is 220 mg/L and the BOD_{10} is 300 mg/L then the ultimate BOD in mg/L is most nearly:

a) 256 b) 348 c) 369 d) 402

Biological Processes
Biological processes can be aerobic, facultative or anaerobic.

Aerobic Processes need free dissolved oxygen in the water to promote growth of the biomass. These are the most common types of biological processes used in the treatment of domestic wastewater. The oxygen is provided by aeration system or by pure oxygen systems.

Facultative processes rely on biomass that can grow with free dissolved oxygen and without free dissolved oxygen. These processes do not operate in a strictly aerobic state nor do they operate in a strictly anaerobic state, but somewhere in between. The dissolved oxygen level in this treatment system is very low. Oxygen required is often supplied by atmospheric interaction with the water in the treatment process.

Anaerobic processes rely on biomass that grows in the absence of free dissolved oxygen. These processes are known for producing odorous gases such as methane. These have an advantage of not requiring the addition of oxygen to the water. However, they are seldom used in treatment of domestic wastewater, but are often used in treatment of industrial wastewater.

Biological processes can also be divided into two groups based on the means of cultivating the biomass growth: suspended growth and attached growth.

Suspended growth processes cultivate the biomass growth in a mixture or suspension of biomass. The biomass is constantly mixed by external means to maintain the suspension. The mixing can be provided by mechanical equipment or by aeration.

Attached growth processes provide a stable media for the cultivation of the biomass. The biomass attaches to the media as it is exposed to the wastewater and growth occurs.

An example of suspended growth processes is the activated sludge process. An example of attached growth processes is the trickling filter process.

Activated Sludge
Activated sludge is a term used to describe a number of different aerobic suspended growth processes. Conventional activated sludge is designed for carbonaceous BOD conversion in a complete mix aerobic reactor. The reactor is often referred to as an aeration basin. Conventional activated sludge is mixed and aerated with diffused air provided by blowers and diffusers.

The activated sludge process involves:

1. wastewater aeration in the presence of a microbial suspension,
2. solid-liquid separation following aeration,
3. discharge of clarified effluent,
4. wasting of excess biomass, and
5. return of remaining biomass to the aeration tank.

In the activated sludge process, wastewater containing organic matter is aerated in an aeration basin in which micro-organisms metabolize the suspended and soluble organic matter. Part of the organic matter is synthesized into new cells and part is oxidized to CO_2 and water to derive energy. In activated sludge systems the new cells formed in the reaction are removed from the liquid stream in the form of a flocculent sludge in settling tanks. A part of this settled biomass, described as activated sludge is returned to the aeration tank and the remaining forms waste or excess sludge.

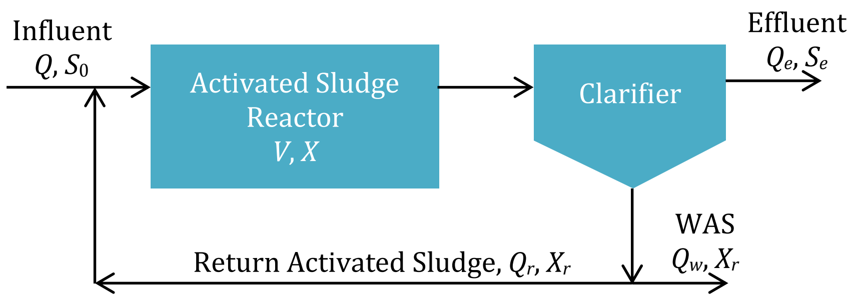

Figure 7 - Typical Activated Sludge Flow Sheet

The flocculent sludge in the activated sludge reactor is a mixture of biomass and other suspended solids that is referred to as the Mixed Liquor Suspended Solids (MLSS). The portion of the MLSS that is biomass can be estimated as the volatile fraction of the MLSS, called the Mixed Liquor Volatile Suspended Solids (MLVSS). The main variables of activated sludge process are the mixing regime, loading rate, and the flow scheme.

Mixing Regime

Generally two types of mixing regimes are of major interest in activated sludge process: ***plug flow*** and ***complete mixing***. In the first one, the regime is characterized by orderly flow of mixed liquor through the aeration tank with no element of mixed liquor overtaking or mixing with any other element. There may be lateral mixing of mixed liquor but there must be no mixing along the path of flow.

In complete mixing, the contents of aeration tank are well stirred and uniform throughout. Thus, at steady state, the effluent from the aeration tank has the same composition as the aeration tank contents.

The type of mixing regime is very important as it affects (1) oxygen transfer requirements in the aeration tank, (2) susceptibility of biomass to shock loads, (3) local environmental conditions in the aeration tank, and (4) the kinetics governing the treatment process.

Loading Rate

Several loading parameters are used to design activated sludge systems. See Figure 7 for a typical flows sheet and the variables used in the following equations. A loading parameter that has been developed over the years is the ***hydraulic retention time*** (HRT), θ, days.

$$\theta = \frac{V}{Q10^6}$$

Where:

V = volume of the aeration tank, Gallons.

Q = influent wastewater flow, MGD.

Another empirical loading parameter is ***volumetric organic loading rate*** (OLR) which is defined as the BOD applied per unit volume of aeration tank, per day.

$$OLR = \frac{QS_0 8.34}{V10^{-3}}$$

Where:

OLR = organic loading rate, lb/d/ 1,000 FT^3.
Q = influent wastewater flow, MGD.
S_0 = influent organic concentration, BOD_5, mg/L.
V = volume of the aeration tank, FT^3.

A rational loading parameter which has found wider acceptance and is preferred is ***specific substrate utilization rate***, U, $days^{-1}$.

$$U = \frac{Q(S_0 - S_e)}{VX}$$

A similar loading parameter is ***mean cell residence time*** (MCRT) or ***sludge retention time*** (SRT), θ_c, days.

$$\theta_c = \frac{VX}{Q_w X_r + (Q - Q_w)X_e}$$

Where:

U = specific substrate utilization rate, d.
Θ_c = mean cell residence time, d.
S_0 = Influent organic concentration, BOD_5, mg/L.
S_e = Effluent organic concentration, BOD_5, mg/L.
X = mixed liquor suspended solids (MLSS) concentration in the aeration tank, mg/L.
X_e = MLSS concentration in the effluent, mg/L.
X_r = Return Activated Sludge (RAS) concentration, mg/L.
Q = influent wastewater flow rate, MGD.
Q_w = Waste Activate Sludge (WAS) flow rate, MGD.
V = volume of the aeration tank, MG.

Under steady state operation the mass of waste activated sludge is given by:

$$P_x = Q_w X_r 8.34 = \frac{YQ(S_0 - S_e)8.34}{1 + \theta_c k_d}$$

Where:

P_x = mass of waste activated sludge(WAS), lb/d.
Y = maximum yield coefficient (microbial mass synthesized / mass of substrate utilized), lb/lb.

θ_c = mean cell residence time, d.
k_d = endogenous decay rate, d^{-1}.
S_0 = Influent organic concentration, BOD_5, mg/L.
S_e = Effluent organic concentration, BOD_5, mg/L.
Q = influent wastewater flow rate, MGD.
Q_w = Waste Activate Sludge (WAS) flow rate, MGD.
X_r = Return Activated Sludge (RAS) concentration, mg/L.

From the above equation it is seen that:

$$\frac{1}{\theta_c} = YU - k_d$$

If the value of S_e is small as compared S_0, U may also be expressed as ***Food to Microorganism ratio***, F/M:

$$F/M = \frac{QS_0}{XV} \approx U = \frac{Q(S_0 - S_e)}{XV}$$

The oxygen required by the system for carbon conversion is given by:

$$O_2 = \frac{Q(S_0 - S_e)8.34}{f} - 1.42P_x$$

Where:
O_2 = Oxygen required, LB/d.
Q = influent wastewater flow rate, MGD.
S_0 = Influent organic concentration, BOD_5, mg/L.
S_e = Effluent organic concentration, BOD_5, mg/L.
P_x = mass of waste activated sludge, LB/d.
f = factor to convert BOD_5 to ultimate BOD, unitless (usually between 0.45 and 0.68).

The θ_c value adopted for design controls the effluent quality, and settleability and drainability of biomass, oxygen requirements and quantity of waste activated sludge.

Flow Scheme
The flow scheme involves:

- The pattern of wastewater addition.
- The pattern of sludge return to the aeration tank.
- The pattern of aeration.

Wastewater addition may be at a single point at the inlet end or it may be at several points along the aeration tank. The sludge return may be directly from the settling tank to the aeration tank or through a sludge re-aeration tank. Aeration may be at a uniform rate or it may be varied from the head of the aeration tank to its end.

Conventional System and its Modifications

The conventional system maintains a ***plug flow*** hydraulic regime or a ***completely mixed*** reactor. ***Plug flow*** is the progression of flow from the influent end of the unit to the effluent end. In a ***completely mixed*** reactor, the influent waste and return sludge are instantaneously mixed with the entire contents of the aeration tank. Modifications to the conventional flow scheme include:

- ***Step aeration:*** settled wastewater is introduced at several points along the length of the aeration tank with a plug flow regime to produce more uniform oxygen demand throughout.
- ***Tapered aeration*** attempts to supply air to match oxygen demand along the length of the aeration tank with a plug flow regime.
- ***Contact stabilization*** provides for reaeration of return activated sludge from the final clarifier, which allows for a smaller aeration or contact tank.
- ***Extended aeration*** operates at a low organic load and long MCRT producing lesser quantity of well stabilized sludge.

Plug Flow Reactor

In a plug flow reactor, the activated sludge reactor shown in Figure 7 has a plug-flow regime. This means that the influent is spread across the width of the reactor and moves as a unit across the length of the reactor to the final clarifier. Thus, the concentration of biomass changes as the microorganisms consume the substrate (BOD_5) in the wastewater. If we let $\bar{X}$ be the average microorganism concentration in the reactor, then the substrate utilization rate under steady state conditions is given by:

$$r_{su} = \frac{kS_e\bar{X}}{K_s + S_e}$$

Where:

r_{su} = rate of substrate (BOD_5) utilization, d^{-1}.
k = maximum specific substrate utilization rate per unit mass of microorganisms, d^{-1}.
S_e = effluent substrate (BOD_5) concentration, mg/L.
K_s = half-velocity constant, mg/L.
$\bar{X}$ = average biomass concentration in the reactor, mg/L.

A given substrate utilization rate can be used to determine the average biomass concentration by using the following equation to determine the effluent BOD_5 under steady state conditions:

$$\frac{1}{\theta_c} = \frac{Yk(S_0 - S_e)}{(S_0 - S_e) + (1+\alpha)K_s \ln(S_i/S_e)} - k_d$$

Where:

θ_c = mean cell retention time, d.
Y = yield coefficient, LB biomass/LB BOD_5.
k = maximum specific substrate utilization rate per unit mass of microorganisms, d^{-1}.
K_s = half-velocity constant, mg/L.
k_d = endogenous decay rate, d^{-1}.
S_0 = Influent organic concentration, BOD_5, mg/L.

S_e = Effluent organic concentration, BOD_5, mg/L.

α = recycle ratio = $\frac{Q_r}{Q}$.

S_i = influent concentration to reactor after dilution with return activated sludge (RAS) = $\frac{S_0+\alpha S_e}{1+\alpha}$.

Complete Mix Reactor

If the influent to the activated sludge reactor in Figure 7 is completely mixed with the contents of the reactor upon entry, then the complete mix model can be used. The concentration of biomass in the complete mix reactor under steady state conditions can be calculated by:

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

And the effluent substrate (BOD_5) concentration can be computed by:

$$S_e = \frac{K_s(1 + \theta_c k_d)}{\theta_c(Yk - k_d) - 1}$$

Where:

X = concentration of microorganisms (biomass)in the reactor, mg/L.

S_e = Effluent organic concentration, BOD_5, mg/L.

S_0 = Influent organic concentration, BOD_5, mg/L.

Θ = hydraulic retention time in reactor, V/Q, d.

Θ_c = mean cell retention time, d.

Y = yield coefficient, LB biomass/LB BOD_5.

k = maximum specific substrate utilization rate per unit mass of microorganisms, d^{-1}.

K_s = half-velocity constant, mg/L.

k_d = endogenous decay rate, d^{-1}.

Example Problem No. 9

A POTW with a design flow rate of 4.0 MGD uses the activated sludge process with a complete mix reactor to achieve secondary treatment of the wastewater. The BOD_5 and TSS in the primary effluent is 190 mg/L and 100 mg/L, respectively. The kinetic coefficients for the process are given in Table 3. The mean cell retention time is 10 days.

Table 3 – Typical Biological Kinetic Coefficients

Biological Kinetic Coefficient	Value
Growth Yield, Y, lb biomass/lb BOD_5	0.6
Max. specific substrate utilization rater per unit mass of microorganisms, k, d^{-1}	5
Half-velocity constant, K_s, mg/L	60
Endogenous decay rate, k_d, d^{-1}	0.06
Ratio BOD_5 to BOD_u, f	0.63

A. The plant will produce an effluent with a BOD_5 in mg/L that is most nearly:

a) 3.38 b) 5.92 c) 12.5 d) 22.3

B. The design biomass concentration is 2,500 mg/L in the activated sludge reactor. The volume of the reactor in FT^3 is most nearly:

a) 50,000 b) 100,000 c) 120,000 d) 150,000

C. The mass of microorganisms in LB/d that must be removed from the system to maintain steady state conditions is most nearly:

a) 9,338 b) 3,803 c) 2,335 d) 1,119

D. The mass of oxygen in LB/d that must be added to the system to maintain steady state conditions is most nearly:

a) 6,338 b) 3,315 c) 2,335 d) 6,566

Trickling Filters

A trickling filter consists of a deep bed of a permeable medium (packing material) to which microorganisms can attach and grow. Wastewater is percolated or trickled through the packing material to promote the growth of microorganisms. Packing material is usually rock, gavel, slag, sand, redwood, and a variety of plastic and other synthetic media.

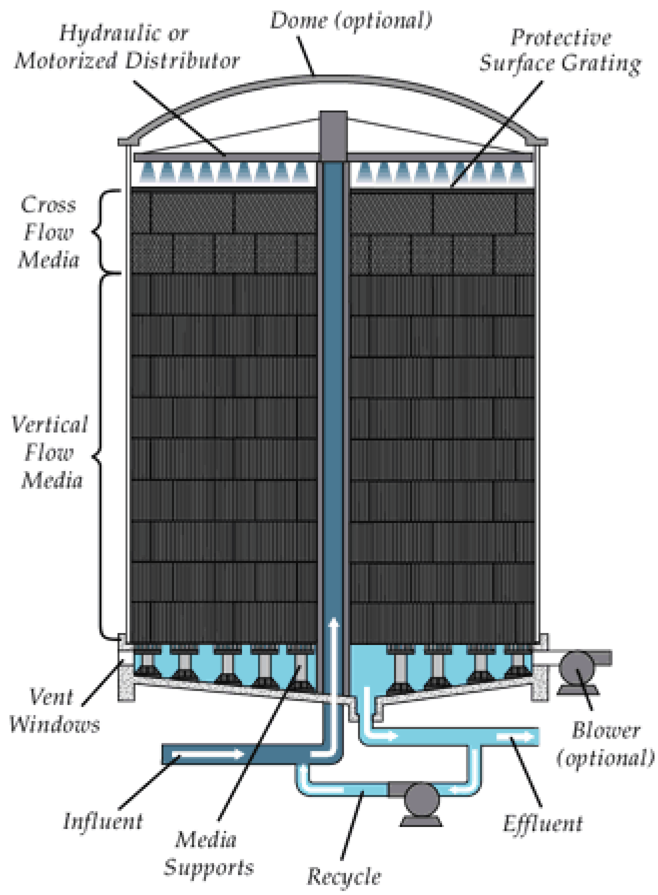

Figure 8 - Modern Trickling Filter with Brentwood Industries Media[iii]

Process Description

- Wastewater is fed into the trickling filter and trickled over the top of the media by means of a spray distributor that rotates to evenly distribute the flow.

- As the wastewater trickles down through the media air is drawn up from openings in the bottom by the chimney effect or is forced up through the media by blowers. The air circulation provides oxygen to the biomass.
- Biomass grows in the form of a slime on the media. Organic material in the wastewater provides food for sustenance and growth. The biomass synthesizes the organic material and forms new cell material. In addition, nitrification (conversion of ammonia to nitrites and nitrates) can occur within the filter.
- As the slime layer thickens with growth, oxygen cannot be transferred throughout the slime layer. Eventually, the slime cannot continue to maintain its hold to the media and it sloughs off and new slime begins to grow in its place.
- The release of the biomass slime from the media is called **sloughing**. Sloughing produces solids in the effluent from the trickling filter that is usually settled in secondary clarifiers, leaving the water with less organic material and thus a lower BOD_5.

Types of Trickling Filters

The processes vary based upon hydraulic and organic loading rates, but can be grouped into two classes: High-rate and Low-rate trickling filters.

Table 4 - Typical Trickling Filter Characteristics

	Low Rate	**High Rate**
Hydraulic Loading Rate (GPD/FT^2)	25 – 100	250 – 1,000
Organic Loading Rate (lb/d BOD_5 per 1,000 FT^3)	5 – 20	20 – 60
Media Depth (FT)	6 – 10	3 – 8
Recirculation Ratio	0	0.5 – 3 (domestic wastewater) Up to 8 for industrial wastewater

- The hydraulic loading rate is the flow applied per unit surface area of the media at the top of the trickling filter per day. The organic loading rate is the mass of BOD_5 applied per unit volume of the media in the trickling filter per day.
- Low rate trickling filters can achieve a 75% to 90% BOD removal and produce a highly nitrified effluent. Low rate filters typically do not benefit from recirculation and are often only single stage filters. Low rate filters are suitable for low to medium strength domestic wastewaters.
- High rate trickling filters can achieve 75% to 90% BOD removal with partial nitrification. High rate filters are often operated in two stages with an intermediate settling basin between the stages. High rate filters are used for medium to high strength domestic wastewaters and industrial wastewaters.

Process Design

The design of trickling filters is performed using empirical relationships developed based on operating filter performance and other experience. Atkinson and colleagues proposed a mass-balance approach based upon the rates of synthesis of organic material into the slime. However, in practice this rational approach has seldom been successful in modeling actual performance. The most commonly used approaches include the NRC equations for rock, gravel and slag filters, and either the Eckenfelder or the Germain and Schulz equations for plastic media.

NRC Equations

The NRC equations are primarily applicable to single-stage and multi-stage rock systems with recirculation. For a single stage or first stage rock filter:

$$E = \frac{100}{1 + 0.0561\sqrt{\frac{W}{VF}}}$$

Where:

E = efficiency of BOD removal for process at 20°C, including recirculation and sedimentation, percent
W = BOD loading to filter, lb/d
V = volume of filter media, 10^3 ft^3
F = recirculation factor

$$F = \frac{1 + R}{\left(1 + {}^{R}/_{10}\right)^2}$$

Where:

R = recirculation ratio = Q_r/Q
Q_r = recirculation flowrate
Q = wastewater flowrate

The equation for the second stage filter is:

$$E_2 = \frac{100}{1 + \frac{0.0561}{1 - E}\sqrt{\frac{W'}{VF}}}$$

Where:

E_2 = efficiency of BOD removal for second stage filter at 20°C, including recirculation and settling, percent
W'= BOD loading applied to second stage filter, lb/d

Adjustments to the efficiency based on wastewater temperature are given by:

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

Where:

E_T = Efficiency of BOD removal at temperature T, percent

E = efficiency of BOD removal at 20°C given by the NRC equations, percent
T = wastewater temperature, °C

Eckenfelder Equation

Eckenfelder proposed the following equation to describe the performance of trickling filters with plastic media:

$$\ln \frac{S_e}{S_i} = -KS_a^m D(Q_V)^{-n}$$

Where:

S_e = BOD of settled effluent from filter, mg/L
S_i = BOD of wastewater applied to filter, mg/L
K = observed reaction rate constant for a given depth of filter, FT/d
S_a = specific surface area of filter $= \frac{unit\ surface\ area\ A_S,\ FT^2}{unit\ volume\ V,\ FT^3}$
D = filter depth, FT
Q_v = volumetric flowrate applied per unit area of filter, $FT^3/d \cdot FT^2$
m, n = empirical constants

Germain and Schultz Equation

Germain and Schultz proposed the following equation for plastic media trickling filters:

$$\ln \frac{S_e}{S_i} = -k_{20} D(Q_V)^{-n}$$

Where:

k_{20} = treatability constant corresponding to a specific filter medium of depth D at 20°C, units vary with exponent *n.*
n = experimental constant, usually 0.5

The treatability constant takes into account the reaction rate and the specific surface area of the filter medium. The treatability constant can be adjusted for temperature using the following equation:

$$k_T = k_{20}(1.035)^{T-20}$$

Since the treatability constant is derived for a filter of specific depth, it must be adjusted when designing for a different depth using the following equation:

$$k_2 = k_1 \left(\frac{D_1}{D_2}\right)^x$$

Where:

k_1 = treatability constant corresponding to a filter of depth, D_1.

k_2 = treatability constant corresponding to a filter of depth, D_2.
D_1 = depth of filter one, FT.
D_2 = depth of filter two, FT.
x = 0.5 for vertical plastic media or rock media filters
0.3 for cross flow plastic media filters

Example Problem No. 10

A POTW with a design flow rate of 0.50 MGD uses the trickling filter process to achieve secondary treatment of the wastewater. The BOD_5 and TSS in the primary effluent is 190 mg/L and 100 mg/L, respectively. The plant includes two, single stage, low rate tickling filters that operate in parallel, each with a rock media volume of 26,410 FT^3. The water temperature is 20°C and there is no recirculation.

A. Using the NRC equations the efficiency of the trickling filters is most nearly:

a) 82% b) 92% c) 75% d) 63%

B. Using the NRC equations, the effluent BOD_5 in mg/L is most nearly:

a) 23 b) 34 c) 45 d) 16

Solids Handling

Solids are removed from the liquid train and processed separately. Some solids are simply removed from the system and sent to a landfill, such as screenings and grit. Other solids require additional handling and treatment before disposal, such as primary sludge and biological sludge (secondary sludge). Often primary sludge and secondary sludge requires stabilization before disposal. Stabilization is achieved by a number of different methods including digestion. Digestion is discussed in the section on Digesters. Primary and secondary sludge is often thickened prior to digestion and dewatered or dried following digestion.

Thickening

Thickening is the removal of excess water to increase the concentration of solids in the sludge slurry. Sludge thickening or concentration is employed to increase efficiency of subsequent processing units. Thicker sludge can result in smaller tanks, lower chemical dosages, and less heat required.

Gravity Thickening is accomplished in specially designed tanks similar to secondary clarifiers. Sludge collection equipment includes deep trusses or vertical pickets to induce stirring and cause the release of water from the sludge. Thickened sludge is drawn from the bottom of the tank. Supernatant (water) is drawn out of the top and returned to the head of the plant. Gravity thickeners are designed based upon hydraulic loading rates and solids loading rates.

Gravity Belt Thickeners are machines with wide porous fabric conveyor belts on which sludge is placed. As the belt carries the sludge from end of the machine to the other, the belt is vibrated and the sludge is plowed to help release liquid from the sludge that pours through the belt. Thickened sludge can then be released directly to dewatering equipment or piped to the next unit process. Gravity belt thickeners are sold in widths measured in meters between 1 and 3 meters wide. They are sized based on solids loading in lb/d per meter of belt width and hydraulic loading in GPM per meter if belt width.

Flotation Thickening involves the use of air combined with the sludge under high pressure. When the sludge/air mixture is released in a tank the air disperses in tiny bubbles floating the solids to the surface of the tank. The solids are then collected from the surface. Flotation thickening is designed in a fashion similar to dissolved air flotation discussed in the section on Primary Treatment.

Centrifugal Thickening uses centrifugal forces to settle solids. Centrifuges are also used to dewater sludge. Solid bowl thickeners are usually mounted horizontally with a tapered end. Sludge is introduced into the spinning bowl continuously. A helical screw turns at a slower speed to carry the cake sludge to the tapered end where it is concentrated and discharged. Perforated bowl centrifuges operate in batches. The bowl is loaded with sludge and then started. The spinning of the bowl causes the sludge to thicken at the walls of the bowl while water pours through the perforations. The centrifuge is sized based on hydraulic loading rates and solids loading rates.

Dewatering

Dewatering is the removal of a sufficient amount of water so that the solids can be transported without leaking out of the container. This is usually a minimum of 15% solids, but is often preferred to be greater than 20% solids. For transportation to a land fill or in solid waste containers, sufficient excess water must be removed to pass the paint filter test.

Paint Filter Test

The paint filter test uses an ordinary paper filter designed to filter paint and a graduated cylinder. The paint filter is placed in the cylinder so that all water that drips through the filter will be captured by the cylinder. A scoop of sludge, about 500mL, is placed in the paint filter. If after 30 minutes, no liquid is found in the graduated cylinder, the sample passes. A passing sample indicates a sufficient level of dewatering for transportation.

Centrifugal Dewatering
Centrifugal dewatering is similar to centrifugal thickening described above. Dewatering results in a higher solids concentration, usually around 20%. To achieve this higher level of moisture reduction, the centrifuge operates at a higher speed and for a longer time.

Belt Filter Press
A belt filter press is similar to a gravity belt thickener and often incorporates a thickener within the same machine. It uses porous fabric belts to convey the sludge through the machine. A second porous fabric belt is positioned above the conveyor. The two belts are passed between rollers that squeeze the belts together and thus the sludge that is between the belts. The clearance between successive rollers is smaller to increase the pressure on the sludge to release the water. Belt filter presses are sold in sizes based on the width of the belt in meters. A 2.0 meter belt is most common in municipal wastewater treatment plants. The units are sized based on solids loading in LB/d per meter of belt width and on hydraulic loading in GPM per meter of belt width.

Drying
Drying is the next step in moisture removal beyond dewatering. Drying is an attempt to achieve solids concentrations greater than 50%. Drying can be achieved with the use of pressure, heat, sunlight, evaporation and time. Sludge drying beds are commonly used to dry sludge. Since the performance of sludge drying beds is dependent upon the climate, sizing of beds differs based on humidity, average rainfall and average temperature. Heat drying is an energy intensive process, but it can achieve less than 10% moisture in the finished product.

Example Problem No. 11
A wastewater treatment plant produces waste activated sludge from the extended aeration process. The sludge is disposed of in a landfill. Currently the plant uses a belt filter press to dewater the sludge before hauling to the landfill. The operations staff is considering upgrading the facility to add a heat drying process to reduce hauling costs and tipping fees. The belt filter press achieves an average of 17% solids. The heat drying process will produce 90% solids. If it costs $10 per ton to dispose of the solids and the plant produces 2,000 LB of dry solids per day, the annual cost savings to be realized by installing the heat drying system is most nearly:

a) $4,055 b) $17,400 c) $21,470 d) $3,650

Digesters

Anaerobic Digestion

Anaerobic sludge digestion is the dominant form of sludge stabilization in the U.S. In the anaerobic process, the organic material in the sludge is converted biologically to a variety of end products including methane, CH_4, and carbon dioxide, CO_2.

In **standard rate digesters**, the sludge is usually unheated and unmixed causing a stratification of the contents. Gas forms at the top. Scum forms above a layer of supernatant which forms above actively digesting sludge. Digested sludge thickened and stabilized forms at the bottom. The detention time in the digester varys from 30 to 60 days.

In **high-rate processes**, the digester contents are mixed and usually heated, gas forms above the sludge. Sometimes a second stage is added that performs in a stratified fashion similar to a standard rate digester. The detention time in a high rate digester is typically 15 days or less.

Final stabilization is caused primarily by the action of organisms known as **methanogenic bacteria**. These bacteria produce methane gas and carbon dioxide through some of the following reactions:

$$4H_2 + CO_2 \rightarrow CH_4 + 2H_2O$$
$$4HCOOH \rightarrow CH_4 + 3CO_2 + 2H_2O$$
$$CH_3COOH \rightarrow CH_4 + CO_2$$
$$4CH_3OH \rightarrow 3CH_4 + CO_2 + 2H_2O$$
$$4(CH_3)_3N + H_2O \rightarrow 9CH_4 + 3CO_2 + 6H_2O + 4NH_3$$

Anaerobic digesters are designed based on empirical loading rates or process microbiology. The mean cell residence time method is rooted in the microbiology of the process. The quantity of methane gas formed is calculated by:

$$V_{CH_4} = (5.62)[Q(S_0 - S)(8.34) - 1.42P_x]$$

Where:

V_{CH_4} = volume of methane produced, FT^3/d.
5.62 = theoretical conversion factor
Q = flowrate (MGD)
S_0 = ultimate BOD in influent, mg/L
S = ultimate BOD in effluent, mg/L
P_x = net mass of cell tissue produce per day, lb/d

The mass of solids synthesized can be estimated by

$$P_x = \frac{Y[Q(S_0 - S)8.34]}{1 + k_d\theta_c}$$

Where:

Y = yield coefficient, lb/lb
$k_{d=}$ endogenous coefficient, d^{-1}
θ_c = mean cell residence time, d

For a complete mix flow through reactor without recycle θ_c is equal to θ, the hydraulic detention time.

Loading Factor are another method for sizing digesters. A popular factor is the lb of volatile solids per ft^3 reactor volume per day. Standard Rate digesters are loaded from 0.03 to 0.10 $lb/ft^3 \cdot d$ volatile solids. High Rate digesters are loaded at from 0.10 to 0.30 $lb/ft^3 \cdot d$ volatile solids. Detention times, vary from 30 days to more than 90 days for standard rate digesters and from 10 to 20 days for High Rate digesters.

Aerobic Digestion

Aerobic digestion is used to stabilize sludge under aerobic conditions. The primary objective of aerobic stabilization is a reduction in volatile solids. Factors affecting design of aerobic digestion facilities include temperature, solids reduction, tank volume, oxygen requirements, mixing requirements and process operation. Solids reduction in aerobic digestion can be estimated by the first order reaction equation:

$$\frac{dm}{dt} = -k_d m$$

Where
$\frac{dm}{dt}$ = rate of change of biodegradable volatile solids per unit time.
k_d = reaction rate constant, d^{-1}.
m = concentration of volatile solids remaining at time t, mg/L.

The time t is equal to the solids residence time in the digester. Values for k_d may vary from 0.05 d^{-1} at 15°C to 0.14 d^{-1} at 25°C. Digester volume can be calculated from the following:

$$V = \frac{Q_i(X_i + YS_i)}{X(k_d P_v + 1/\theta_c)}$$

Where:
V = digester volume, FT^3
Q_i = average influent flowrate, FT^3/d
X_i = average influent suspended solids concentration, mg/L
Y = fraction of the influent BOD_5 consisting of raw primary sludge
S_i = influent BOD_5, mg/L
X = digester suspended solids, mg/L
k_d = reaction rate constant, d^{-1}
P_v = volatile fraction of digester suspended solids
θ_c = mean solids retention time, days

The term YS_i can be neglected if no primary sludge is included in the sludge load to the digester.

Example Problem No. 12

A wastewater treatment plant digests the sludge in high rate anaerobic digesters. The feed sludge is about 50,000 GPD at 3% solids and a BOD_5 of 23,000 mg/L. The digester has a volume of 750,000 gallons and operates with an efficiency of waste utilization of 60%. The kinetic coefficients are: Y=0.05 LB/LB and k_d=0.03 d^{-1}.

A. The mass of volatile solids synthesized by the digester in LB/d is most nearly:

a) 13,800 b) 600 c) 198 d) 50

B. The volume of methane produced by the digester in FT^3/d is most nearly:

a) 30,800 b) 198 c) 19,800 d) 25,200

Disinfection

Wastewater treatment plant effluent is disinfected to reduce pathogens before discharging. Disinfection inactivates pathogens. Strong oxidants such as chlorine can be used for disinfection as well as non-traditional disinfectants such as ultraviolet light.

Chlorine Disinfection

Chlorine is the most popular disinfectant for wastewater. Since chlorine is toxic to aquatic organisms, dechlorination is usually required. Chlorine is dosed sufficient to produce a minimum of a 0.5 mg/L free residual after 15 to 20 minutes of contact time minimum. This is usually sufficient disinfection to reduce fecal coliforms to less than 100 CFU/100mL.

Dechlorination is usually accomplished with Sulfur dioxide. Activated carbon can also be used. Sodium sulfite and sodium metabisulfite have also been used for dechlorination.

Chlorine concentrations in wastewater decay with time in contact with water. The following expressions can be used to describe the decay of chlorine used in disinfection of wastewater effluents:[iv]

$$C_c = 0.7C_0 + 0.3C_0 e^{-t} \text{ for } t < 1 \text{ min.}$$

$$C_c = 0.7C_0 e^{-0.003t} \text{ for } t > 1 \text{ min.}$$

Where:

C_C = residual chlorine concentration after t minutes, mg/L.

C_0 = chlorine dose, mg/L.
t = chlorine contact time, min.

Given the above relationships to predict chlorine residual, two methods are commonly used to estimate the microbial destruction of coliform and/or E. Coli using chlorine. Method 1 uses the two relationships below:[v]

$$\frac{N_t}{N_0} = (0.175C_0t + 0.75C_0te^{-t})^{-2.82} \text{ for } t < 1 \text{ min.}$$

$$\frac{N_t}{N_0} = (0.175C_0te^{-0.003t})^{-2.82} \text{ for } t > 1 \text{ min.}$$

Where:

N_0 = number of coliform organisms at t=0.
N_t = number of coliform organisms at time t.

Method 2 is derived from the Chick-Watson equation and may be used in the following form:[vi]

$$-\ln\frac{N_t}{N_0} = 2.86 \times 10^{-3}C_0^{1.46} + 14.4C_0^{1.25} - 14.5C_0^{1.25}e^{-0.00375t}$$

Generally, both methods are used and results compared when designing chlorine disinfection systems. The system must be designed to meet the permitted effluent criteria under all conditions.

Factors that affect the efficiency of Chlorine include temperature, pH, contact time, ammonia concentration, residual chlorine concentration and other characteristics of the wastewater. Chlorine is generally more effective at higher temperatures and lower pH. Longer contact times result in fewer coliform organisms. Ammonia combines with chlorine to form chloramines which are weaker disinfectants.

Example Problem No. 13

Effluent from a waste treatment process is disinfected with chlorine in a contact chamber. The hydraulic retention time of the contact chamber is 10 minutes. The effluent concentration of coliform organisms is 25,000 CFU/100mL prior to disinfection and the permit limit is 200 CFU/100mL.

A. The chlorine dose in mg/L computed using Method 1 required to achieve the required level of disinfection is most nearly:

a) 1.30 b) 2.21 c) 3.26 d) 4.27

B. The residual chlorine measured in mg/L after the contact chamber is most nearly:

a) 1.30 b) 2.21 c) 3.26 d) 4.27

Nitrification and Denitrification

Biological nitrogen conversion occurs in two-steps, nitrification and denitrification. Nitrification is the conversion of ammonia to nitrite and then to nitrate. Denitrification is the conversion of nitrate to nitrogen gas.

Nitrification

The following equations describe the chemical reactions involved in nitrification:

$$NH_4^+ + \frac{3}{2}O_2 \xrightarrow{Nitrosomonas} NO_2^- + 2H^+ + H_2O$$

$$NO_2^- + \frac{1}{2}O_2 \xrightarrow{Nitrobacter} NO_3^-$$

and the overall reaction is:

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

From the stoichiometry of the overall nitrification reaction it can be shown that 4.57 lb of oxygen are required for each lb of ammonia-nitrogen nitrified (64 g oxygen to 14 g of nitrogen). Also, nitrification of 1 mg/L of NH_4^+-N destroys about 7.1 mg/L of alkalinity as $CaCO_3$.

Nitrification requires aerobic conditions, generally a free dissolved oxygen content greater than 1.0 mg/L. Nitrification is achieved in the pH range of 7.2 to 9.0 and is more efficient at higher temperatures.

Design for nitrification requires consideration of the following factors:

- Dissolved Oxygen in the reactor must be greater than about 1.0 mg/L. Higher dissolved oxygen concentrations will increase the efficiency of nitrification.
- Operating pH should be between 7.2 and 9.0. A pH above or below this range will have a very low rate of nitrification if it can occur at all.
- Water temperature is a factor influencing the rate of nitrification. When temperatures are higher, 15° C or above the rate is faster while lower temperatures see lower rates of nitrification.

- Mean cell residence time has an impact on the nitrification. The nitrification rate is generally slower than the organic substrate utilization rate, thus a longer MCRT is required for nitrification.
- Oxygen required for nitrification is dependent upon the level of nitrification achieved and is given by:

$$O_2 = 4.57Q(N_0 - N)8.34$$

Where:

O_2 = Oxygen required for nitrification, LB/d.
Q = Flow treated, MGD.
N_0 = Influent ammonia-nitrogen concentration, mg/L.
N = Effluent ammonia-nitrogen concentration, mg/L.

This is the oxygen required for nitrification alone as in a separate stage nitrification process.

Nitrification processes are either single-stage or separate stage. A single-stage process achieves carbon reduction and nitrification within the same reactor. A separate stage process achieves nitrification in a reactor separate from the carbon reduction reactor as shown in Figure 9.

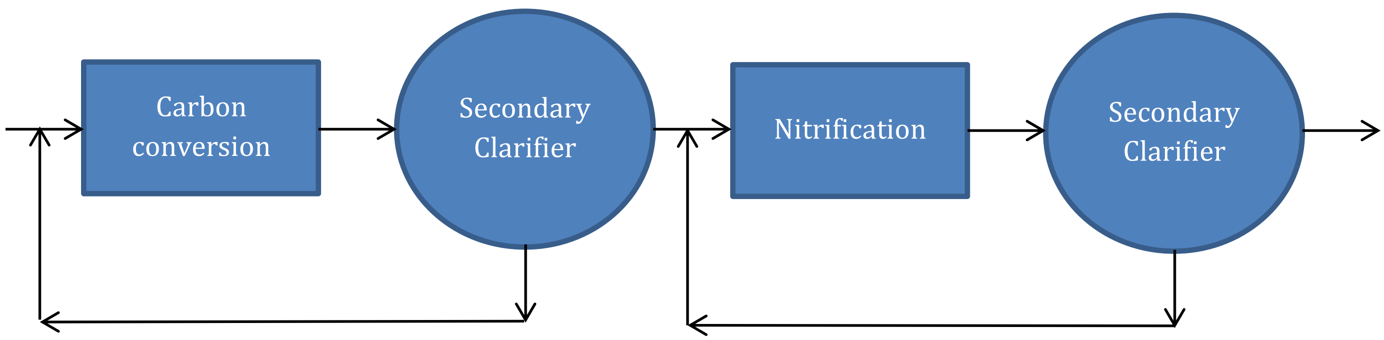

Figure 9 - Separate sludge nitrification

To achieve carbon reduction and nitrification in the same reactor the MCRT must be increased to allow for nitrification and the oxygen supplied must be increased to provide for nitrification. When nitrification is to be achieved in a single sludge system, the total oxygen required is given by:

$$O_2 = \frac{Q(S_0 - S_e)8.34}{f} - 1.42P_x + 4.57Q(N_0 - N)8.34$$

Where:

O_2 = Oxygen required for nitrification, LB/d.
Q = Flow treated, MGD.
S_0 = Influent BOD_5 concentration, mg/L.
S_e = Effluent BOD_5 concentration, mg/L.
P_x = Waste biomass, LB/d.

N_0 = Influent ammonia-nitrogen concentration, mg/L.
N = Effluent ammonia-nitrogen concentration, mg/L.

Denitrification

Denitrification is the conversion of nitrites to nitrogen and oxygen. The oxygen is metabolized by the biomass and the nitrogen, being relatively insoluble in water, is released to the atmosphere. Thus the combination of nitrification and denitrification results in removal of nitrogen from the wastewater.

Since Nitrate is a compound consisting of only nitrogen and oxygen, a facultative biomass will use the oxygen and leave the nitrogen:

$$organics + NO_3^- \rightarrow CO_2 + N_2 + energy$$

Thus, denitrification occurs in the absence of free dissolved oxygen. However, conditions do not need to be completely anaerobic, but merely anoxic, meaning that there is insufficient oxygen for all biomass which encourages the facultative bacteria to seek out the oxygen attached to nitrates and nitrites. The nitrogen molecules are then free to escape into solution and since nitrogen is not easily soluble in water, it escapes to the atmosphere.

Using methanol as a carbon source, denitrification can be described by the following:

$$6NO_3^- + 5CH_3OH \rightarrow 3N_2 + 5CO_2 + 7H_2O + 6OH^-$$

From this equation, we see that about half of the alkalinity destroyed by nitrification is returned by denitrification.

- The process is more efficient in warmer water temperatures
- Free dissolved oxygen less than 1.0 mg/L is typically required.
- A pH between 7 and 8 is considered optimal.
- The time required for denitrification depends upon the denitrification rate and the quantity of nitrate to convert. Denitrification rates generally range from 0.3 to 0.9 d^{-1} and endogenous decay rates vary from 0.04 to 0.08 d^{-1}. Thus, the minimum MCRT for denitrification can be given by:

$$\frac{1}{MCRT} = \mu_m - k_d$$

Where:

$MCRT$ = Mean Cell Residence Time in Denitrification Reactor, d.
μ_m = denitrification rate, d^{-1}.
k_d = endogenous decay rate, d^{-1}.

Systems can be single sludge or separate sludge. Either way, a carbon source is necessary for the required energy. Single sludge systems operate more slowly than separate sludge systems but have the advantage of being able to use organic material already in the wastewater as a carbon source.

Separate sludge systems require methanol addition or an equivalent carbon source. In single sludge systems, a portion of the oxygen demand required for carbonaceous oxidation is satisfied by the denitrification reaction, thus reducing oxygen requirements.

Example Problem No. 14

A wastewater treatment plant uses activated sludge for secondary treatment and needs to improve nitrification. The average flow is 4.0 MGD and the influent ammonia concentration is about 35mg/L as Nitrogen. The influent BOD_5 to the activated sludge process is about 200 mg/L. The organic conversion efficiency of the plant is 98.0%. The operators waste 2,650 LB/d of biomass from the activated sludge reactor.

A. If the ratio of BOD_5 to ultimate BOD is 0.60 and the process achieves complete nitrification of the influent ammonia the oxygen mass in LB/d required for both organic removal and nitrification is most nearly:

a) 12,500 b) 5,340 c) 7,140 d) 17,840

B. The minimum alkalinity in mg/L $CaCO_3$ needed in the influent to allow for nitrification is most nearly:

a) 150 b) 200 c) 250 d) 300

C. The denitrification rate is 0.6 d^{-1} and the endogenous decay rate is 0.06 d^{-1}. The MCRT in days required to denitrify this water is most nearly:

a) 1.85 b) 0.52 c) 2.56 d) 3.02

[i] "Water Quality and Technology-Based Permitting." *EPA.gov*. U.S. Environmental Protection Agency. Web. 03 Jan. 2012.
[ii] Ibid.
[iii] "Trickling Filter Systems." *Brentwood Technologies for Water and Wastewater Treatment*. Brentwood Technologies. Web. 02 Jan. 2012.
[iv] *Design Manual Municipal Wastewater Disinfection.* Cincinnati, OH: U.S. Environmental Protection Agency, Office of Research and Development, Water Engineering Research Laboratory, Center for Environmental Research Information, 1986. Print., p. 77-78.
[v] Ibid.
[vi] Ibid.

Wastewater Treatment

Formula Sheet

Wastewater flow rates

Average flows

$$Q_{Average} = \frac{Annual\ flow\ volume}{365\ days}$$

$$Q_{Per\ Capita} = \frac{Q_{Average}}{Population}$$

$$Q_{Design\ average} = Q_{Per\ Capita} \times Population_{Design\ Year}$$

Peak Flows

$$Q_{Peak} = Q_{Average} \times F_P$$

$$F_P = 1 + \frac{14}{4 + \sqrt{P}}$$

Where:

F_p = peaking factor or ratio of peak flow to average flow
P = population in thousands

Harmon's equation is just one equation commonly used to determine peaking factors. Other equations and charts can be used to determine peak flows and peaking factors.

Collection System Design flows

$$Q_{Design} = Q_{Peak} + Allowance_{I/I}$$

I and I Allowance:

$$Allowance_{I/I} = Area\ allowance \times Service\ Area$$

Or

$$Allowance_{I/I} = Pipe\ allowance \times \sum Pipe\ diameter \times Pipe\ length$$

Collection Systems

Lift stations

Wet Well design

$$V = \frac{T_{Min} q_P}{4}$$

Where:

V = volume of the wet well between pump on and pump off, gallons.
T_{Min} = Minimum cycle time, minutes.
q_P = the pump's average flow rate, gallons per minute.

Unit Operations and Processes

Typical Unit Process

A mass balance around a unit process will follow the laws of conservation of mass:

$$Influent\ Mass = Effluent\ Mass + Waste\ Mass$$

Likewise, the flows around a unit process will be conserved as well:

$$Influent\ Flow = Effluent\ Flow + Waste\ Flow$$

Removal Efficiency

$$E = \frac{S_{in} - S_{out}}{S_{in}} \times 100$$

Where:

E = removal efficiency of unit process in percent.
S_{in} = mass of contaminant entering unit process, Influent mass.
S_{out} = mass of contaminant leaving unit process, Effluent mass.

$$S_{in} - S_{out} = Waste\ Mass$$

Dissolved Air Flotation

$$A/S = \frac{1.3 s_a (fP - 1)}{S_c}$$

Where:

A/S = air to solids ratio, mL/mg
s_a = air solubility, mL/L
f = fraction of air dissolved at pressure P, usually 0.5
P = pressure, atm
p = gage pressure, lb/in^2
S_c = sludge solids, mg/L

Note that:

$$P = \frac{p + 14.7}{14.7}$$

And that:

Temp °C	0	10	20	30
s_a, mL/L	29.2	22.8	18.7	15.7

For systems that pressurize a recycle flow to cause flotation the following equation may be used:

$$A/S = \frac{1.3 s_a (fP - 1) R}{S_c Q}$$

Where:

R = pressurized recycle, MGD
Q = influent flow, MGD.

Adsorption

$$\frac{x}{m} = kC^{1/n}$$

Where:

x = the mass of compound adsorbed onto the carbon, lb.
m = the mass of activated carbon, lb.
C = the residual concentration of compound, mg/L.
k and n = isotherm constants determined experimentally for the specific compound and for the specific activated carbon.

For design values, a safety factor is included:

$$\left(\frac{x}{m}\right)_d = kC_0^{-1/n} * SF$$

Where:

$\left(\frac{x}{m}\right)_d$ = the design capacity of the GAC to retain the compound to be removed in lb of compound per lb of GAC.
C_0 = Influent Concentration, mg/L
SF = Safety factor for design, usually between 0.75 and 0.90.
k and n = isotherm constants.

Biological Treatment

Biochemical Oxygen Demand

Unseeded Test:

$$\mathrm{BOD}_t = \frac{D_i - D_t}{p}$$

Seeded Test:

$$\mathrm{BOD}_t = \frac{(D_i - D_t) - (B_i - B_t)f}{p}$$

Where:

BOD_t = the biochemical oxygen demand at time t, (mg/L)
D_i = the initial dissolved oxygen concentration, (mg/L)
D_t = the dissolved oxygen concentration at time t, (mg/L)
p = the decimal fraction of sample water in test
B_i = the initial dissolved oxygen concentration of the seed control, (mg/L)
B_t = the dissolved oxygen concentration of the seed control at time t, (mg/L)
f = the ratio of seed in sample to seed in control

$$\mathrm{BOD}_t = \mathrm{BOD}_u(1 - e^{-kt})$$

Where:

BOD_u = the ultimate BOD, (mg/L)
k = a decay constant determined experimentally, (d^{-1})
t = time, days

Activated Sludge

$$\theta = \frac{V}{Q10^6}$$

Where:

θ = Hydraulic retention time, days.
V = volume of the aeration tank, Gallons.
Q = influent wastewater flow, MGD.

$$OLR = \frac{QS_0 8.34}{V10^{-3}}$$

Where:

OLR = organic loading rate, lb/d/ 1,000 FT^3.
Q = influent wastewater flow, MGD.
S_0 = influent organic concentration, BOD_5, mg/L.
V = volume of the aeration tank, FT^3.

$$U = \frac{Q(S_0 - S_e)}{VX}$$

$$\theta_c = \frac{VX}{Q_w X_r + (Q - Q_w)X_e}$$

Where:

U = specific substrate utilization rate, d.
θ_c = mean cell residence time, d.
S_0 = Influent organic concentration, BOD_5, mg/L.
S_e = Effluent organic concentration, BOD_5, mg/L.
X = mixed liquor suspended solids (MLSS) concentration in the aeration tank, mg/L.
X_e = MLSS concentration in the effluent, mg/L.
X_r = Return Activated Sludge (RAS) concentration, mg/L.
Q = influent wastewater flow rate, MGD.
Q_w = Waste Activate Sludge (WAS) flow rate, MGD.
V = volume of the aeration tank, MG.

$$P_x = Q_w X_r 8.34 = \frac{YQ(S_0 - S_e)8.34}{1 + \theta_c k_d}$$

Where:

Px = mass of waste activated sludge(WAS), lb/d.
Y = maximum yield coefficient (microbial mass synthesized / mass of substrate utilized), lb/lb.
θ_c = mean cell residence time, d.
kd = endogenous decay rate, d-1.
S_0 = Influent organic concentration, BOD_5, mg/L.
S_e = Effluent organic concentration, BOD_5, mg/L.
Q = influent wastewater flow rate, MGD.
Q_w = Waste Activate Sludge (WAS) flow rate, MGD.
X_r = Return Activated Sludge (RAS) concentration, mg/L.

$$\frac{1}{\theta_c} = YU - k_d$$

$$F/M = \frac{QS_0}{XV} \approx U = \frac{Q(S_0 - S_e)}{XV}$$

$$O_2 = \frac{Q(S_0 - S_e)8.34}{f} - 1.42P_x$$

Where:

O2 = Oxygen required, LB/d.
Q = influent wastewater flow rate, MGD.
S_0 = Influent organic concentration, BOD_5, mg/L.
S_e = Effluent organic concentration, BOD_5, mg/L.
P_x = mass of waste activated sludge, LB/d.
f = factor to convert BOD5 to ultimate BOD, unitless (usually between 0.45 and 0.68).

Plug Flow Reactor

$$r_{su} = \frac{kS_e\bar{X}}{K_s + S_e}$$

Where:

r_{su} = rate of substrate (BOD_5) utilization, d^{-1}.

k = maximum specific substrate utilization rate per unit mass of microorganisms, d^{-1}.
S_e = effluent substrate (BOD_5) concentration, mg/L.
K_s = half-velocity constant, mg/L.
$\bar{X}$ = average biomass concentration in the reactor, mg/L.

$$\frac{1}{\theta_c} = \frac{Yk(S_0 - S_e)}{(S_0 - S_e) + (1+\alpha)K_S \ln(S_i/S_e)} - k_d$$

Where:

θ_c = mean cell retention time, d.
Y = yield coefficient, LB biomass/LB BOD_5.
k = maximum specific substrate utilization rate per unit mass of microorganisms, d^{-1}.
K_s = half-velocity constant, mg/L.
k_d = endogenous decay rate, d^{-1}.
S_0 = Influent organic concentration, BOD_5, mg/L.
S_e = Effluent organic concentration, BOD_5, mg/L.
α = recycle ratio = $\frac{Q_r}{Q}$.
S_i = influent concentration to reactor after dilution with return activated sludge (RAS) = $\frac{S_0+\alpha S_e}{1+\alpha}$.

Complete Mix Reactor

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

$$S_e = \frac{K_s(1+\theta_c k_d)}{\theta_c(Yk - k_d) - 1}$$

Where:

X = concentration of microorganisms (biomass)in the reactor, mg/L.
S_e = Effluent organic concentration, BOD_5, mg/L.
S_0 = Influent organic concentration, BOD_5, mg/L.
θ = hydraulic retention time in reactor, V/Q, d.
θ_c = mean cell retention time, d.
Y = yield coefficient, LB biomass/LB BOD_5.
k = maximum specific substrate utilization rate per unit mass of microorganisms, d^{-1}.
K_s = half-velocity constant, mg/L.
k_d = endogenous decay rate, d^{-1}.

Trickling Filters

NRC Equations

$$E = \frac{100}{1 + 0.0561\sqrt{\frac{W}{VF}}}$$

Where:

E = efficiency of BOD removal for process at 20°C, including recirculation and sedimentation, percent
W = BOD loading to filter, lb/d
V = volume of filter media, 10^3 ft^3
F = recirculation factor

$$F = \frac{1+R}{\left(1+{}^{R}/_{10}\right)^2}$$

Where:

R = recirculation ratio = Q_r/Q
Q_r = recirculation flowrate
Q = wastewater flowrate

The equation for the second stage filter is:

$$E_2 = \frac{100}{1 + \frac{0.0561}{1-E}\sqrt{\frac{W'}{VF}}}$$

Where:

E_2 = efficiency of BOD removal for second stage filter at 20°C, including recirculation and settling, percent
W' = BOD loading applied to second stage filter, lb/d

Adjustments to the efficiency based on wastewater temperature are given by:

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

Where:

E_T = Efficiency of BOD removal at temperature T, percent
E = efficiency of BOD removal at 20°C given by the NRC equations, percent
T = wastewater temperature, °C

Eckenfelder Equation

$$\ln\frac{S_e}{S_i} = -KS_a^m D(Q_V)^{-n}$$

Where:

S_e = BOD of settled effluent from filter, mg/L
S_i = BOD of wastewater applied to filter, mg/L
K = observed reaction rate constant for a given depth of filter, FT/d
S_a = specific surface area of filter = $\frac{unit\ surface\ area\ A_S,\ FT^2}{unit\ volume\ V,\ FT^3}$
D = filter depth, FT
Q_v = volumetric flowrate applied per unit area of filter, $FT^3/d{\cdot}FT^2$
m, n = empirical constants

Germain and Schultz Equation

$$\ln\frac{S_e}{S_i} = -k_{20}D(Q_V)^{-n}$$

Where:

k_{20} = treatability constant corresponding to a specific filter medium of depth D at 20°C, units vary with exponent n.
n = experimental constant, usually 0.5

$$k_T = k_{20}(1.035)^{T-20}$$

$$k_2 = k_1\left(\frac{D_1}{D_2}\right)^x$$

Where:

k_1 = treatability constant corresponding to a filter of depth, D_1.
k_2 = treatability constant corresponding to a filter of depth, D_2.
D_1 = depth of filter one, FT.
D_2 = depth of filter two, FT.
x = 0.5 for vertical plastic media or rock media filters
0.3 for cross flow plastic media filters

Digesters

Anaerobic Digestion

$$V_{CH_4} = (5.62)\left[Q(S_0 - S)(8.34) - 1.42P_x\right]$$

Where:

V_{CH_4} = volume of methane produced, FT^3/d.
5.62 = theoretical conversion factor
Q = flowrate (MGD)
S_0 = ultimate BOD in influent, mg/L
S = ultimate BOD in effluent, mg/L
P_x = net mass of cell tissue produce per day, lb/d

$$P_x = \frac{Y\left[Q(S_0 - S)8.34\right]}{1 + k_d\theta_c}$$

Where:

Y = yield coefficient, lb/lb
$k_{d=}$ endogenous coefficient, d^{-1}
θ_c = mean cell residence time, d

Aerobic Digestion

$$\frac{dm}{dt} = -k_d m$$

Where

$\frac{dm}{dt}$ = rate of change of biodegradable volatile solids per unit time.
k_d = reaction rate constant, d^{-1}.
m = concentration of volatile solids remaining at time t, mg/L.

$$V = \frac{Q_i(X_i + YS_i)}{X(k_d P_v + 1/\theta_c)}$$

Where:

V = digester volume, FT^3
Q_i = average influent flowrate, FT^3/d
X_i = average influent suspended solids concentration, mg/L
Y = fraction of the influent BOD_5 consisting of raw primary sludge
S_i = influent BOD_5, mg/L
X = digester suspended solids, mg/L
k_d = reaction rate constant, d^{-1}
P_v = volatile fraction of digester suspended solids
θ_c = mean solids retention time, days

The term YS_i can be neglected if no primary sludge is included in the sludge load to the digester.

Disinfection

Chlorine Disinfection

$$C_c = 0.7C_0 + 0.3C_0e^{-t} \text{ for } t < 1 \text{ min.}$$

$$C_c = 0.7C_0e^{-0.003t} \text{ for } t > 1 \text{ min.}$$

Where:

C_C = residual chlorine concentration after t minutes, mg/L.
C_0 = chlorine dose, mg/L.
t = chlorine contact time, min.

Method 1 uses the two relationships below:

$$\frac{N_t}{N_0} = (0.175C_0t + 0.75C_0te^{-t})^{-2.82} \text{ for } t < 1 \text{ min.}$$

$$\frac{N_t}{N_0} = (0.175C_0te^{-0.003t})^{-2.82} \text{ for } t > 1 \text{ min.}$$

Where:

N_0 = number of coliform organisms at t=0.
N_t = number of coliform organisms at time t.

Method 2 is derived from the Chick-Watson equation and may be used in the following form:

$$-\ln\frac{N_t}{N_0} = 2.86 \times 10^{-3}C_0^{1.46} + 14.4C_0^{1.25} - 14.5C_0^{1.25}e^{-0.00375t}$$

Nitrification and Denitrification

Nitrification

$$O_2 = 4.57Q(N_0 - N)8.34$$

Where:

O_2 = Oxygen required for nitrification, LB/d.
Q = Flow treated, MGD.
N_0 = Influent ammonia-nitrogen concentration, mg/L.
N = Effluent ammonia-nitrogen concentration, mg/L.

$$O_2 = \frac{Q(S_0 - S_e)8.34}{f} - 1.42P_x + 4.57Q(N_0 - N)8.34$$

Where:

O_2 = Oxygen required for nitrification and Carbon conversion, LB/d.
Q = Flow treated, MGD.
S_0 = Influent BOD_5 concentration, mg/L.
S_e = Effluent BOD_5 concentration, mg/L.
P_x = Waste biomass, LB/d.
N_0 = Influent ammonia-nitrogen concentration, mg/L.
N = Effluent ammonia-nitrogen concentration, mg/L.

Denitrification

$$\frac{1}{MCRT} = \mu_m - k_d$$

Where:

$MCRT$ = Mean Cell Residence Time in Denitrification Reactor, d.
μ_m = denitrification rate, d^{-1}.
k_d = endogenous decay rate, d^{-1}.