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High-Rate Biological Treatment of Wastewater at the Pantex Facility – Alternative to the Existing Aerate Lagoon-Pond System

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A Report on

High-Rate Biological Treatment of Wastewater at the Pantex Facility Alternative to the Existing Aerate Lagoon-Pond System

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Abstract

The purpose of this research is the development of design criteria and operating parameters for high-rate biological treatment of the wastewater generated at the Pantex Plant in Amarillo, Texas. The objective is biological treatment that offers an efficient and cost effective alternative for the existing Wastewater Treatment Facility (WWTF). The alternative system will alleviate the operation problems experienced by the existing biological WWTF at Pantex and eliminate the need to provide flood protection of the existing WWTF, which is in the 100-year flood plain.

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1. BACKGROUND

The constituents of the effluent of the existing WWTF at the Pantex Plant frequently exceed the permit limits established by the Texas Natural Resources Conservation Commission (TNRCC) (Permit No. 02296) and the US Environmental Protection Agency National Pollution Discharge Elimination System (NPDES), Permit TX0107107. Concentrations of TSS (total suspended solids), and NH_3 (ammonia nitrogen) are of primary concern; however, oil and grease (O&G), BOD_5 (5-day Biochemical Oxygen Demand), and COD (Chemical Oxygen Demand) also exceed the limits. The existing WWTF is within the 100-year flood plain established by the U.S. Corps of Engineers. A properly designed and operated aerobic biological treatment system, and an extended aeration activated sludge plant, will assure compliance with the effluent limits for TSS, BOD_5, and NH_3.

The maximum allowable discharge of treated effluent after chlorination and 30-minutes of contact is 600,000 gallons per day. The permit also limits the effluent to a maximum allowable COD of 300 mg/L and a maximum BOD_5 of 50 mg/L.

2. WASTEWATER TREATMENT FACILITY

The existing wastewater treatment facility includes screens that provide pretreatment of the influent prior to discharge into a pond system. Biological treatment is accomplished in a pond that is approximately 200 feet wide and 300 feet long with an average depth of approximately 15 feet. The total operating volume is approximately 6.5 million gallons. The pond is divided into two distinct zones, a mechanically aerated section and an unmixed zone. Two 15-hp high-speed aerators are installed in the influent portion of the pond. The aerated lagoon portion occupies about 60% of the total volume (approximately 3.9 million gallons). The remainder of the pond volume functions as a facultative stabilization pond. The pond effluent is chlorinated to maintain a chlorine residual between 1.0 and 4.0 mg/L after 30 minutes of contact. The chlorinated effluent is discharged into a dry creek that flows into a playa. Final disposal of the effluent is on the land; therefore, there is no discharge into surface waters of the State of Texas.

The WWTF is an aerated lagoon-polishing pond system. The first stage, the aerated lagoon, is designed to remove most of the influent BOD_5 and ammonia by aerobic biochemical conversion biodegradation to carbon dioxide (CO_2), water, and oxidized forms of nitrogen (nitrates, NO_3^-). The second stage consists of a quiescent portion of the pond that serves to polish the effluent of the aerated lagoon. In this portion of the pond the primary mechanisms include removal via sedimentation of the biological solids (biomass) produced as a result of the bioconversion of the influent BOD_5 and ammonia. Some additional removal of these BOD_5 by aerobic bacteria and conversion of nitrogen to algae cells also occurs. The solids that settle to the bottom of the second stage undergo anaerobic biodegradation.

An aerated lagoon-polishing pond system requires little operator attention other than assuring that the aerators are operating and maintaining the equipment. However, this type of treatment system is usually characterized by limited operational controls that are available to accommodate variations in the chemical composition of the influent and variations in the quality effluent. The organic load to the aerated lagoon-polishing pond system is low, typical of a dilute domestic wastewater.

The temperature of the water in the pond system usually increases during the period ranging from approximately April through October. During periods of high water temperature and when sunlight is available, algae grow in the second stage polishing pond portion of the basin because ample nutrients (nitrogen and phosphorus) are also available. The rapidly growing algae mass removes carbon dioxide and carbonates from the wastewater and generates oxygen in return by photosynthesis during daylight hours. The pH increases as a result of the removal of carbonates from the water column. However, at night, under conditions of no sunlight, the algae undergo respiration and use dissolved oxygen and generate carbon dioxide. Therefore, the dissolved oxygen concentration decreases and the pH in the pond decreases. In addition, algae convert a portion of the ammonia into algae cells that remain in the pond effluent. The algae are measured as total suspended solids (TSS) in the analytical procedure used to measure TSS. Algae can also contribute to BOD_5 concentration in the effluent, since the BOD_5 analytical procedure calls for incubation of the bottle in the dark. Algae respire and consume oxygen in the absence of sunlight. Aerobic bacteria that are present in the water column in the pond system use the oxygen that is generated by the algae as electron acceptors during the degradation of organic compounds (BOD_5), and for the biological oxidation of ammonia to nitrates; therefore, the hydraulic capacity of the treatment system is ample for the existing and projected future waste loading.

2.1 Wastewater Treatment Plant Performance

An overview of the performance of the WWTF based on the data presented in the Pantex Plant annual reports for 1996, 1994, and 1995 plus unpublished data for 1996 was prepared by Malina (1996). The graphical presentations of the BOD5, TSS, and ammonia influent and effluent data for the period May 1996 through April 1997 are consistent with the conclusions that the quality of the effluent of the treatment system usually is well within the limits set by the TNRCC Permit (Malina, 1996).

Data reported in 1993 and 1994 indicate that the maximum COD concentrations never exceeded 106 mg/L. The average concentrations of COD in the treated effluent were less than 50 mg/L for the samples collected during each of these years. No COD data was reported for 1995. The average BOD_5 removal achieved by the aerated lagoon-polishing pond system is good for this type of system.

The problems that the system has had with respect to permit compliance are for two regulated pollutants, TSS and ammonia. The performance of the WWTF was evaluated and the characteristics of the effluent chemical characteristics were analyzed in an attempt to determine the underlying cause(s) of concentrations in the effluent of the WWTF that exceeded the permit limits.

2.1.1 BOD_5

Effluent BOD_5concentrations generally are well within the effluent permit limits. However, during periods of time when algal growth and resulting effluent TSS concentrations were high, some high BOD_5 concentrations were also recorded (April 1997). This observation may be attributable to the analytical procedure used to measure BOD_5. Specifically, the procedure calls for incubating the BOD bottles in the dark in order to prevent interference by photosynthetic oxygen production by algae. However, in the absence of sunlight algae undergo respiration, utilize dissolved oxygen, and exert an oxygen demand. The oxygen demand of respiring algae cells is approximately 0.3 mg BOD_5 /mg TSS; therefore, 60 mg/L of algae as TSS can translate to a BOD_5 concentration of 18 mg/L.

The maximum effluent BOD_5 exceeded the maximum concentration of 50 mg/L allowed by the permit in 1993 and 1995. The data reported for 1994 and 1996 indicated that the effluent BOD_5 concentrations were below 50 mg/L.

The average monthly BOD_5 concentration of the chlorinated effluent in 1993 was 14.7 mg/L; however, the reported maximum concentration of BOD_5 was 94 mg/L. The monthly average effluent BOD_5 concentrations ranged from 4 mg/L to 29 mg/L in 1994 and the average monthly BOD_5 was 11.5 mg/L. During this time period, the average monthly TOC concentrations ranged from 8 mg/L to 22 mg/L.

The data reported for 1995 indicate that the average effluent BOD_5 concentration was 17 mg/L with a range of monthly BOD_5 concentrations from <2 mg/L to 63 mg/L. Effluent TOC concentrations ranged from 7 mg/L to 22 mg/L with an average of 11 mg/L.

The data for the first 6 1/2 months of 1996 (January through mid-June) indicate that the average monthly BOD_5 for the January through May 1996 time period were: 2.86 mg/L, 7.25 mg/L, 11.6 mg/L, 23.8 mg/L , and 29.2 mg/L, respectively. The effluent BOD_5 was less than 10 mg/L during the first three months except for one reported concentration of BOD_5, of approximately 40 mg/L. During the next two months (April and May) the reported BOD_5 concentrations exceeded 20 mg/L more frequently than the concentrations that were less than 20 mg/L.

The reason(s) for the increasing BOD_5 concentrations from January through May 1996 are not apparent. One explanation lies in the fact that as the water temperature increases, the rate of biodegradation increases, thereby increasing the rate of oxygen utilization. At the same time the concentration of dissolved oxygen in the water decreases at the higher temperature.

2.1.2 TSS (Algae)

The effluent concentrations of Total Suspended Solids (TSS) are relatively high. The reported maximum effluent TSS concentrations were 36 mg/L, 46 mg/L, and 42 mg/L, respectively for 1993, 1994, and 1995. The average concentrations of TSS in the chlorinated effluent were 22.3 mg/L, 22.31 mg/L and 18 mg/L, respectively for the samples collected in 1993, 1994, and 1995. During the time period January through May 1996 the average monthly TSS concentrations were 13.4 mg/L, 24.5 mg/L, 25.5 mg/L, 30.7 mg/L, and 37 mg/L.

This trend of increases in the concentrations of BOD_5 and TSS as the year progresses may reflect the tendency of algae growth to develop as the temperature and the period of sunlight increase in the portion of the pond that is not aerated. This phenomenon would provide some explanation for the increase in the effluent TSS concentrations.

One would expect that the concentrations of BOD_5, TSS, and TOC would follow a definite pattern, i.e., all three parameters would be high at the same time and increase and decrease proportionately. However, no such relationships seem to exist among the monthly average concentrations of these parameters. For example, the highest monthly average BOD_5 was 29 mg/L, and the average TSS that month was 36 mg/L and the TOC was 9 mg/L. However, when the average monthly TSS was highest at 40 mg/L, the average monthly BOD_5 was 12 mg/L, and the TOC was 22 mg/L. The discrepancies in these data can be attributable to the fact that 40% of the volume of the pond system was not aerated. The portion of the pond that is not aerated is quiescent and serves as a sedimentation basin in which the total suspended solids (biomass) (TSS) separate by gravity from the liquid effluent. Algae also grow is the quiescent portion of the pond system and may contribute to the higher concentrations of TSS.

The TSS problems that have been observed with high TSS concentrations in the effluent of the aerated lagoon-polishing pond system are attributable to the growth of algae in the quiescent areas of the pond, the polishing area. The effluent of the treatment system contains nitrogen and phosphorus concentrations that have been a concern to the operating personnel. Reported effluent ammonia concentrations in 1993 ranged from 1.4 mg/L to 6.5 mg/L. The average concentration of influent ammonia was 2.4 mg/L. Ammonia concentrations that were reported for the samples of effluent that were collected and analyzed in 1994 ranged from 0.3 mg/L to 5.6 mg/l of ammonia with an average effluent ammonia concentration of 2.69 mg/L. In 1995, effluent ammonia concentrations were not reported; effluent concentrations of nitrates and nitrites were reported instead. These oxidized forms of nitrogen could have more impact on ground water than ammonia.

The concentrations of nitrogen and phosphorus in the water column in the quiescent areas of the pond are similar to the concentrations of these nutrients in the influent domestic sewage and are sufficiently high to enhance the growth of algae.

Carbon that is required for algae growth is derived from the carbon dioxide and carbonates that are produced during aerobic bioconversion of BOD_5, and carbon dioxide that diffuses into the water from the air. Ample sunlight is available to energize the photosynthetic processes. In the winter when the pond temperatures are colder (growth rates are low when the temperature is less than 10ºC) the TSS concentrations are within the permit limits. Algae growth in the quiescent areas of the pond also is controlled somewhat by self-shading by algae cells in the summer (i.e., when algae growth is dense in the near-surface water layers and light penetration below the surface is reduced and algae growth is inhibited.

Problems with algae are not unique to the quiescent areas of the pond at the Pantex Plant. Significant amounts of algae growth occurs in the warm months of the year in almost all waste stabilization and polishing ponds that treat municipal and industrial wastewater. Generally the average effluent TSS concentration exceeds the limit of 30 mg/L (EPA, 1979; EPA, 1983). The USEPA and the Texas Natural Resource Conservation Commission (TNRCC) have established relaxed TSS limits for pond systems used by publicly owned treatment works (POTW) in order to accommodate these TSS excursions.

The TNRCC regulations provide specific effluent limits of a daily average of 90 mg TSS/L for effluents of oxidation ponds that treat domestic wastewater. There is no daily maximum TSS limit.

The effluent regulations for TSS concentrations in the effluent of stabilization pond treatment systems that treat domestic wastewater demonstrate the ubiquitous nature of algae in the pond effluents and the effects on meeting effluent TSS limits. The problems with algae in the effluent of industrial treatment systems are the same. However, industrial pond treatment systems have mass limits (kg/d or lb/d) on TSS that are based on the maximum monthly average effluent flow rate rather than on concentrations. Therefore, industry can meet effluent permit limits on TSS by storing water in the pond system thus regulating the quantity of effluent that is discharged.

The TSS concentrations in the effluents of many industrial stabilization pond facilities exceed established TSS limits during the warm months of the year (Gloyna, et.al., 1976; EPA, 1979). Expensive technologies have been installed to control algae (TSS) in the effluent.

2.1.3 Ammonia

The ammonia concentration in the untreated wastewater at the Pantex Plant generally ranges from about 1 mg/L to 20 mg/L, based on the 1996-1997 data. These ammonia concentrations are similar to those observed for dilute domestic wastewater. Influent ammonia concentrations as high as 48 mg/L have been reported in three grab samples of effluent during the period from May 1996 through April 1997. These influent ammonia concentrations should not have any significant effect on the performance of the wastewater treatment facilities.

Effluent ammonia concentrations are less variable. Effluent ammonia concentrations ranged between 2 mg/L and 10 mg/L during the May 1996 through April 1997 period. The average monthly ammonia concentration in the effluent ranged from 2 mg/L to 6 mg/L during 1995and during June and July 1996 (Malina, 1996).

Mechanisms of ammonia removal in aerated lagoon systems include:

(a) Incorporation into bacterial and algae cell mass;
(b) Biological nitrification, in which ammonia is oxidized into nitrates by aerobic bacteria (***Nitrosomonas*** and ***Nitrobacter***);
(c) Volatilization of ammonia to the air at high pH in the water.

The volatilization pathway is insignificant since the pH in the water column in the aerated lagoon system is near neutral.

The nitrogen that is incorporated in the cell biomass as protein either leaves the system as suspended solids that settle to the bottom of the pond or are carried in the effluent. The biomass (TSS) that settles to the bottom of the pond undergoes anaerobic biodegradation and a portion of the nitrogen that was incorporated in the biomass will be released back to the water column.

The bacteria that are responsible for nitrification are strict aerobes; i.e., bio-oxidation of ammonia occurs only when the water column contains free dissolved oxygen. Most of the oxidation of ammonia to nitrates occurs in the mechanically aerated section (aerated lagoon) of the wastewater treatment facility. In the aerated lagoon section, dissolved oxygen is available in relatively high concentrations and the biomass and the dissolved oxygen are thoroughly mixed in the water column.

The increase in effluent ammonia concentrations noted in June and July 1996 apparently is the result of release of ammonia from the biomass in the sediments in the pond. The influent ammonia loading was low during and preceding this event.

The effluent nitrite/nitrate concentrations are indicative of biological nitrification of ammonia. Although data on the effluent nitrite/nitrate concentrations are relatively limited, the 1995 and 1996 data demonstrate that nitrification of ammonia is an important removal mechanism, especially in the warmer months (Malina, 1996). Nitrate concentrations in the treated effluent ranged from 0.05 mg/L to 7.08 mg/L, in 1995. The average nitrate concentration in the effluent samples collected was 1.91 mg/L. The reported effluent nitrite concentrations ranged from 0.11 to 0.98 mg/L and the average nitrate concentration reported was 0.54 mg/L.

Other data also show nitrate/nitrite concentrations that are consistent with successful nitrification in the summer months. Low nitrate/nitrite concentrations in the winter months are consistent with reduced nitrification in cold temperatures (Texas Tech, 1996). Nitrifying bacteria are more sensitive to cold temperatures than the aerobic bacteria responsible for the bio-conversion of BOD_5; therefore, the rate and extent of nitrification in the winter decreases and possibly no nitrification would be observed under extremely low temperatures in the water column. During winter conditions, the primary mechanism of ammonia removal is incorporation of nitrogen into biomass during cell synthesis. The aerobic bacteria responsible for the biodegradation of BOD_5 also require nitrogen for cell synthesis.

Consistently removal of ammonia during winter months is a problem that is intrinsic to pond systems used to treat municipal and/or industrial wastewater (EPA, 1979; Gloyna, et. al., 1976). Biological activity including bacterial growth and algae growth slows at cold temperatures; therefore, biological removal of ammonia in aerated lagoon systems is a challenging problem in winter months.

In 1993, the pH recorded for the wastewater treatment facility was within the permit limits specified by the TNRCC of pH = 6.0 and pH = 9.0, except for 16 excursions, during which the pH increased to between pH 9.1 to pH = 9.3. During most of the excursions (12 of 16) the pH was pH = 9.1. On the other occasions the pH reached pH = 9.2 three (3) times and on in one instance the pH was pH = 9.3. However, on each occasion remedial action brought the pH to within the permit limits. A sulfuric acid injection system was installed in September 1993 and after that date the pH reached pH = 9.3 on one occasion when the supply of sulfuric acid was depleted.

3. CAPACITY OF THE EXISTING WASTEWATER TREATMENT SYSTEM

The hydraulic detention time in the pond treatment system based on a flow rate of 380,000 gallons per day and a total volume of 6.4 million gallons is 16.8 days. Therefore, there is sufficient hydraulic capacity to continue to operate this pond system as an aerated lagoon. Typical hydraulic detention times for aerated lagoons used for the treatment of industrial wastewater are two to eight days. Therefore, the existing pond can continue to be used for treatment of an increased volumetric flow rate of wastewater. A portion of the volume of the pond could also be reserved without aeration to allow separation of biological solids from the effluent before chlorination. This conversion would require the installation of some caisson walls or a circular caisson that could serve as the solids/liquid separation unit. A small vacuum pump and a swimming pool floor vacuum device could be used to return the bio-solids from the separator to the aerated portion of the basin.

Ample hydraulic detention time is available to accommodate the increased organic loading and to biologically treat this waste stream. It is understood that this waste stream will contain no organic solvents but will contain nitrogen.

3.1 Alternative Treatment

The monthly average BOD_5, TSS, and ammonia limits in the TNRCC and NPDES permits issued for the wastewater treatment facilities at the Pantex Plant are based on an activated sludge treatment system. Money will have to be spent to move the existing equipment above the 100-year flood plain; therefore, installation of a completely new activated sludge package plant is an alternative.

An activated sludge plant provides a very high level of assurance that the permit limits will be complied with, if it is designed and operated correctly. The activated sludge unit would be constructed as a tank system, probably above ground. The system would be subject to the RCRA hazardous waste permit exemption for NPDES treatment systems. This means that if a hazardous waste is discharged in the wastewater to such a unit, either accidentally or for planned treatment (with proper notification), then the units are exempt from RCRA permitting, although registration as a treatment, storage, or disposal facility is still required. A wastewater system that uses surface impoundment for treatment must not only be permitted, it must also meet the minimum technology requirements and land disposal restrictions, which include use of a double-liner and removal of any accumulated solids on an annual basis. Although Pantex does not treat hazardous wastes in the existing WWTF, and may not plan to ever do so, an accidental spill of hazardous waste that enters the existing treatment system would raise a number of regulatory problems. If a tank system were used for wastewater treatment, regulatory exposure from this type of incident would be minimal.

The type of activated sludge package plant that would be used for Pantex would have an integral influent structure, aeration tank, clarifier, aerobic sludge digester, and chlorinator. It would be of the extended aeration type (approximately 24-hour retention time) to assure consistent nitrification of ammonia. The activated sludge could be digested aerobically prior to concentration by a belt filter press. The filter cake could be disposed of in a landfill. Alternately, the aerobically digested sludge could be thickened and applied to the land as a soil amendment. Approval of land application of sludge would have to be obtained from TNRCC. However, based on the solids analyses performed by Texas Tech, approval of land disposal should not be a problem.

4. SUMMARY OF PERFORMANCE DATA

The TSS compliance problem that is experienced at the Pantex Plant WWTF is a common problem for municipal and industrial aerated lagoon/ waste stabilization pond/ polishing pond systems. Algae grow to concentrations that exceed the effluent TSS concentration of 30 mg/L that was established by TNRCC and USEPA. Nitrogen and phosphorus are in sufficient supply in the domestic wastewater that makes up a significant amount of the Pantex Plant wastewater. In addition, sunlight and long hydraulic retention time support algae growth. Therefore, TNRCC and USEPA have developed special TSS provisions that allow the discharge of higher average TSS for municipal pond systems concentrations. Pantex Plant is an industrial facility; therefore, the higher average TSS limits were not incorporated in the TNRCC and NPDES effluent discharge permits.

The monthly average ammonia limit of 5 mg/L is based on activated sludge plants with biological nitrification. The WWTF is able to achieve this limit most of the time. However, apparent excursions of higher effluent concentrations usually are associated with cold weather. Nitrification is inhibited at low temperatures. Other incidence of high effluent concentrations possibly may be attributable to a spring or fall increase that may have been associated with a rapid release of ammonia from accumulated sediments. The existing WWTF should be capable of achieving the established ammonia limit since the ammonia loading on the WWTF is low, and ensuring sufficient oxygen is supplied to the system. Also, the first stage (aerated) should be maintained in a well-mixed condition. Other alternatives may have to be explored to achieve the ammonia limits if cold temperatures cause inhibition of nitrification reactions.

A pre-manufactured activated sludge plant is an alternative process. This facility could be located outside of the 100-year floodplain. A well-designed and properly operated activated sludge plant will assure meeting the TSS and ammonia limits.

4.1 Biodegradation of Organic Materials in Wastewater

The objective of biological treatment is the conversion of biodegradable soluble organic compounds to innocuous end-products that may be discharged into a receiving course without risk to human health and the environment. Biological wastewater treatment includes the enhanced stabilization of dissolved organic compounds and incorporation of non-settleable colloidal solids into a zoogleal biomass (sludge). The incorporation of colloidal material into the biomass is a type of biological coagulation.

A variety of microorganisms, primarily bacteria with some fungi, molds, yeasts, and protozoa are responsible for the biotransformation of organic material. The microbial consortium degrades and uses the organic material to meet requirements for energy, bacterial cell maintenance, and cell synthesis. The end-products are carbon dioxide, water and new cell biomass.

Biological degradation of biodegradable organic materials by heterotrophic bacteria is illustrated schematically by the diagram presented Figure 1. Heterotrophic bacteria derive energy and carbon from the organic substrate that is oxidized in aerobic systems.

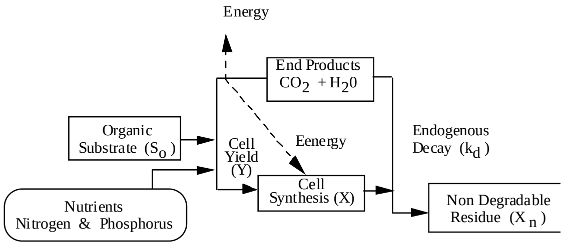

Figure 1: Chemoheterotrophic Microbial Metabolism

4.2 Bacteria Composition, Cell Synthesis and Biotransformation of Organic Material

Microbial growth requirements are: energy, carbon, inorganic macronutrients (nitrogen, and phosphorus), inorganic micronutrients (principal nutrients including sulfur, potassium, calcium, magnesium, iron, sodium, and chlorides; and minor nutrients such as zinc, manganese, molybdenum, selenium, cobalt, copper, nickel, vanadium and tungsten), and organic growth factors (vitamins, amino acids, purines, pyrimidines).

The elemental composition of cells is $C_{60}H_{87}O_{23}N_{12}P$. This formulation indicates that nitrogen makes up about 12% to 13% of the dry weight of the biomass and the phosphorus content is approximately 2% to 3% of the dry weight of the biomass. A simplified formulation of the cell composition frequently is used, $C_5H_7O_2N$.

Aerobic decomposition of an organic substrate and bacteria growth can be defined by the following equations. Glucose is used to represent the substrate.

Substrate oxidation:

$$C_6H_{12}O_6 + O_2 \rightarrow 6\ CO_2 + H_2O \qquad \textbf{(1)}$$

Cell synthesis:

$$20\ C_6H_{12}O_6 + 24\ NH_4^+ + 2\ H_2PO_4^- + O_2 + \text{Bacteria} \rightarrow C_{60}H_{87}O_{23}N_{12}P + 83\ H_2O + \text{Residual Non Degradable Organic Compounds} \qquad \textbf{(2)}$$

The endogenous decay or auto oxidation of biomass can be rewritten using the simpler representation of the cell composition ($C_5H_7O_2N$).

$$C_5H_7O_2N + 5\ O_2 \rightarrow 5\ CO_2 + NH_4^+ + H_2PO_4^- + 2\ H_2O + \text{Non Degradable Residue} \qquad \textbf{(3)}$$

Equation 3 indicates that the theoretical ultimate oxygen required to oxidize the degradable fraction of the biomass is 1.42 g O_2 /g biomass. Therefore, the ultimate BOD of the biomass is 1.42 times the concentration of biomass.

Three major pathways must be considered in high-rate biological systems:

(a) Biochemical oxidation (biotic),
(b) Sorption on the biomass (abiotic),
(c) Volatilization (abiotic).

The aerobic biodegradation of organic chemicals (substrate) is affected by:

(a) Type and concentration of the substrate (organic compound(s)),
(b) Concentration of acclimated organisms,
(c) Availability of dissolved oxygen,
(d) Availability of nutrients (N and P),
(e) Temperature,
(f) pH and,
(g) Toxicity constiutents.

4.3 Activated Sludge Process

For the past 40 years, the activated sludge process has been the most widely used wastewater treatment system in the United States. This system is applicable for the treatment of a wide range of municipal and industrial wastewater that contains biodegradable organic material.

The activated sludge process includes an aeration basin in which a biological floc (biomass) is mixed with the wastewater containing organic constituents, and oxygen to enhance the biochemical oxidation of the organic material. Colloidal material in the influent is incorporated in and soluble BOD is oxidized by the biomass floc particle. The mixture of biological floc and wastewater commonly is called mixed liquor suspended solids (MLSS). The reactor combined with a secondary clarifier constitutes the conventional activated sludge process.

The suspended solids (biomass) that are generated in the reactor are separated by gravity and concentrated in the clarifier. A portion of the concentrated biosolids (underflow from the clarifier) is recycled to the reactor and the remainder is discharged from the system.

The Extended Aeration modification of the activated sludge system incorporates an long hydraulic retention time (24 hours) and a high sludge age that minimizes the net sludge production.

The removal of biodegradable organic compounds is a function of the rates of degradation of organic, biomass concentration (MLSS), sludge age (solids retention time), and hydraulic retention time. Therefore, process variables include:

(a) Type and concentration of organic compounds;
(b) Influent suspended solids;
(c) Nutrient availability (N and P);
(d) Sludge age;
(e) Aeration time(hydraulic retention time;
(f) Acclimated bacterial consortium (MLSS);
(g) Presence or absence of biologically toxic constituents (e.g. heavy metals, toxic organic compounds, etc.);
(h) Potential inhibitors, (e.g. total dissolved solids (TDS), chlorides, etc.);
(i) pH;
(j) Alkalinity, and;
(k) Acidity.

The activated sludge process or some variation has been used extensively to treat municipal wastewater. The design of the activated-sludge process must consider:

(a) Effluent characteristics required,
(b) Reactor type, i.e. conventional plug flow, completely stirred, contact stabilization, extended aeration, etc.,
(c) Organic loading criteria,
(d) Sludge production,
(e) Oxygen requirements and transfer,
(f) Nutrient requirements,
(g) Solids liquid separation, and
(h) Control of filamentous organisms.

Kinetic parameters required for the design of aerobic biological treatment systems may be developed after establishing the biodegradability of the constituents in the wastewater. Bench-scale or pilot-scale treatability studies will yield the necessary data required to evaluate kinetic and design parameters.

The hydraulic detention time and the biological cell residence time (sludge age) are two important design variables that affect the performance of biological treatment systems. The sludge age reflects the average time that the biomass is in the system while the hydraulic detention time indicates the time that the biomass is in contact with the constituents of the wastewater before the wastewater is discharged from the reactor. Important factors that affect the performance of the activated sludge process include:

(a) Type and concentration of organic constituents;
(b) Mixing intensity in the biological reactor;
(c) Air flow rate and oxygen transfer in aerobic systems;
(d) Geometry of the reactor;
(e) Inventory of active biomass, and;
(f) Microbial growth pattern

A schematic flow diagram of the activated sludge process is illustrated in Figure 2.

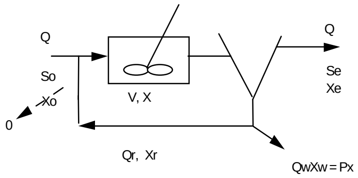

Figure 2: Activated Sludge Process

The terms included in Figure 2 are defined below.

Q = Flow rate of wastewater, (L/d)
S_o = Influent substrate (BOD_5) concentration, (mg/L)
S_e = Effluent substrate (BOD_5) concentration, (mg/L)
X = Biomass concentration in the reactor (MLSS) (mg/L)
X_o = Biomass concentration in the influent (MLSS) (mg/L)
X_e = Biomass concentration in the effluent (MLSS) (mg/L)
V = Volume of the reactor (L)
Q_w = flow rate of excess activated sludge (L/d)
Q_r = rate of flow of recycled activated sludge (L/d)
X_w = concentration of excess activated sludge (MLSS) (mg/L)
X_r = concentration of activated sludge (MLSS) after settling in the final clarifier (mg/L)
X_w = X_r
P_x = biomass produced daily (kg/d)

The design relationship that describes the aerobic biodegradation phenomenon in wastewater treatment in the activated sludge process may be written as:

$$\frac{1}{\theta_c} = YU - k_d \qquad \textbf{(4)}$$

where:

θ_c = cell residence time, (days)
Y = Cell Yield Coefficient, (mg MLVSS / mg substrate removed)
k_d = Endogenous Decay rate, (d^{-1})
U = Specific Substrate Utilization rate, (mg BOD5 removed per mg MLVSS-d)

but:

$$U = Y\frac{Q(S_o - S)}{VX} \qquad \textbf{(5)}$$

where:

Q = flow rate (L/d);
S_o = influent BOD or COD (mg/L);
S = effluent BOD or COD (mg/L)
V = reactor volume (L); and
X = MLVSS (mg/L).

The cell residence time is also called solids retention time, or sludge age, and may be defined as:

$$\theta_c = \frac{\text{biomass in the reactor}}{\text{biomass produced per day}} \qquad \textbf{(6)}$$

$$\theta_c = \frac{VX}{Q_w X_w + Q X_e} \text{ (days)}$$

However, QX_e can be considered to be negligible, and:

$$\theta_c = \frac{VX}{Q_w X_w} \text{ (d)} \qquad \textbf{(7)}$$

Therefore, the reciprocal is:

$$\frac{1}{\theta_c} = \frac{QX}{VX} = \frac{\text{biomass produced per day}}{\text{biomass in the reactor}} (d^{-1})$$

$$Y = \text{cell yield coefficent} \left(\frac{\text{kg MLSS}}{\text{kg BOD}_5\text{ - d}}\right)$$

U= specific substrate utilization rate

$$U = \frac{Q(S_o - S_e)}{VX} \left(\frac{\text{kg BOD5 removed}}{\text{kg MLSS - d}}\right) \text{ or}$$

$$U = \frac{(S_o - S_e)}{X\theta_h} \qquad \textbf{(8)}$$

where:

θ_h = hydraulic detention time (d)

$$\theta_h = \frac{V}{Q} \;.$$

The kinetic coefficients Y and k_d can be evaluated graphically using a modification of the Equation 8.

A plot of $\frac{1}{\theta_c}$ vs U or $\frac{S_o - S}{\theta X}$ (see Figure 3).

results in a straight line. The slope = Y and the intercept at U = 0 is k_d.

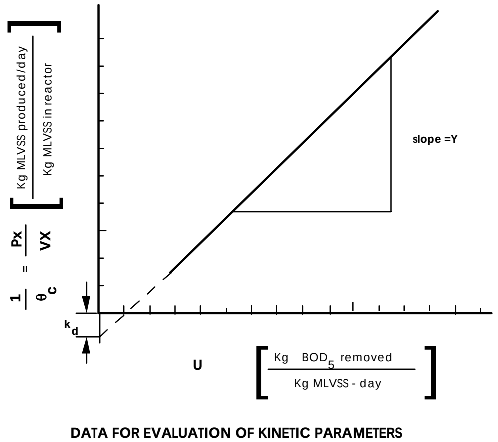

Figure 3: Graphical Determination of Kinetic Coefficients

The numerical value of the cell yield coefficient (Y) and the decay rate (k_d) will be different depending on the parameter used to define substrate, viz. BOD, COD, or TOC and the concentration of biomass used, i.e. MLSS or MLVSS.

4.4 Operational Considerations

4.4.1 Sludge Production

The biomass produced in the aeration basin must be wasted from the system in order to maintain steady state conditions. The amount of biomass (sludge) produced can be calculated using an equation that results from rearranging the terms in the design equation:

$$\frac{1}{\theta_c} = \frac{P_x}{VX} = Y\frac{Q\,(S_o - S)}{VX} - k_d \qquad \textbf{(9)}$$

and,

$$P_x = Y\,Q(S_o\text{-}S) - k_dVX \qquad \textbf{(10)}$$

where:

P_x = amount of biomass prodcued daily (kg/d]

$Q(S_o\text{-}S)$ = amount of substrate removed (kg BOD_5/d) and

VX = amount of biomass in the reactor (kg biomass).

The excess biomass also may be calculated from the design sludge age:

$$\frac{1}{\theta_c} = \frac{P_x}{VX}. \qquad \textbf{(11)}$$

Therefore,

$$P_x = \frac{VX}{\theta_c} \qquad \textbf{(12)}$$

4.5 Oxygen Requirements

The aerobic environment is maintained in the reactor by introducing air through diffusers or by mechanical aeration. The biomass is also kept in suspension by the mechanical or diffuse aeration system. The amount of aeration required is based on the oxygen uptake requirement and the energy required for mixing. The theoretical oxygen requirement can be calculated from the ultimate BOD removed in the process less the theoretical ultimate BOD of the biomass wasted from the reactor per day. Therefore, the theoretical oxygen required is:

$$\text{O.U.R.} = \frac{Q(S_o - S_e)}{f} - 1.42\,P_x \qquad \textbf{(13)}$$

where:

O.U.R. = oxygen utilization rate (kg O_2/d)

f = conversion of BOD_5 to $BOD_{ultimate}$ (usually 0.68)

1.42 = conversion of biomass to $BOD_{ultimate}$ (see page 9)

$Q(S_o\text{-}S)$ =amount of substrate removed (kg BOD_5/d)

P_x = biomass produced (wasted) (kg/d)

When nitrification occurs in the aeration basin, the total oxygen requirement includes the oxygen required for removal of carbonaceous BOD plus the oxygen required for conversion of ammonia-N to the nitrate form (see page 19, Section 5). The equation can be modified to include the oxygen required for nitrification:

$$\text{O.U.R.} = \frac{Q(S_o - S_e)}{f} - 1.42\ P_x + 4.57\ Q(N_o\text{-}N) \qquad \textbf{(14)}$$

where:

4.57 = oxygen required for complete nitrification (kg O2/kg TKN)

$Q(N_o\text{-}N)$ = is the amount of nitrogen oxidized (kg TKN_5/d).

If oxygen uptake rates are measured in the aeration basin the oxygen uptake rate in pounds per day can be calculated using the following equation:

$$\text{O.U.R.} = a'\ Q(S_o - S_e) + b'\ VX \qquad \textbf{(15)}$$

A plot of the specific oxygen uptake rate (kg O_2 utilized per kg of biomass in the reactor) versus the specific substrate utilization rate (U) yields a straight line and

a’ = slope of the line

b’ = intercept of the line when U =0.

Good mixing in the aeration basin is a function of the airflow rate, aeration device or energy input from mechanical aerators as well as the surface area, depth and shape. Airflow rates for diffused aeration systems for a spiral roll pattern are 20 to 30 ft^3/min -1000 ft^3 of tank volume (20 to 30 m^3/min-1000 m^3). When the diffused aeration devices are installed in a grid pattern the air flow rates are 10 to 15 ft^3/min - 1000 ft^3 of tank volume (10 to 15 m^3/min-1000 m^3). Typical energy requirements for mixing activated sludge with mechanical aeration devices range from 0.75 to 1.50 hp/ 1000 ft^3 (19 to 19 kW/ 1000 m^3).

4.5.1 Sludge Settleability

Separation of the biomass takes place in the secondary clarifier resulting in a clear effluent. In addition the separated biomass concentrates so that the concentration of the recycled sludge is much higher than the MLSS entering the clarifier (usually between 1 and 2% solids on a dry weight basis).

If the biomass is not removed from the effluent stream, treatment is not accomplished since biosolids exert an additional oxygen demand on the receiving stream. Therefore, effective removal or biosolids is essential. The secondary clarifiers also provide another opportunity to remove floatable solids before discharge.

One parameter that commonly is used to assess the settleability of the solids and the ability to concnetrate the solids is the Sludge Volume Index (SVI). SVI is the volume occupied [mL] by one gram of MLSS (dry weight) after settling for 30 minutes in a one-liter graduate cylinder and has units of (mL/g).

$$\text{SVI} = \frac{\text{volume occupied (mL)}}{\text{mass of solids (g) (dry wt.)}} \qquad \textbf{(16)}$$

The rate at which the sludge/water interface moves through the depth of the graduated cylinder can be used to calculate the “zone settling velocity” (ZSV). The ZSV can be used to establish the solids loading to the clarifier and to establish the design overflow rate. The design of secondary clarifiers is based on an overflow rate of 16 to 32 m^3/m^2 - day at design flow rates and 40 - 50 m^3/m^2 - day for peak flow conditions.

4.5.2 Temperature Effects

Temperature affects the degradation rate. The effects of temperature can be expressed as a simplified form of the Arrhenius equation:

$$\frac{k_1}{k_2} = \theta^{(T_2 - T_1)} \quad \textbf{(17)}$$

where:

T_1, T_2 = liquid temperature (ºC)
k_1, k_2 = substrate removal rate (d^{-1})
θ = temperature coefficient (usually 1.02 to 1.06).

The liquid temperature also can have a significant effect on the substrate removal rate and subsequent organic removal efficiency. In general, the rate of reaction increases as the temperature increases. Temperature has a dramatic effect on the nitrification process.

5. BIOCHEMICAL NITRIFICATION

The oxidation of ammonia to nitrates is called nitrification. Bio-oxidation of ammonia results in the formation of nitrites that subsequently are oxidized to nitrates.

__Nitrosomonas__ bacteria oxidize ammonia to nitrites. The reaction is:

$$NH_3 + 1.5\ O_2 \rightarrow HNO_2 + H_2O \qquad \textbf{(18)}$$

Nitrites are oxidized by ***__Nitrobacter__*** bacteria to nitrates. The reaction is:

$$HNO_2 + 0.5\ O_2 \rightarrow HNO_3 \qquad \textbf{(19)}$$

The overall nitrification reaction is:

$$NH_3 + 2\ O_2 \rightarrow HNO_3 + H_2O \qquad \textbf{(20)}$$

The nitrifying organisms (***__Nitrosomonas__*** and ***__Nitrobacter__***) are much more sensitive to environmental conditions than the heterotrophic (carbon oxidizing) bacteria. The nitrifying bacteria are slow growers that are more sensitive to temperature changes, and more vulnerable to bio-toxicants that may be introduced into the system. The process variables for effective nitrification include but are not limited to:

(a) Sludge age;
(b) Hydraulic retention time;
(c) Temperature;
(d) Total and free ammonia concentration of the feed stream;
(e) Concentration of dissolved oxygen in the aeration basin;
(f) Mass of nitrifying bacteria in the reactor;
(g) Presence or absence of bio-toxicants.

The oxygen required to oxidize ammonia to nitrates theoretically is:

3.765g O_2 per g NH_3 oxidized to HNO_3^-

OR

4.57 g O_2 per g N oxidized based on ammonia-nitrogen (NH_3-N).

5.1 Objectives

The objective of this research is the development of design criteria and operating parameters for a high-rate biological treatment that would provide an alternative to the existing wastewater treatment facility (WWTF). The performance of the extended aeration activated sludge process was evaluated at different cell residence times.

5.2 Experimental Procedures

Three bench-scale aerobic biological treatment systems were operated at various organic and hydraulic loading to establish design criteria and operating parameters that will maximize treatment performance.

Three bench-scale extended aeration activated sludge reactors, each with a total volume of approximately 10 liters were operated at different food to microorganism ratios (F/M), solids residence times (sludge age), and hydraulic residence time. The sludge age and the hydraulic detention times are the principal variables. The performance of the systems will be evaluated at times of four, 8, and 12 days. The F/M and the hydraulic residence times are interdependent and the hydraulic residence time will be varied at each sludge age. Influent and effluent concentrations of COD, BOD_5, and TSS, were used to establish treatment efficiency.

The start of the project was delayed. The cause of the delay was difficulty in obtaining approval from the Department of Defense to collect samples of wastewater from the Pantex treatment facility in Amarillo, Texas.

Untreated wastewater was collected from the influent to the existing wastewater treatment facility at the Pantex Plant in Amarillo, TX on May 20, 1998. Eighteen metal 55-gallon drums were filled with wastewater and three (3) five-gallon containers were filled with the biomass from the aerated portion of the treament system. These samples were transported to The University of Texas at Austin and stored at a temperature of 4ºC, a constant temperature environmental chamber in the laboratories of the Environmental and Water Resources Engineering Program on the main campus. The untreated wastewater was transported as needed to the Center for Research in Water Resources characterized, and fed to the bioreactors.

The concentration of biomass in the samples withdrawn from the aerated lagoon at the Pantex Plant was low (less than 100 mg/L). Therefore, activated sludge that contained a microbial consortium was collected at the South Austin Regional Wastewater Treatment Facility in Austin, Texas and introduced into the three laboratory bench-scale reactors. The biomass (activated sludge) in the bench-scale reactors was acclimated using the untreated wastewater that was collected at the Pantex treatment system.

The liquid volume in each of the three bioreactors is approximately 10 liters. The reactors were operated at a hydraulic detention time of one day with no wastage of biomass from the reactors during the acclimation period that extended through the first week of August 1998. Approximately one month (30 days) was estimated as the minimum time that would be required to acclimate the biomass to the wastewater. Acclimation was based on the concentrations of biomass that were maintained in the reactors, the settling characteristics of the sludge and the effluent concentrations of Chemical Oxygen Demand (COD). The initial target concentration of biomass in the reactors was approximately 2,500 mg/L (MLSS). The Chemical Oxygen Demand (COD) of the influent to the bioreactors during acclimation ranged from less than 100 mg/L to about 200 mg/L, and the effluent COD ranged from about 10 to 20 mg/L.

However, the concentration of biomass in the reactors decreased to 1322 mg/L, 1310 mg/L, and 980 mg/L, respectively. This deterioration, in part, was the result of the low concentrations of COD and BOD_5 in the Pantex wastewater. In spite of the decrease in the MLSS concentrations, the effluent COD concentrations ranged from 7mg/L to 12 mg/L, and the effluent BOD_5 concentrations were less than 3 mg/L in each of the three reactors.

The COD and BOD_5 concentration in the stored wastewater, at times, was below detection levels. Therefore, in late October 1998, untreated municipal wastewater from the South Austin Regional Treatment Facility was used as influent to the reactors. The concentrations of BOD_5 and COD of the untreated municipal wastewater were higher than the concentrations in the Pantex wastewater; therefore, the feed containing untreated municipal wastewater represents a worse case scenario for establishing design criteria and operating parameters for the treatment of wastewater at the Pantex facility.

6. RESULTS

The observed data were analyzed in order to develop kinetic parameters. The design relationship that was used to evaluate the kinetic coefficients is:

$$1/\theta_c \quad = \quad YU - k_d \tag{21}$$

The development of this relationship was discussed earlier in this report (see pages 14-15, Section 4.3) and the respective terms were defined.

Three reactors were operated in parallel with the same hydraulic detention time but different sludge ages (SRT). The operating conditions are summarized in Table 1. The flow rate to each of the reactors is represented by the symbol Q. The volume of solids removed from the system daily, Q_w, is also referred to as the sludge wastage rate.

Only one set of total BOD_5 removal data was available because of various difficulties previously discussed. The results used to evaluate the kinetic coefficients for total BOD_5 are presented in Table 2.

Table 1: Operating Conditions in Reactors

Reactor	Liquid Volume (L)	Flow Rate Q (L/day)	Nominal θ_h (days)	Waste Sludge Flow Rate Q_w (L/day)	SRT (days)
A	10.0	10.0	1.0	2.50	4
B	10.1	10.0	1.0	1.25	8
C	10.1	10.0	1.0	0.83	12

Table 2: BOD_5 Results

Reactor	Biomass X_{MLVSS} (mg/L)	Influent BOD_5 S_o (mg/L)	Effluent BOD_5 S (mg/L)	Specific Substrate Utilization Rate U (day^{-1})
A	493	174.77	5.32	0.34
B	667	174.77	3.04	0.26
C	752	174.77	1.52	0.23

The results used to evaluate the kinetic coefficients for total COD are presented in Table 3.

Table 3: COD Results

Reactor	Biomass X_{MLVSS} (mg/L)	Influent BOD_5 S_o (mg/L)	Effluent BOD_5 S (mg/L)	Specific Substrate Utilization Rate U (day^{-1})
A	270	126.48	70.36	0.21
B	679	126.48	42.02	0.12
C	737	126.48	28.99	0.13

The total BOD_5 data are plotted in Figure 4. The data indicate that the cell yield coefficient, Y, is 1.47 mg MLVSS/mg BOD_5 utilized. The endogenous decay coefficient is 0.25 day^{-1}.

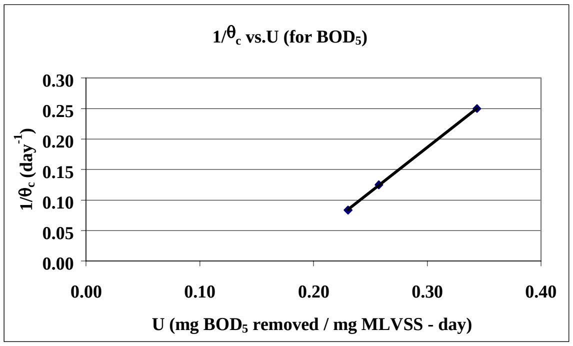

Figure 4: Relationship Between Specific Total BOD_5 Utilization Rate and Reciprocal of the Cell Residence Time

The total COD data are presented in Figure 5. The cell yield coefficient, Y is 1.78 mg MLVSS/mg COD utilized and the endogenous decay coefficient, k_d was 0.12 day^{-1}.

Other kinetic parameters, the half-velocity constant, K_s, and the maximum specific substrate utilization rate, k (mg/L BOD_5 or COD removed per day per mg/L MLVSS), also were evaluated.

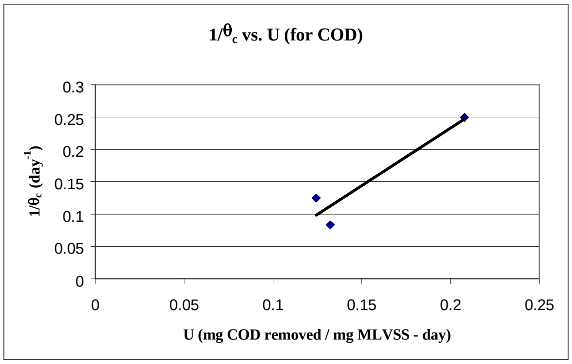

Figure 5: Relationship Between Specific Total COD Utilization Rate and Reciprocal of the Cell Residence Time

The half-velocity constant, K_S, represents the substrate concentration (mg/L) that results in biological cell growth of one-half the maximum growth rate. The maximum specific rate of substrate utilization, k, and the half-velocity constant are related in equation.

A plot of 1/U versus 1/S is a straight-line relationship. This relationship based on total BOD_5 and MLVSS is shown in Figure 6.

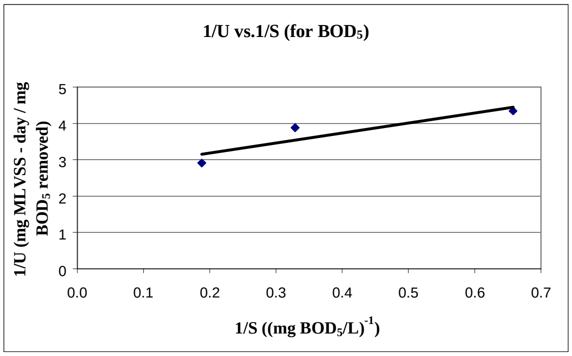

Figure 6: Determination of K_S and k based on BOD_5 Removal

The intercept of the line with the y-axis intercept of the plot is 1/k and the slope of the line is Ks/k. The data presented in Figure 6 indicate that k_{BOD5} 0.38 mg/L BOD_5/ mg/L MLVSS– d. The half-velocity constant $K_{S\ BOD5}$ was 1.05 mg total BOD_5/L.

A similar plot for the total COD data is presented in Figure 7.

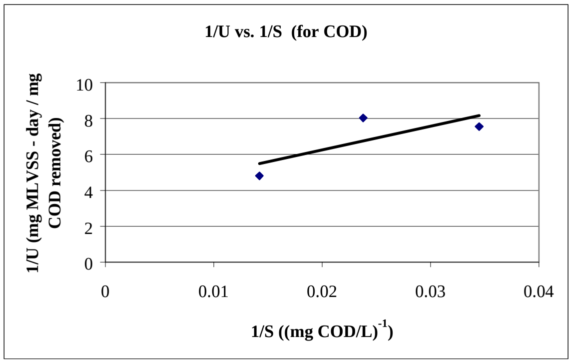

Figure 7: Determination of K_S and k based on COD Removal

The K_S was 36 .6 mg total COD/L and k was 0.28 mg/L total COD/ mg/L MLVSS-d.

The efficiency of the activated sludge process in the removal of total BOD_5 and total COD as a function of sludge age are shown in Figures 8 and 9.

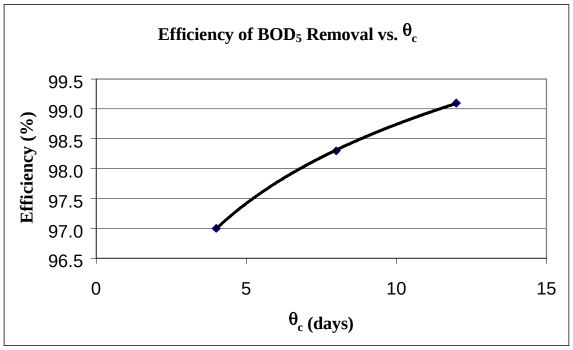

Figure 8: BOD_5 Removal Efficiency as a Function of Sludge Age

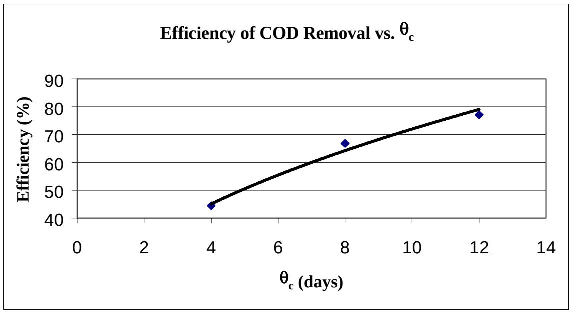

Figure 9: COD Removal Efficiency as a Function of Sludge Age

The BOD_5 removal efficiencies ranged from 97.0%, 98.3%, and 99.1% for sludge ages of four, eight, and 12 days, respectively. The COD removal efficiencies for sludge ages of four, eight and 12 days were 44.4%, 66.8%, and 77.1%, respectively.

The effluent suspended solids results are summarized in Table 4. The average effluent TSS concentrations in reactors A, B, and C were 11.6mg/L, 11.8mg/L, and 9.1mg/L, respectively. The average effluent VSS concentrations in reactors A, B, and C were 8.7mg/L, 8.9mg/L, and 6.8mg/L, respectively.

Table 4: Effluent Suspended Solids Concentrations

Date	Reactor	Effluent TSS (mg/L)	Effluent VSS (mg/L)
23-Nov	A	12.2	9.2
	B	11.1	8.3
	C	8	6.0
30-Nov	A	9.2	6.9
	B	12.6	9.5
	C	8.5	6.4
7-Dec	A	15.4	11.6
	B	17.2	12.9
	C	11.8	8.9
14-Dec	A	9.5	7.1
	B	6.3	4.7
	C	7.7	5.8

The concentrations of biomass in the reactors, in terms of mixed liquor total suspended solids (MLSS) and mixed liquor volatile suspended solids (MLVSS) are presented in Table 5. After initial biomass acclimation, the average MLSS concentrations in reactors A, B, and C were 628mg/L, 1441mg/L, and 1313mg/L, respectively. The average MLVSS concentrations in reactors A, B, and C were 401mg/L, 839mg/L, and 783mg/L, respectively. The biomass in all three reactors followed the same trend of decreasing concentration through the first week followed by an increasing trend near the end of the experimental run.

Sludge settling characteristics frequently are reported in terms of volume (mL/L) of settled sludge (SSV) after 30-minute. Settling characteristics of activated sludge also are reported in terms of a sludge volume index (SVI). The SVI represents the volume in milliliters occupied by 1 gram of mixed liquor suspended solids after 30 minutes of settling. Settling characteristics are summarized in Table 5. The SVI in all reactors steadily increased throughout the experiment. The SVI in reactors A, B, and C increased from 36.1mL/g, 25.4mL/g, and 36.4mL/g to 243.1mL/g, 43.7mL/g, and 44.8mL/g, respectively.

Table 5: Mixed Liquor and Sludge Settling Characteristics

Date	Reactor	MLSS (mg/L)	MLVSS (mg/L)	SSV (mL/L)	SVI (mL/g)
16-Nov	A	3569	1859	129	36.1
	B	4731	2521	120	25.4
	C	1898	1095	69	36.4
19-Nov	A	2870	1498	92	32.1
	B	3486	1854	100	28.7
	C	1891	1080	61	32.3
23-Nov	A	2029	1104	61	30.1
	B	2500	1245	63	25.2
	C	2361	1348	72	30.5
29-Nov	A	634	399	22	34.7
	B	1389	752	41	29.5
	C	1574	907	56	35.6
1-Dec	A	853	530	41	48.1
	B	1761	1056	60	34.1
	C	1550	948	58	37.4
2-Dec	A	703	352	48	68.3
	B	1655	827	62	37.5
	C	1497	648	58	38.7
3-Dec	A	604	424	57	94.4
	B	1550	934	60	38.7
	C	1324	835	57	43.1
4-Dec	A	401	254	38	95.0
	B	2030	1186	85	41.9
	C	1101	661	47	42.7
14-Dec	A	379	284	43	113.5
	B	1187	736	53	44.7
	C	1199	775	49	40.9
15-Dec	A	399	270	80	200.5
	B	1109	679	51	45.9
	C	1169	737	50	42.7
16-Dec	A	771	493	160	207.5
	B	1144	667	54	47.2
	C	1244	752	57	45.8
18-Dec	A	905	602	220	243.1
	B	1143	718	50	43.7
	C	1160	788	52	44.8

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