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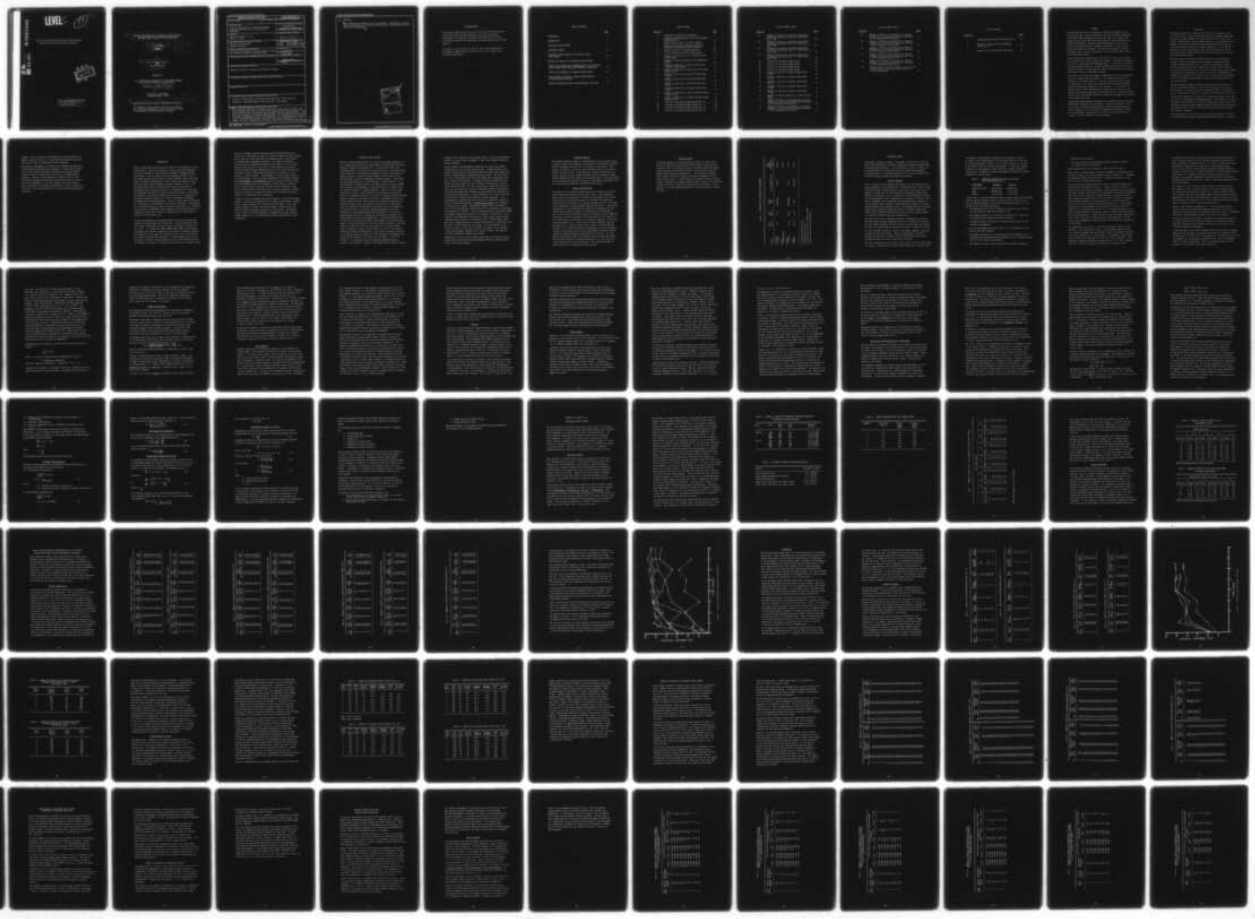

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This study was performed to determine methods of waste water treatment that could maintain high effluent quality even during start-up and periods of transient conditions such as rapidly varying waste flows and characteristics. The study was confined to activated sludge systems, flow-through and solids-return. The method recommended is the use of coagulants and weighting agents. The effects of varying environmental conditions such as pH, temperature, flow and organic load were explored and the results showed both the flow-through systems

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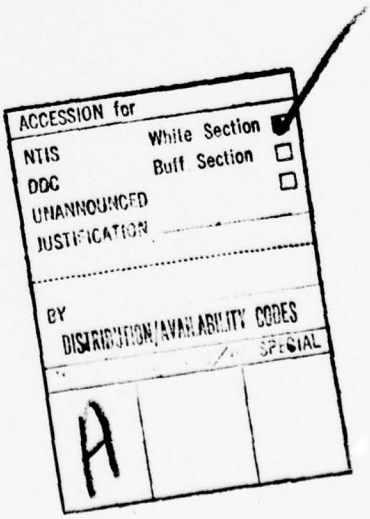
20. Abstract

and solids-return systems to be very resilient. Chlorination requirements of effluents from systems operating under non-equilibrium conditions were determined.



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ABSTRACT

The United States Army is concerned with pollution abatement at all of its facilities. Since the Army must meet the standards established for publicly owned treatment facilities and there are in many cases rapidly varying waste flows and characteristics, it was essential that methods be developed to maintain high effluent quality even during start-up and other periods of transient conditions. It was for this purpose the study was performed. The study was confined to aerobic biological liquid culture systems such as the activated sludge system and modifications thereto.

Several methods were explored for rapid start-up of both flow-through and solids-return activated sludge type systems. The method recommended is the use of coagulants and weighting agents. The addition of 10 g/cu m of aeration basin volume of a cationic polyelectrolyte and 2 kg/cu m of bentonite resulted in high immediate removal efficiencies and 95 percent COD removal within 72 hours after start-up.

The effects of varying environmental conditions such as pH, temperature, flow, and organic load were explored. The effects were studied for systems in a transient state, i.e., during start-up, and for systems at equilibrium. The results show that both the flow-through systems and the solids-return systems are very resilient. It is possible to change both the organic and hydraulic loading rate by a factor of two without significantly affecting the effluent quality. The flow-through systems were significantly affected by temperature changes but the solids-return systems were not.

Chlorination requirements of effluents from systems operating under non-equilibrium conditions were determined. Trends were detected. First, as the system approached equilibrium the chlorine dosage required to meet a standard such as 200 fecal coliforms/100 ml decreases. Second, as the temperature increases from 5 to 30°C the required dosage decreases. At 35°C, however, the trend reverses.

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CONCLUSION

The United States Army is concerned with pollution abatement at all of its facilities. Since the Army must meet the standards established for publicly owned treatment facilities and there are in many cases rapidly varying waste flows and characteristics, it was essential that methods be developed to maintain high effluent quality even during start-up and other periods of transient conditions. It was for this purpose the study was performed. The study was confined to aerobic biological liquid culture systems such as the activated sludge system and modifications thereto.

The first phase of the study developed methods for rapid start-up. Several methods were explored which included: bacterial seeding, organic enrichment and the use of coagulants and weighting agents. Bacterial seeding using soil, commercially available dried bacterial cultures and mixed liquor from an activated sludge system was not successful since the concentration of active cells was not significantly increased by the addition of the seed, i.e., the bacterial concentration in the raw waste was approximately the same as the concentration in the seed materials.

Organic enrichment greatly increased the bacterial concentration but was not recommended because it was costly and required the system to be bypassed for a period equal to the retention time of the aeration basin. Under certain conditions, however, this system may be practical.

The method recommended for start-up of an activated sludge type system is the use of coagulants and weighting agents. The addition of 10 g/cu m of aeration basin volume of a cationic polyelectrolyte and 2 kg/cu m of bentonite resulted in high immediate removal efficiencies and 95 percent COD removal after 72 hours. This method is also much more economical than organic enrichment.

The second phase of the project tested the proposed method of start-up on systems operating under varying environmental conditions. Start-up

and operation of systems were performed at 5 degree intervals from 5° to 35°C. At 5°C the removal efficiencies were not satisfactory, the MLVSS remained low, and the concentration of the polyelectrolyte had to be increased from 10 to 25 mg/l and repeated at 12 hour intervals to maintain the floc in a condition for efficient clarification. The removal efficiency was considerably greater at 10°C than at 5°C but still was not consistantly greater than 80 percent. Within the range of 15 to 30°C there was little difference in process efficiencies. In this temperature range the system reached equilibrium in less than four days and maintained an efficiency greater than 90%. At 35°C the maximum removal efficiency was considerably lower (68%) than at 30°C (95%). These studies indicate that high treatment efficiencies can be achieved over a wide range of operating temperatures. Special design modifications will be required however, for areas where there is a wide variation between mean summer and winter temperatures.

The effect of varying the pH of the influent waste on start-up and efficiency of the waste treatment system was also determined. To determine the effects of pH on the start-up characteristics of the activated sludge system the pH of the wastewater was adjusted to values of 6, 7, 8, and 9 with either sodium hydroxide or sulfuric acid. At an influent pH of 6 the pH in the system stabilized at a value of 6.8 after 5 days. The settling was poor due to bulking conditions. The operation was improved considerably by increasing the pH to 7. At an influent pH of 7 or 8 the pH of the system stabilized at a value of approximately 8. Bulking did not occur and the COD removal efficiency increased to over 90% within 4 days of start-up. A pH of 9 inhibited some of the free swimming protozoa, settling was poorer and the removal efficiencies less than at a pH of 7 or 8.

The flow rate to a waste treatment system is continuously varying and therefore it is necessary to determine the extent to which it can

vary and still not adversly affect the system to a degree that it cannot be controlled. This test was performed at a constant organic loading of 75 lbs COD/1000 ft^3/day, while the hydraulic loading rate was varied by a factor of 4, i.e., the hydraulic retention time was varied from 4 to 16 hours. The results indicate that as the concentration of the influent increases simulating a decrease in flow rate the effluent concentration increases. However, the total pounds of COD discharged decreases as the retention time increases.

The organic loading rate is a design parameter used to predict the effluent quality. These data show that the activated sludge system is very resilient. It is possible to change the organic loading rate by a factor of 2 without significantly affecting the removal efficiencies or effluent quality. Increasing the organic loading rate by a factor of 4, from 50 to 200 lbs COD/1000 ft^3/day resulted in a decreased effluent quality. These data are predicated on the availability of sufficient oxygen.

Aerated lagoon systems were significantly affected by temperature. The systems were run at temperatures of 10-, 15-, 20-, 25-, 30-, and 35°C. The time to reach 80 percent COD removal from start-up ranged from 10 days at 30°C to 32 days at 10°C.

The effects of shock loading an activated sludge system hydraulically and organically were determined. These results indicate that a transient hydraulic load of two times the design flow is not detrimental to the system if the solids settling rates are increased consistant with the increase in overflow rates. This can be accomplished by the addition of a cationic polyelectrolyte. A transient hydraulic load of three times the design flow decreases the efficiency significantly.

Organic overload factors of 1.5, 2 and 3 were simulated. An organic overload factor of 2 resulted in a concomitant rise in MLVSS while maintaining a high COD removal efficiency of 96-97 percent. The sludge produced was readily settled and solids washout was not a

problem. Very high removal efficiencies were also realized at an organic overload factor of 3. The effluent COD however, was high and would not meet the EPA NPDES effluent standards.

Chlorination requirements of effluents from systems operating under non-equilibrium conditions were determined. Trends were detected. First, as the system approached equilibrium the chlorine dosage required to meet a standard such as 200 fecal coliforms/100 ml decreases. Second, as the temperature increases from 5 to 30°C the required dosage decreases since the chlorine reaction rates increase with temperature. At 35°C, however, the trend reverses. This is probably due to higher concentrations of fecal coliforms at 35°C then at lower temperatures due to more favorable environmental conditions.

INTRODUCTION

Public concern about water pollution caused by the discharge of wastewaters into our nation's waterways is an ever increasing phenomenon and public demands for water pollution abatement programs parallel this concern. This is evident from the enactment of Public Law 92-500 which requires that all publicly owned treatment works, including those of military installations, attain removal efficiencies equal to best practicable waste treatment technology (BPWTT) by July 1, 1983. The United States Army is concerned with pollution abatement of discharges from its facilities since it is imperative that they meet the standards established for publicly owned facilities. The problems encountered by the Army are complicated at many of its facilities by the rapid population changes that result from summer and weekend encampments and training maneuvers. Also, the Army operates facilities in a wide range of environmental and climatic conditions. Consequently the treatment systems must control pollution discharges at an acceptable level and must be dependable and economical, while being subjected to temperatures from below freezing to temperatures over 100°F and to rapidly varying loading rates.

To meet these demands, an increasing number of activated sludge type systems are being planned and constructed. Activated sludge systems have several advantages over other types of biological treatment systems since: 1) biochemical oxygen demand (BOD) removal efficiencies of greater than 95% may be expected if the system is properly designed and operated, 2) low capacity, packaged units for small installations are readily available and economical, 3) large installations are also practical if they are designed specifically for that installation with its inherent variations, 4) operational controls such as aeration rates,

mean cell residence time and bacterial solids concentration may be varied over a wide range in order to attain the highest effluent quality, and 5) they are readily adaptable and compatible with a wide variety of advanced waste treatment operations. For example, phosphorus reduction may be accomplished chemically by proper use of such coagulants as lime and alum or iron salts. These chemicals are usually introduced before primary sedimentation but may be added directly into the aeration basin, permitting the phosphorus to settle out in the primary or final clarifier. The phosphorus is then disposed of with the wasted sludge. Also, by proper adjustment of the sludge retention time, Nitrosomonas and Nitrobacter bacteria will have sufficient time to convert ammonia (NH_3) in the sewage to nitrate (NO_3^-) which may then be anaerobically reduced and stripped from the system as a gas. The low solids content of effluent from a properly operating activated sludge unit makes it possible to further treat the wastes with sand filters or to chlorinate for disinfection.

A number of factors influence the proper operation of activated sludge systems. These include temperature, pH, organic and hydraulic loading rates, sludge retention time, aeration rate and others. Short term extreme changes or chronic but subtle changes of these factors may upset the proper operating equilibrium of this system. The purpose of this study is to develop methods to compensate for these changes, thereby insuring that the established effluent standards will be met consistently.

ACTIVATED SLUDGE SYSTEMS

There are several modifications of the activated sludge system but in all of them organic wastes are simultaneously oxidized and converted to bacterial solids in an aeration tank. The bacterial solids form a settleable mass or "floc" which is then settled out of the mixed liquor (the biomass plus the suspending wastewater) which is now reduced in organic content. A portion of the biomass is recirculated to the aeration basin and used to assimilate incoming organics. The treated and clarified effluent is then discharged to surface waters or to an advanced waste treatment process. The biomass produced, which exceeds that necessary for maintenance of the mixed liquor, is removed from the system generally to a sludge digester. Bacteria are the primary stabilizing agents in the biomass of the activated sludge system. When food is abundant relative to the mass of bacteria and the dissolved oxygen is adequate, the bacteria will have a logarithmic growth rate. In this stage of growth, which would exist during the start-up of an activated sludge system, the bacterial solids are in a high energy state and resistant to flocculation, and would tend to be washed out of the system with the effluent, thereby lengthening the period necessary to reach expected operating efficiencies. As the food to microorganism (F/M) ratio declines and the biomass increases, floc forming bacteria begin to predominate and a settleable biomass forms. At the end of the log growth phase, free swimming, ciliated protozoa begin to consume the bacteria. As the F/M ratio continues to decrease, stable floc particles form and the bacteria are reduced in number. Then the stalked ciliates attach directly to the floc particles and become the dominant protozoan form. Only after the organic content of the clarified liquor is reduced will the succession of organisms take place with the appearance of higher forms of animal life such as rotifers. At this point, when an equilibrium between bacteria, protozoa, and rotifers is reached, the sludge mass will settle readily,

leaving a clear effluent low in organic content. The activated sludge system will now be fairly resistant to short term changes in environmental conditions.

Various changes in environmental conditions can reduce the removal efficiencies of an activated sludge system. A rapid increase in the organic loading rate can increase the F/M ratio which will promote the growth of high energied, dispersed bacterial cells. This will result in a loss of bacterial solids from the system with a consequent reduction of removal efficiencies. Increased hydraulic loading rates may exceed the design capacity of the clarifier, resulting in a loss of solids from the system. If this continues for a sufficient period of time, the solids loss will result in negligible removal efficiencies. The dissolved oxygen (DO) concentration also affects the operation of an activated sludge system. A DO concentration of at least 2 mg/l must be maintained to support the desirable, aerobic organisms. It is advantageous to maintain less than 5 mg/l DO to inhibit the growth of certain filamentous organisms as Sphaerotilus natans which contribute to bulking of the sludge. Temperature changes affect the metabolic rate of the microorganisms involved and must be considered in the design and operation of a system. A drop in temperature may sufficiently reduce the organic assimilative capacity of the system to necessitate an increase in the concentration of bacterial solids (mixed liquor volatile suspended solids, MLVSS) in the aeration tank. A rapid change in pH may cause a significant drop in removal efficiencies, at least until the bacterial flora acclimates to the change. If the pH change of the system is severe, it may kill or inhibit the growth of all organisms. A less severe change, extended over a period of time, may cause undesirable organisms to dominate the system.

Modification of the activated sludge system that are presently used by various Army installations include extended aeration, contact stabilization, and aerated lagoons.

Extended Aeration

The extended aeration system is a modification of the completely mixed activated sludge type treatment process wherein the hydraulic retention time is increased from 3-8 hours to a period of 18-36 hours. This process is generally limited to small treatment plants of one million gallons per day or less. In this process, primary clarification is usually omitted and the sludge retention time is increased to permit aerobic digestion of the sludge in the aeration basin. With this system, the need for solids disposal facilities is diminished considerably from that required by conventional activated sludge systems.

Contact Stabilization

The biomass of an activated sludge system has the ability to quickly adsorb dissolved organics from the waste stream. This phenomenon, which occurs in 30-45 minutes, is used to an advantage in the contact stabilization or biosorption system. The influent is brought into contact with the bio-solids in a contact aeration basin for the length of time necessary to attain maximum removal efficiencies. The mixture then flows to a clarifier where the effluent is discharged and the concentrated solids are pumped to a stabilization aeration basin. Here a portion of the adsorbed organics are oxidized and the remainder converted to cellular materials. The bio-solids are then recycled to the contact basin or discharged to an anaerobic digester or some other solids separation unit. This process is highly dependent upon the maintenance of an optimal contact time in order to achieve maximum removals. Due to the fluctuations in wastewater flow rate throughout the day, this optimal contact time is impossible to maintain without extensive storage facilities to equalize the flow rates. For this reason, and because contact stabilization cannot achieve BOD removal efficiencies as high as conventional systems, it is not currently a viable alternative for new construction. The existing systems are being phased out or followed by another process.

Aerated Lagoons

An aerated lagoon is an aerobic/anaerobic system in which a mixed liquor of relatively low bacterial solids content is maintained in an aerobic zone in the upper portion of the basin, while the heavier sludge particles are allowed to settle to the bottom of the basin where they are digested anaerobically. Long hydraulic retention times, usually 30 days or longer, are used to compensate for the lower solids content. In this system, no attempt is made to recycle sludge. Therefore, the sludge retention time is equal to the hydraulic retention time and the MLVSS are wasted with the effluent. A comparison of the operational parameters for the above systems is shown in Table 1.

Table 1. COMPARISON OF OPERATIONAL PARAMETERS

Process	HRT[a] (hours)	SRT[b] (days)	MLVSS[c] mg/l	lb BOD/1000ft^3/day	BOD removal efficiencies (%)
Activated sludge	3-8	5-15	1500-5000	30-120	85-95
Contact stabilization					80-90
Extended aeration	18-36	20-30	3000-6000	10-25	75-95
Aerated lagoons	10-45[d]	10-45	200	0.25-0.50	80-95

[a] hydraulic retention time

[b] sludge retention time

[c] mixed liquor volatile suspended solids

[d] HRT is in days for aerated lagoons

LITERATURE REVIEW

This chapter contains a survey of literature related to the start-up, control and operation of activated sludge waste treatment systems under non-equilibrium conditions. It also includes a review of research on the determination of coliform bacteria and coliform bacteriophage (viruses parasitic to coliform bacteria) die-off rates resulting from activated sludge treatment and disinfection methods.

Military Systems

The U.S. Army has long recognized the necessity of treating the wastewater generated at its installations. During World War I, septic tanks, trickling filters, contact beds and intermittent filters were used (1). However, the waste control facilities were considered inadequate and difficult to maintain. Prior to World War II the Construction Division of the Quartermaster Corps, transferred to the Office of Chief of Engineers in December, 1941, appointed an Engineering Board, composed of the consulting firms of Metcalf & Eddy and Greely & Hansen, to develop sewage treatment plant design criteria. The Engineering Board preferred sedimentation tanks with mechanical removal of sludge, rather than Imhoff tanks; trickling filters rather than activated sludge systems; and high rate filters rather than standard rate.

The Army in June, 1941, adopted the Board's recommendation, with the provision that treatment standards be no more stringent than required by local stream standards and local custom, and that high rate filters be built only where stone was difficult to obtain or space was limited.

By 1945, the Army had built a total of 399 sewage treatment facilities at bases in the United States. These included primary treatment, standard and high rate trickling filters, intermittent sand filters, activated sludge, and contact stabilization systems.

The above systems are also typical of those used to treat civilian domestic sewage. The differences between wastes generated in military bases

and typical civilian domestic sewage has been studied and found to differ in the concentrations of constituents but not in the type of constituents present (Table 2) (1). Military sewage, at the time of the study, tended to be more concentrated. It also had a higher volatile to fixed suspended solids ratio and a higher variation in diurnal flow. "Industrial" wastes were generally sewered and treated separately at military bases.

Table 2 . COMPARISON BETWEEN MILITARY AND CIVILIAN SEWAGE CHARACTERISTICS (1).

Constituent	Military	Civilian
Suspended Solids	378-486 mg/1	200-300 mg/1
BOD_5	280-360 mg/1	175-275 mg/1
Grease	126-162 mg/1	75-125 mg/1

In 1959 the U.S. Army Environmental Hygiene Agency studied small extended aeration sewage treatment plants and compared the operational data with the design parameters in the Army's engineering manual (2). They arrived at the following conclusions:

1. The design peak flow rate of 150 gpcd for residents of a housing area was about the same as the measured flow.
2. The design allowance of 127.5 gpcd for residents and 45 gpcd for non-residents were greater than observed values.
3. A design allowance of 80 gpcd for residents and 30 gpcd for non-residents was suggested.
4. The 4 hour maximum design flow, 175% of daily average flow, was greater than that observed.
5. The design allowance of 0.2 lb/capita/day for the BOD and 0.2 lb/capita/day for suspended solids was greater than the observed values of 0.13 lb and 0.09 lb respectively.
6. BOD removal of 85-90% could be expected with properly operating

extended aeration plants.

7. The plants studied provided neither adequate hydraulic capacity nor adequate control of aeration.

Design standards were then adjusted to reflect these findings.

More recently, the Department of Defense commitment to pollution control in fields of water, air and solid waste has been reported (3). Increased environmental budgets were forecasted to enable the Department of Defense to meet its environmental goals.

The Army operates treatment systems in areas in which the climatic conditions vary from tropical to arctic. Studies of Army facilities in South Vietnam have also been reported. These installations were virtually the first systems outside major South Vietnamese cities more sophisticated than septic tanks. Because of limited materials, construction skills, short completion deadlines and lack of technical training for operation and maintenance personnel, plant construction in South Vietnam after 1967 was restricted to waste stabilizatiom lagoons (4). These lagoons were subject to environmental factors typical of tropical areas not common to more temperate areas of the U.S. The monsoon climate of the area is characterized by consistently high air temperature, high humidity and heavy precipitation. Annual rainfall exceeds the annual evaporation rate. Solar radiation is intense and so uniform throughout the year that the mean temperature varies less than $10^{o}C$ between the warmest and coolest months.

The temperature of the water in the South Vietnamese lagoons studied was $28^{o}C$ - $35^{o}C$ throughout the year. The warm temperatures and abundant sunlight enhanced algal growth and bacterial metabolism, resulting in a rapid assimilation of organic wastes. The design of waste stabilization lagoons was adjusted to benefit from these conditions. Consequently, hydraulic detention time was set at 17 to 27 days, compared with 30 to 60 days for similar installations in temperate zones.

The Department of Defense also maintains waste stabilization ponds in arctic areas which have entirely different operating conditions from those in tropical or temperate areas. In a study of lagoons for arctic and subarctic regions it was noted that the conventional design, construction, operation and maintenance applied to sewage lagoons in temperate climates cannot be applied because of the temperature extremes experienced (5). However, the effluent standard criteria it was stressed, are actually rendered less important because of the sparse populations and high stream flows. Table 3 lists design characteristics for arctic lagoons.

The single cell, long retention lagoon requires a retention time of 8-12 months to allow sufficient storage to prevent discharge during the period when the ponds were frozen over. During the winter months the treatment level was only slightly better than the primary treatment experienced by physical settling because of reduced biological activity and obstruction of sunlight by ice. Two to three weeks after the summer thaw the treatment level reached secondary quality.

Primary, short retention lagoons with 2-4 day retention times in each of 2-4 cells provided a BOD_5 removal efficiency of 70% in the summer and 30% during the winter when the ponds were frozen. Accumulated solids were degraded anaerobically in the bottom of the ponds.

Secondary, long retention lagoons were generally constructed based on the same criteria as the single cell ponds. For populations greater than 300, a system of short retention lagoons in series with a long retention lagoon was recommended.

A study was made on the Central Alaskan Eielson Air Force Base oxidation pond which was subjected to extremes in temperatures ranging from 23°C (75°F) in June to -47°C (-53°F) in December (6). The pond was frozen from mid November to late February, during which only 0.05 lb of BOD_5/day/1000 ft^3 could be assimilated. This was principally because of low biological activity and low sunlight intensity.

Table 3. DESIGN CHARACTERISTICS FOR ARCTIC LAGOONS (5)

Type of Lagoon	Retention Time	Summer Operating Depth	Winter Operating Depth	BOD Loading
Single cell long retention	8-12 mo.	4-5 ft.	6-8 ft.	20 lb/day/acre
Primary Single cell	10 days	10-25 ft.	10-25 ft.	6-9 lb/day/1000 $ft.^3$
Secondary Long retention	8-12 mo.	4-5 ft.	6-8 ft.	20 lb/day/acre
Aerated	30 days	10-20 ft.	5-15 ft.	0.6 lb/day/1000 $ft.^3$

In the study a blower aeration unit was installed to permit aeration when the pond was frozen. The system consisted of submerged perforated distribution pipes and an enclosed air compressor. The lagoon froze over, minimizing heat loss from evaporation or back-radiation. The only protection afforded the system came from heat generated by compressing the air which was sufficient to keep water from freezing in exposed pipes. At a detention time of 20 days during the coldest part of the winter, BOD_5 removals were nearly 80 percent. BOD_5 assimilation was increased from 0.05 lb/day/1000 ft^3 to 0.5 lb/day/1000 ft^3. A longer detention time did not improve BOD removal efficiencies (6).

Temperature Effects

Temperature is an important parameter in biological wastewater treatment systems, because of the effects it has on microbial growth and predomination (7). Many researchers have reported on the effect of varying temperatures on biological systems with little consensus of opinion. Bloodgood (8) by analyzing four years of operational data found that decreasing the temperature 10°C resulted in a 36 percent increase in the effluent concentration. Porges, et. al. (9) in a study on milk wastes found that the COD removal efficiencies significantly decreased as the temperature was decreased from 30 to 10°C. They found to maintain equal COD removal efficiencies, approximately 30 hours were required at 30°C and 70 hours at 10°C. Carter (7) studied the effects of temperature shock on a biological system by acclimating a system with a sludge retention time of 2 days and a sludge retention time of 5 days to a temperature of 35°C and then performing oxygen uptake studies in a Warburg respirometer at temperatures of 20, 40, and 45°C. In all three of these Warburg runs the COD removal efficiencies were 85 percent or better. However, a difference in the length of time required to achieve a minimum COD value was noted. A minimum COD value of about 60 mg/l was attained in 2 hours in both the 35°C and

40°C runs. The 20°C and 45°C tests exhibited similar curves. Both had a COD of 200 mg/l at 2 hours and did not reach a minimum for 5 hours at 20°C and 3-4 hours at 45°C. However, after 10-12 hours the COD removal value approached a common level. Benedict and Carlson (10) studied temperature acclimation in aerobic bio-oxidation systems. They, starting with a culture from a system operating at 15°C to 19°C, acclimated cultures to 4°, 19°, and 32°C. The response of the 19°C culture was constant for the research period indicating no requirement for temperature adaption. Approximately 2 weeks were required for acclimation to 4°C. However, the temperature adaptation response of the 32°C culture was severe, indicating that the acclimation time required by mixed cultures at high temperatures is on the order of months. Benedict and Carlson also studied the sludge settleability, as measured by gross settling rates and SVI. They found that the settleability was not adversely affected by operating temperatures between 4°C and 32°C. However, at 32°C the culture becomes partially dispersed and supernatant rather turbid. They also found that the reaction rate constant k_r (mg O_2/hr) increased significantly with an increase in temperature.

Assuming Benedict and Carlson's data follow the modified Arrhenius equation

$$\frac{k_1}{k_2} = \Theta^{T_1 - T_2}$$

where: k_1 and k_2 = reaction rates at temperatures T_1 and T_2

Θ = temperature coefficient

Then the temperature coefficient ranges from 1.05 to 1.07.

Friedman and Schroeder (11) studied the effects of temperature on the growth and yield of activated sludge. They stated that the effect of

temperature on growth or oxidation rate and stoichiometry are important and should be considered in process design, but that the modified Arrhenius equation could not be used to describe the temperature effects on an activated sludge system. The lack of observable temperature effects on BOD removal may be attributed to excess reactor capacity and in many cases over design of the process.

Sludge Settleability

The operational problem most discussed in the literature is bulking or non-settling sludge. The ability of sludge to settle is generally measured by the sludge volume index (SVI) (12).

The sludge volume index is measured in a one liter graduated cylinder in which a mixed liquor is allowed to settle quiescently for 30 minutes. The volume of sludge in milliliters is divided by the mixed liquor volatile suspended solids, measured in grams. A sludge with a SVI of 50-200 is said to have normal settling characteristics, while a SVI greater than 200 is indicative of a poorly settling sludge.

The sludge volume index is not always a good indicator of settleability. If the MLVSS equals 10,000 mg/l and no settling occurred in the liter graduate cylinder the sludge volume index would be:

$$SVI = \frac{\text{volume of sludge in ml}}{\text{MLVSS in grams}} = \frac{1000}{10} = 100.$$

This SVI indicates normal settling for a sludge which, in fact, does not settle at all.

Bulking of sludge has been traced to a wide variety of causes. The problem may be produced by excessive growth of filamentous organisms. A number of organisms have been identified as causing or contributing to sludge bulking (13). These include filamentous bacteria such as Sphaerotilus natans and Thiothrix, filamentous fungi, yeasts, and in some cases, protozoa.

Of these, the role of S. natans in bulking has been studied in detail,

with sometimes conflicting results. S. natans, in one study, was found to be a factor in bulking, but was in itself an effect of influent characteristics (14). It tended to grow when influent characteristics were rapidly changed, but could not be induced to grow when the system was operating at steady state. Studies have also correlated growth of S. natans with high carbohydrate content in wastewater influent (13). Other causes of bulking have been proposed or identified. The published literature, in fact, contains such a wide variety of causative factors and cures for sludge bulking that the causes in one case may very well be the solution in another. Causes have been traced to insufficient oxygenation (15), high carbohydrate levels (16), high grease levels, septic effluent (17), hot weather (18), phenols and cyanides (19), chromium (20), copper, nickel, zinc, and chromium (21), and pH outside the range of 6-9 (22).

Of particular interest to this study is the role played by shock loads in bulking. Large fluctuation in wastewater flow rate or concentrations has been found capable of causing bulking, although poor treatment efficiencies in this case may be caused by a washout of an otherwise settleable sludge due to excessive clarifier overflow rates (17).

Shock Loading

Normally steady state conditions are assumed in the development of kinetic models for the design and operation of aerobic liquid culture systems. It is well known, however, that perturbation, or shock loadings, occur which tend to upset the system thereby decreasing the treatment efficiencies. George and Gaudy (23) studied the response of completely mixed flow through systems to hydraulic shock loads using glucose as a carbon source. They considered two types of shock loads; in the first called "constant feed concentration," a change in the dilution rate was imposed with no change in the inflowing substrate concentration; where as in the second, the change in the dilution rate was accompanied

by a compensating change in feed substrate concentration so that the daily organic loading, i.e., milligrams of substrate fed per unit of time remained constant. The latter was termed "constant daily organic loading ." In this study a simple step change, with the new flow rate holding until a new steady state conditions approach, was selected. The system studied was a flow through system i.e., no cell feedback with a baseline dilution rate for the initial steady state of 0.125 hr^{-1}, i.e., a detention time of 8 hours. George and Gaudy studied both increases and decreases in the dilution rate.

Under conditions of constant feed concentrations (1000 mg/1), the flow rate was varied from reductions to 25 percent of the reference level to increases of 350 percent. Under these conditions, the organic loading level at the base dilution rate was 198 lb COD/1000 cu ft (3.17 kg/m^3 to 11.10 kg/m^3). The results of these studies provided evidence that hydraulic shocks constituting an increase in the dilution rate of 100 percent can be successfully accommodated.

When hydraulic shocks of like magnitude are imposed upon the system under conditions of constant daily organic loading, a decrease in dilution rate comprises the more severe perturbance since the system must also accommodate a rise in the influent concentration. For example, decreasing the dilution rate to one-fourth its former value with a concomitant increase in the influent concentration by a factor of 4 caused a rather severe transient disturbance before the system approached a new steady state condition. Shock loadings consisting of increases in dilution rate with concomitant decreases in the influent concentration led to a rather orderly wash-out of biological solids to a new steady state level with small fluctuations in effluent COD. These data indicate that hydraulic shock loads, under conditions of constant organic loading can be accommodated without serious disruption if the dilution rate and the change in influent concentrations do not change by more than a factor of two in either direction.

Another study with organic shock loading of modified extended aeration systems indicated that batch systems could handle shock loads double the normal 500 mg/l glucose feed rate with no loss of COD removal efficiency, and that continuous flow systems could tolerate even greater shock load without reduction in organic removal efficiency (24). A laboratory study of hydraulic overloading led to the conclusion that a completely mixed system could tolerate a 100% hydraulic overload with no decrease in organic removal efficiency, provided the organic loading rates were constant (25).

A study of organic shock loading with sludge recycle concluded that the ability to adsorb shock loads was inversely related to the steady state growth rate prior to the shock (26).

Start-up

Start-up of new or inoperative sewage treatment plants has been largely unreported, although digester start-up has been covered to some extent (27, 28,29,30). The U.S. Environmental Protection Agency (31) recommends start-up procedures for activated sludge systems both with and without the use of seed sludge. The recommended seed sludge concentration is at least 500 mg/l mixed liquor suspended solids. A one MGD plant would require 8 to 10 thousand gallons of sludge. The mixed liquor is then aerated, with 10 percent of the plant flow introduced initially and 10 percent increments added daily if there is no indication of process deterioration. Thus, at best, this method would require a discharge equal to 4.5 days untreated flow into the receiving stream.

In the EPA start-up procedure, utilizing no seed sludge, the aeration basin is filled with raw influent rather than primary effluent. The aeration unit is then bypassed for 8 hours while the raw wastewater is being aerated. After the aerators are turned off the aeration basin is settled for one hour. The supernatant is drained and the aeration basin refilled with raw effluent. This process is repeated until the

mixed liquor suspended solids level has reached at least 500 mg/l. During this start-up procedure the dissolved oxygen must be maintained at 2 mg/l. As in the first method, the aeration basin must be bypassed during the start-up procedure.

Activated sludge plants have usually been started by filling the aeration tank with wastewater and beginning operation in a normal manner. Buildup of a settleable floc is slow, however, and several days to weeks can elapse before the bacterial solids can be brought to the desired level.

One engineer accelerated the start-up procedure for extended aeration basins without primary sedimentation by allowing the basin to fill and overflow (32). After three days aeration was begun and sludge buildup was rapid. During the three day period the effluent quality was lower than primary quality and obnoxious odors were produced.

System Control

Methods for the correction of sludge bulking include; the addition of materials to the aeration basin or final clarifier, such as activated carbon (33,34), digested sewage sludge (35), raw sewage solids (36), inorganic (37) or organic polyelectrolyte type coagulants (38).

Activated carbon has been used on both industrial (dye and alcohol) wastes and domestic wastes to enhance organic removal efficiencies (33,34). The industrial waste system utilized powdered carbon of various types to enhance color removal, aid in removal of refractory organics, improve BOD removal efficiency, increase hydraulic loading capacity and decrease foaming in aeration basins. Studies with granulated carbon have included the use of coal (39) and granulated activated carbon (33,34), with recycling of the carbon to the aeration basin after it has been regenerated in a carbon slurry aerated regeneration basin.

The use of coagulants to minimize operational problems due to shock loads and overloading has been studied. Chemical coagulants may be divided into two categories, inorganic and organic. The inorganic coagulants treat the bacteria as a biocolloidal system. They reduce the net negative surface charge and thus the zeta potential, thereby permitting the bacteria to agglomerate. Inorganic cations, such as iron salts (Fe^{+2} as ferrous sulfate, Fe^{+3} as ferric chloride and ferric sulfate) and aluminum salt (Al^{+3} as alum) are used to reduce this negative surface charge (40,41). Sewage systems contain sufficient alkalinity so that lime addition is not necessary when using alum as a coagulant. Alum addition must be closely controlled since high concentrations have been found to be toxic to the protozoan population of activated sludge systems (41). However, this toxicity may not markedly affect the organic removal efficiencies, since bacteria are the primary removers of organics. Protozoa primarily enhance bioflocculation and assist in removing dispersed bacteria from the system, which yields a clear effluent. If the bacteria are artificially flocculated through alum addition, allowed to settle, and thereby prevented from being washed out with the effluent, the necessity of a good protozoan population is minimized.

In addition to acting as coagulating agents, some inorganic salts such as alum, ferric chloride and ferric sulfate have the desirable effect of reducing the phosphate content of the sewage (42,43). Although usually applied prior to the primary sedimentation basin, coagulants may perform a dual function of phosphate precipitation and flocculation of biosolids if added to the aeration tank.

Coagulating agents may also be organic in nature. These polyelectrolytes are available in anionic, cationic and nonionic form, with relatively linear or highly branched structures. They are widely used in water treatment coagulation process. These same coagulants may be used in wastewater treatment to aid primary sedimentation (44) or in aeration

basins as an aid to bioflocculation.

The mechanism of action of the polyelectrolytes is two fold. First the charge imparted to the polymer in either the anionic or cationic form acts to reduce the zeta potential of the colloid. Second, they act as bridging agents binding the colloids together into a floc. The number of charges on the polymer may vary from zero, as in the case of the nonionic polymer polyethylene-oxide, to very highly charged polyacrilimide. The efficiency of these highly ionized polymers is a function of the number of charges per molecule.

Finger (45) stated that excessive solids loss in secondary clarifiers is normally a result of sludge bulking associated with either hydraulic or biological upsets. He further states that hydraulic bulking affects most plants to some degree. This condition can be sporadic due to storm flow or chronic due to overloaded clarifiers. Finger studied the feasibility of solids control using alum. He found that the addition of alum during full-scale plant tests resulted in: (1) a rapid decrease in the sludge volume index from 130 to 75, (2) a disappearance of the filamentous organisms, (3) a decrease in the average depth of sludge, and (4) a greatly increased sludge density. Based on his studies Finger stated that it is possible to double the design capacity of clarifiers by the addition of alum.

Singer et.al. (38) studied the flocculation of bulked activated sludge with polyelectrolytes and alum. He determined the efficiency of treatment with two cationic and one anionic polyelectrolyte and alum. The anionic polyelectrolyte had essentially no effect on settling. The cationic polyelectrolytes decreased the SVI by 67 to 75 percent at dosages ranging from 2 to 5 mg/1. They concluded that the quantitative effect of the flocculant in reducing the sludge volume and the SVI depends on the initial SVI and the SS concentration. An increase in SS concentration requires an increase in the polyelectrolyte concentration. Alum also proved successful in promoting settling of bulked sludge,

but much greater concentrations of alum than polymers were required. Concentrations of 50 to 300 mg/l resulted in a SVI decrease of 64 to 73 percent.

The use of ferric chloride, coupled with polyelectrolyte addition, enabled an overloaded plant to produce acceptable effluent, even though the plant was operating at 240 percent of its design capacity (46). Moreover, a BOD_5 and SS removal of 85-90 percent was maintained by using cationic polymers for raw sewage flocculation and anionic polymers for mixed liquor flocculation.

The addition of oxidizing agents, such as hydrogen peroxide (47, 48, 49) has been successful in some cases of filamentous bulking associated with S. natans. When Thiothrix was involved (50) it was found beneficial to reduce the concentration of sulfides in the aeration basin.

Controlling operational parameters have also been used to reduce or eliminate bulking. These include variation of operational procedures such as maintenance of suitable MLVSS and aeration rates (51) and control of return sludge aeration (52).

Chlorination and Disinfection of Wastewater

The main purpose of wastewater treatment is to reduce the concentration of organic and inorganic nutrients discharged into the receiving stream. For public health reasons, it is also necessary to reduce the concentration of pathogenic organisms commonly found in domestic sewage. These organisms, generally bacteria and viruses, are causative agents for a number of diseases which include typhoid, cholera, poliomyelitis, hepatitis, and others.

The pathogenic organisms are always present in raw wastewater, but not necessarily in concentrations which readily permit detection and identification. Identification is further hampered by difficulties encountered in culturing host specific pathogens in an artificial environment. To overcome this problem indicator organisms, (microbes

which live in the alimentary tract of man) are used. The coliform bacteria are generally used as indicators of bacterial contamination. A. aerogenes is a soil organism; E. coli is fecal in origin. The tests for total coliforms will detect both of these organisms, but without differentation. The fecal coliform membrane procedure as outlined in section 909C of Standard Methods gives 93% accuracy for differentiating between coliforms from warm-blooded animals and coliforms from other sources.

The die-off rates of various enteric organisms have been studied extensively for stabilization ponds where algae-bacteria mixed cultures occur (53). It was found that enteric bacterial reductions closely paralleled that for pathogenic reduction, and that E. coli inhibited regrowth potential while pathogens such as Salmonella, Shigella, and Vibrio did not.

In addition to doubts as to true parallels between the die-off rates of coliform and pathogenic bacteria, the lack of evidence of direct relationship between removal rates for coliform compared with viruses and pathogenic protozoan cysts casts even more doubt on the advisability of using E. coli as a measure of fecal contamination (54).

The use of indicators for determining the presence of pathogenic viruses is not always reliable. Pathogenic viruses are so host specific that tissue culture techniques must generally be used to grow them. This procedure is beyond the capabilities of most sanitary laboratories. The minute quantities of a given virus present and the difficulties in concentrating them further complicates the process. Therefore, indicator organisms are frequently used. A number of viruses parasitic to E. coli (the E. coli bacteriophages) may be measured quantitatively. Attempting to draw a parallel between the die-off of E. coli bacteriophage and pathogenic viruses is complicated by the fact that a reduction of the host coliform will naturally result in a reduction of the bacteriophage which may not accurately reflect the kill rate of viruses in general.

Removal efficiencies for some pathogenic viruses in various water and waste treatment units have been measured (55). Primary settling removed only three percent of poliovirus I in three hours. Activated sludge removed 94 percent and activated sludge with lime coagulation removed 99.9 percent. One study (56) reported primary treatment removals of 33-67 percent of the viruses in twenty-four hours while another study (57) measured removals of 90 percent using chemical precipitation of raw wastewater influent and up to 99 percent removals were obtained with conventional activated sludge.

Several methods of effecting bacterial kill have been used. Nearly all installations use chlorine either in gaseous form or as hypochlorite (58). In addition to chlorine, bromine and iodine are also available as disinfecting agents. Ozone is available for use in waste treatment plants although it is not widely used. Recent work with ozone in combination with high frequency sound indicates that this may be a cost effective method of disinfection. A less practical method which has been employed is the use of ultraviolet light. The penetrating power of ultraviolet is only a few centimeters in clear water and is further reduced as the turbidity of the waste stream increases. This problem requires high energy ultraviolet sources, efficient suspended solids removals, long contact periods and thorough mixing in the contact basin.

Chlorine (either as elemental, Cl_2, or hypochlorite, OCl^-) has been the principal disinfecting agent for wastewater. The mechanism of chlorine disinfection has been studied at some length (59). In the case of Cl_2 the chemical reaction involved is

$$Cl_2 + H_2O \rightleftarrows HOCL + H^+ + Cl^-$$

$$HOCL \rightleftarrows H^+ + OCl^-$$

The quantity of HOCl and OCl^- is free available chlorine, with HOCl having a killing efficiency of 40 to 80 times that of OCl^-. The HOCl in turn reacts with ammonia in water to a succession of compounds called chloramines: $NH_3 + HOCl \rightleftarrows NH_2Cl + H_2O$

$$NH_2Cl + HOCl \rightleftarrows NHCl_2 + H_2O$$

$$NHCl_2 + HOCl \rightleftarrows NCl_3 + H_2O$$

The principal compounds formed are NH_2Cl and $NHCl_2$ and are known as combined available chlorine. These have some disinfecting power but are slow reacting. It is therefore desirable to minimize the formation of chloramines by minimizing the ammonia present.

Chloramines also have the characteristic of being long lived as compared to free available chlorine. When chlorinated wastewaters are discharged to a receiving stream the action of reducing agents and sunlight serve to rapidly deplete the free available chlorine whereas the combined available chlorine is more tenacious and may cause undesirable effects on in-stream organisms (60). It is of critical importance to see that no more than sufficient chlorine is applied for the requisite degree of disinfection and this should be applied in the most efficient manner. This puts demands on the engineering and operational aspects of effluent chlorination with regard to dosage rates, contact times, amperametric chlorine residuals, mixing, temperature effects, ammonia content and suspended solids removal (61).

There are two reasons why the present chlorination practice does not insure a biologically safe water or waste water (62). One is the shortcomings of the coliform test and the other is the presence of compounds in wastewater that interfere with chlorine. Suspended solids and BOD have long been considered important factors controlling the amount of chlorine necessary to obtain satisfactory disinfection (63). The study carried out on this aspect has shown that chlorination of settled trickling filter effluent is more efficient than chlorination of the unsettled effluent (64). This has been explained by the fact that suspended solids are thought to provide shelter from the chlorine for embedded organisms (65). Wastewater which has a high concentration of ammonia, sulfides, organics, inorganic reducing materials, or is highly alkaline, is difficult to disinfect (66). Low temperatures also decrease the efficiency of disinfection.

COMPLETELY MIXED KINETICS OF ACTIVATED SLUDGE TYPE SYSTEMS

A substantial research effort has been directed toward attaining an indepth understanding of the factors affecting the activated sludge process and other liquid culture aerobic processes. Significant progress has been made for systems operating under "equilibrium" conditions. Initially aeration basin design was based on the hydraulic retention time (HRT) and the mass of BOD_5 applied per 1000 cu ft of aeration basin volume. Later the food (organic matter) to microorganism ratio (F/M) was applied to the design of aeration basins indicating that researchers were aware of the importance of maintaining a controlled population of microorganisms in the basin (67,68). In the past several years Eckenfelder(69), McKinney (70), Lawrence and McCarty (71), and other engineering researchers have applied microbial continuous culture theory to wastewater treatment systems. The rational approaches to the design of biological wastewater treatment systems have been based on pure culture research efforts of Monod (72) who applied enzyme kinetics to pure culture systems. To develop equations which describe the wastewater treatment system requires the use of materials balance equations around system boundaries and a knowledge of process reactions. For a completely mixed process operating under steady state conditions, as shown in Figure 1, the following relationships were developed.

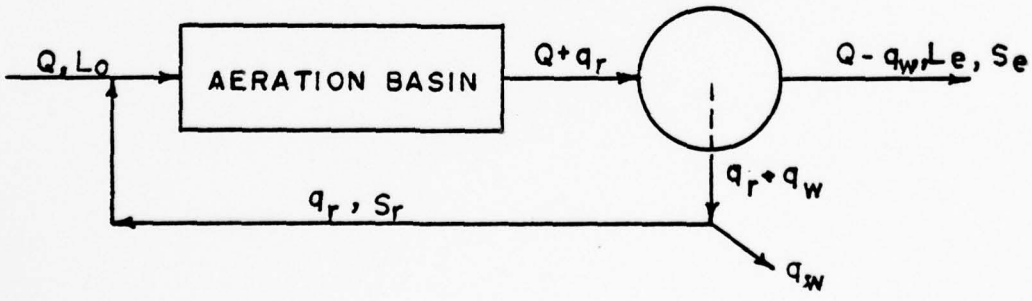

Figure 1. Completely Mixed Activated Sludge Process

Hydraulic Retention Time

$$HRT = \frac{V}{Q_T}$$

HRT = Hydraulic retention time

V = Aeration basin volume

Q_T = Influent volumetric flow rate plus recycle $(Q + q_r)$.

Sludge Retention Time

$$SRT = \frac{VS}{q_w S_r + (Q-q_w) S_e}$$

where

SRT = Sludge retention time

q_w = Solids wastage flow rate

S = Mixed liquor suspended solids concentration in the aeration basin

S_e = Suspended solids concentration in the effluent

S_r = Suspended solids concentration in the return sludge

$$\frac{ds}{dt} = a \frac{dL}{dt} - b\,S = S/SRT \qquad (3)$$

where

$\frac{dS}{dt}$ = Net solids produced

$\frac{dL}{dt}$ = Organic utilization rate

a = Volatile solids synthesised per unit BOD removed, called the sludge yield coefficient.

b = Volatile solids destroyed per unit of mixed liquor volatile solids, called the endogenous respiration coefficient.

Rate of Organic Removal

$$\frac{dL}{dt} = \frac{V_m L_e S}{k_s + L_e} \qquad (4)$$

where

$\frac{dL}{dt}$ = Organic utilization rate

V_m = Maximum rate of substrate utilization per unit weight of microorganisms

L_e = Substrate concentration

k_s = Velocity coefficient, equal to substrate concentration when $dL/dt = \frac{1}{2} V_m$

This equation is continuous over the entire range of substrate concentrations. In two extreme cases where the substrate concentration is very low ($k_s >> L_e$) or where it is very high ($L_e >> k_s$) two discontinuous relationships can be developed:

$$\frac{dL}{dt} = k\ L_e\ S \quad \text{and}$$

$$\frac{dL}{dt} = V_m\ S$$

where

$$k = \frac{V_m}{k_s}$$

These equations have been used by several researchers.

Effluent Concentrations

Solving the above equations for the effluent concentration (L_e) the following relationships have been developed.

For low organic concentrations:

$$\frac{L_o - L_e}{(HRT)} = K\ L_e\ S_a$$

$$\text{or } L_e = \frac{L_o}{1 + k\ S_a t} \qquad (5)$$

where

L_o = Influent substrate concentration

S_a = Average mixed liquor suspended (biomass) concentration

For high organic concentrations:

$$\frac{L_o - L_e}{(HRT)} = V_m\ S_a$$

$$L_e = L_o - V_m\ S_a (HRT) \qquad (6)$$

Based on the continuous Monod function, equation (4), and the equation describing the net solids production equation (3)

$$L_e = \frac{k_s \{1 + b\ (SRT)\}}{SRT\ (a\ V_m - b) - 1} \tag{7}$$

Microorganism Concentration

The following equation for the concentration of microorganisms in the reactor can be developed by rearranging equation (3).

$$S = \frac{a\ (L_o - L_e)}{1 + b\ (SRT)}\ \frac{SRT}{HRT} \tag{8}$$

For a flow through system with no sludge recycle (SRT = HRT) the equation reduces to

$$S = \frac{a\ (L_o - L_e)}{1 + b\ (HRT)} \tag{9}$$

Relationship Between HRT and SRT

By striking a material balance around the biological reactor alone a relationship between the solids concentration in the recycle (S_r), the recycle flow rate (q_r), the influent flow rate (Q), the reacter volume (V) and SRT can be developed. The material balance is

$$\frac{ds}{dt}\ V = (Q + q_r)S - q_r\ S_r \tag{10}$$

from which

$$\frac{1}{SRT} = \frac{1}{V}\left[(Q + q_r) - q_r\ \frac{S_r}{S}\right]$$

or

$$\frac{1}{SRT} = \frac{Q}{V}\left[1 + r - r\ \frac{S_r}{S}\right] \tag{11}$$

where $r = \frac{q_r}{Q}$

By returning to equation (10) and substituting S/SRT for ds/dt a relationship between HRT, SRT, Q, q_r, S, and S_r can be developed. The equation is:

$$HRT = SRT\left[1 - \left(\frac{q_r}{Q + q_r}\right)\frac{S_r}{S}\right]$$

For a system with no recycle ($q_r = 0$)

$$HRT = SRT$$

Relationship Between S_r and SVI

Assuming the settling basin is highly efficient the return solids concentration S_r can be correlated with the sludge volume index SVI.

$$S_r = \frac{10^6}{SVI} \tag{12}$$

Assuming the system is in equlibrium so that the biosolids produced is equal to that wasted then the term in equation (10)

$$\frac{ds}{dt} V = q_w S_r + (Q - q_w) S_e$$

Since $S_e << S_r$ then

$$(Q + q_r) S = S_r (q_r + q_w) \tag{13}$$

Combining equations (12) and (13) gives

$$S = \frac{10^6 (q_r + q_w)}{(Q + q_r) SVI} \tag{14}$$

If $q_r >> q_w$ then

$$S = \frac{10^6 (q_r)}{(Q + q_r) SVI}$$

or

$$S = \frac{10^6 r}{(1 + r) SVI} \tag{15}$$

where

q_r = Sludge recycle flow rate

q_w = Sludge waste flow rate

$r = q_r/Q$

All of the above equations have been developed for systems operating at equilibrium. Since systems rarely operate at equilibrium because of the natural variation in influent flows and concentrations it is imperative that methods be developed to insure the compliance with NPDES permits. To assure compliance engineers commonly are conservative in their design which increases the costs of the waste treatment

system and frequently results in decreased removal efficiencies as a result of excessive retention times in the clarifiers and aeration basins.

The principal reasons for failure of a system to operate as designed are:

1. Fluctuating flows
2. Fluctuating organic loading.
3. Unstable pH
4. Bulking or floating solids
5. Inadequate aeration and mixing
6. Inhibition by toxic chemicals

Long hydraulic retention times in aeration basins have been chosen in the past to "buffer" against transient or shock loads and possibly decrease the pH fluctuations. Increased retention times however frequently promote excessive algal growths (in aerated lagoon systems) and the formation of difficult to settle "pin floc" (in sludge recycle systems). Therefore, over-design of the aeration basin is not the answer. Over-design of clarifiers also frequently result in inefficient operation as a result of floating solids, anaerobic conditions, and algal growths.

During start-up systems are also operating under non-equilibrium conditions. The F/M ratio is high resulting in dispersed growths and poor settling characteristics. It is therefore essential that methods be developed which allow the operator to control the operational parameters which dictate the treatment efficiency.

Referring to the above equations these parameters are:

1. Cell concentration in the aeration basin which is a function of SRT, HRT and the mass organic loading rate.
2. Return solids concentration which is a function of the settling characteristics (SVI)

3. Sludge recycle and wasting rates
4. Sludge production rates

Methods developed for the purpose of controlling these parameters are reported in the following chapters.

METHODS FOR START-UP OF

ACTIVATED SLUDGE SYSTEMS

Two liter aeration units were used in the initial trials of the various start-up methods for activated sludge type systems because they permitted rapid assessment of a given technique under varying conditions. The more promising methods were then checked using 12-liter continuous feed fermenters. One method of start-up was to increase the mixed liquor volatile suspended solids content by bacterial seeding. A second method was organic enrichment of the wastewater to accelerate the growth of bacterial solids. The third method was the use of coagulants, weighting agents and adsorbants to decrease initial bacterial losses.

Bacterial Seeding

When seeding of activated sludge systems is employed for rapid start-up, consideration must be given to the bacterial density in the seed material and in the wastewater to be treated. In the experiments reported here, bacterial concentration was determined by suspending a measured amount of the materials tested in sterile buffer solution by using a Vortex Junior mixer. Mixed liquor from the Moore, Oklahoma, extended aeration plant was stirred to partially disperse the floc particles. All samples were plated on nutrient agar and incubated for 48 hours at 35°C.

The bacterial content of the plant effluent of industrial waste streams may be quite low. Furthermore, the types of activated sludge organisms such as Alcaligenes, Flavobacterium, Bacillus or Pseudomonus, would undoubtedly be present in even lower concentrations. For this reason it is common to add soil from the bacteria laden "A" horizon or mixed liquor from a nearby activated sludge plant when starting-up a new industrial system. The bacterial concentration of domestic wastewater on the other hand is much higher than for industrial wastes.

The analysis of the bacterial content of grab samples taken from the Norman, Oklahoma sewage treatment plant at various times is shown in Table 4. The average was 13 x 10^{10} aerobic or faculatively anaerobic bacteria per milliter. When assessed by the same method, the average bacterial concentration of the supplemental seeding materials is less than the concentration in raw wastewater, with the exception of the mixed liquor from the Moore sewage treatment plant (Table 5). From this information one would not expect the addition of seed to significantly affect the start up of municipal system but may be required to start-up an industrial waste treatment system. To check this hypothesis several experimental tests were performed. In the first test, varying amounts of soil from the dairy pasture and varying amounts of domestic wastes were added to six two-liter aeration chambers and aeration was begun. Initial conditions for the system are shown in Table 6. After six hours aeration, the chambers were coagulated with 150 mg/l of ferric chloride. Aeration was stopped, the mixed liquor was allowed to settle and the top 1.5 liters were siphoned off. The level in the chamber was returned to 2.0 liters by adding sewage of the same strength as that used in the initial start-up. This procedure was followed at six-hour intervals for 96 hours and the BOD, COD, and TOC of the clarified liquor and the MLVSS were measured. After 36 hours the ferric chloride dosage was reduced to 75 mg/l and in 72 hours it was discontinued since it was no longer needed for coagulation. Table 7 summarizes the MLVSS build-up and COD removal efficiencies. The addition of topsoil, did not greatly affect the MLVSS content and the COD removal. Assuming a one million gallon per day system with an eight hour hydraulic retention time, the 8 gm/l application of soil would require 22,000 lb, or roughly 6 cubic yards, of topsoil. The minimal improvement over an unseeded system does not justify the addition of topsoil.

The second start-up procedure tested dried bacterial cultures (commercially available Rid-X, for consumer septic tank application) as a seed material. The procedure for dried bacterial seeding was identical to

Table 4. AEROBIC & FACULTATIVE ANAEROBIC BACTERIAL POPULATION OF PRIMARY CLARIFIER EFFLUENT

Date	Time	BOD mg/l	COD mg/l	Bacteria per ml.
9-20-72	800	225	327	9.4×10^{10}
	1200	256	361	12.0×10^{10}
	1600	296	414	11.9×10^{10}
	2000	216	299	10.6×10^{10}
9-21-72	800	264	413	16.4×10^{10}
	1200	293	412	12.3×10^{10}
	1600	274	400	14.0×10^{10}
	2000	315	481	18.5×10^{10}

Table 5. BACTERIAL CONTENT OF SEEDING MATERIALS

Source	Bacterial Content
Soil from Dairy Farm	3.0×10^{10}/gm
Dried Sludge from Anaerobic Digester	1.5×10^{10}/gm
Septic Tank Additive	9.5×10^{10}/gm
Mixed Liquor from Moore, OK, Plant (fresh)	26.0×10^{10}/ml
Mixed liquor from Moore, OK, Plant (stale)	3.8×10^{10}/ml

Table 6. INITIAL CONDITIONS FOR SOIL SEEDED SYSTEM

Chamber	Soil Added (gm)	Initial BOD (mg/l)	Initial COD (mg/l)
1	0.0	196	324
2	0.5	193	330
3	1.0	204	335
4	2.0	215	344
5	4.0	214	365
6	8.0	256	412

Table 7. SUMMARY OF START-UP OF A SOIL SEEDED ACTIVATED SLUDGE SYSTEM

	Soil Added (gm/l)											
	0.0		0.5		1.0		2.0		4.0		8.0	
Time (hrs.)	MLVSS	COD	MLVSS	COD	MLVSS	COD	MLVSS	COD	MLVSS	COD	MLVSS	COD
0*	-	324	-	330	-	335	-	334	-	365	-	412
12	300	221	321	210	168	263	214	280	300	294	319	216
24	461	210	463	233	321	215	364	154	318	216	426	154
36	520	145	591	152	460	187	432	129	319	154	639	125
48	429	136	658	159	527	139	496	114	526	112	750	100
60	364	119	757	126	593	131	554	76	421	90	725	85
72	731	100	717	130	412	102	519	83	628	93	777	90

*COD value measured after soil addition

that of soil seeding except that only four chambers were used, with concentrations of 0.0, 0.25, 0.5, and 1.0 gm/l. It was felt that the cost of any dosage over 1 gm/l would be prohibitive even if effective. Assessed on this basis, the start-up for supplemental seeding with dried bacteria is ineffective in accelerating the start-up process (Table 8).

A third type of bacterial seeding used mixed liquor from the Moore, Oklahoma, extended aeration type treatment plant. The Moore plant at this time, September, 1972, was operating at a very high organic removal efficiency and the mixed liquor had excellent settling characteristics with a sludge volume index of 63. Mixed liquor was added to the two-liter chamber to equal 0, 5, 10, and 20 percent of the volume of the chamber. A measured value of 26×10^{10} organisms/ml increased the bacterial content of the sewage by roughly 16 percent. Even with this increase, seeding with mixed liquor from the Moore Sewage Treatment Plant did not enhance removal efficiency (Table 9).

Organic Enrichment

Since bacterial seeding of wastewater was not effective in decreasing the start-up time of activated systems it was necessary to test a second approach, the use of organic enrichment to accelerate the growth of bacterial solids. Initially, varying amounts of nutrient broth, without additional seeding material, were added to the raw sewage. The mixture was then settled, decanted, and refilled after 24 hours, and thereafter at six-hour intervals. For a wastewater containing a COD of 250 mg/l the MLVSS after 24 hours was 590 mg/l, compared with a MLVSS of 1530 for a wastewater containing 2000 mg/l. Thus, it is evident that organic enrichment enhances the growth of bacterial solids. The start-up operation using organic enrichment of wastewater worked well on a batch basis and was tried on a continuous feed basis using the microferm fermenters. In this instance, 12 liters of wastewater organically enriched with nutrient broth were aerated for 24 hours and coagulated chemically.

Table 8. SUMMARY OF START-UP USING RID-X AS SUPPLEMENTAL FEEDING

	Rid-X Added (gm/1)							
	0.0		0.25		0.5		1.0	
Time (hrs.)	MLVSS	COD	MLVSS	COD	MLVSS	COD	MLVSS	COD
0*	-	355	-	355	-	355	-	355
12	260	261	439	284	521	251	804	230
24	412	205	354	227	324	218	521	204
36	320	144	426	152	486	226	550	176
48	540	136	394	120	599	154	496	111
60	536	115	521	119	640	118	764	135
72	679	84	580	70	636	75	750	86

*COD value measured prior to Rid-X addition

Table 9. SUMMARY OF START-UP OF A MIXED LIQUOR SEEDED ACTIVATED SLUDGE SYSTEM

	Mixed Liquor added (% by volume)							
	0		5		10		20	
Time (hrs.)	MLVSS	COD	MLVSS	COD	MLVSS	COD	MLVSS	COD
0*	30	361	179	361	324	361	621	361
24	170	191	241	211	280	271	544	204
48	236	121	316	161	386	146	465	159
72	384	79	511	86	491	72	596	89

*Initial MLVSS after seeding with mixed liquor

Varying amounts of enrichment were used. The system was then operated on a continuous feed basis using unaltered raw sewage with sludge recycle.

The optimum enrichment dosage for nutrient broth was 3000 mg/l. At this level, after 24 hours aeration, the COD of the supernatant was about 300, approximating that of the unaltered sewage. A removal efficiency of 95 percent was realized by starting the continuous feed operation after 24 hours and allowing the system to operate as a conventional activated sludge system for an additional 72 hours.

On a practical basis, a plant system at start-up would be on line for a period equal to one retention time. Additional nutrients would then be added and the aeration tank would have to be bypassed for 24 hours. This alone, apart from the cost of nutrients, makes this start-up method objectionable . At 3000 mg/l it would be necessary to add approximately 8000 pounds of nutrient material. Soluble proteinaceous materials are very expensive; carbohydrate such as crude sugars are less so. The carbon to nitrogen ratio in the supplement would have to be similar to that in domestic wastewater to encourage the proper bacterial flora. In any case, the enrichment material cost would be in the thousands of dollars for a one-million-gallon per-day plant. Since this procedure is cost prohibitive for many systems a third approach was tried.

Coagulants and Weighting Agents

The third approach to securing a rapid start-up was to add coagulants and weighting agents. Coagulants and inorganic weighting agents were tested as a means of minimizing the wasting of newly formed bacterial solids. Bentonite clay was selected as the weighting agent because it is readily available, relatively economical, fairly easy to handle and widely used in many industries as a clarifying agent.

Bentonite forms a colloidal suspension with a negative surface charge. Because the bacterial solids of an activated sludge system also possess a negative electrical charge, coagulation of the bentonite-bacterial suspension with inorganic coagulants proved difficult. Large doses were

required for even minimal results. This necessitated the use of a highly cationic polyelectrolyte. A dosage rate of 10 mg/l of Calgon ST-260 caused a rapid coagulation of the suspension and left a clear supernatant.

The start-up procedure with the bentonite consisted of the following:

a. Fermentor was reactor filled to 12 liters with raw sewage
b. Bentonite was added and mixed until suspended
c. Calgon ST-260 was added, effecting rapid coagulation
d. Fermentor was placed on continuous feed with sludge clarification and recycle.

The optimum dosages for the above were 2 gm/l bentonite and 10 mg/l of ST-260. Several desirable effects of this procedure were observed: (1) A "synthetic floc" was formed immediately and normal operation began as soon as the reactor chamber filled, eliminating the need for bypassing during the solid buildup period, (2) a very clear effluent was produced indicating the retention of high percentage of bacterial solids, (3) a high suspended solids removal efficiency was immediately realized, and (4) the bentonite has the ability to adsorb dissolved organic material, further increasing the immediate removal efficiency of the system.

The use of bentonite and ST-260 proved to be the most effective method of start-up tested on the bench scale basis, consistently offering high immediate removal efficiencies and 95 percent COD removal after 72 hours. This would be well suited to small and portable systems such as military field installations. The method is also much more economical than the other methods tested. A million-gallon-per-day system with an eight-hour retention time would require 5500 pounds of bentonite and 20 pounds of coagulant at a cost of less than 200 dollars.

START-UP AND OPERATIONAL CHARACTERISTICS OF AN ACTIVATED SLUDGE SYSTEM UNDER VARYING ENVIRONMENTAL CONDITIONS

Waste treatment systems at Army installations may undergo radical changes in waste characteristics and loading rates. Therefore, it was desirable to determine the effects of changing environmental conditions. Twelve-liter, continuous flow models were used to determine the start-up and operational characteristics under varying environmental conditions. The method of start-up using coagulants and weighting agents, outlined in the preceeding section, was tested with the systems operating under conditions varying from low to high temperatures, low to high pH, and low to high hydraulic and organic loading rates. The results of these studies are reported below.

Varying Temperatures

An external refrigeration unit was connected to the fermentors to allow operation at various temperatures. With this unit and an internal heating unit, temperatures could be maintained within 0.5°C of the selected temperature. The start-up and operation of systems were performed at 5-, 10-, 15-, 20-, 25-, 30-, and 35°C. Wastewater from the Norman plant's north clarifier was added to the system and the heating and cooling units were connected. As soon as the temperature reached the desired level, 2 g/l of bentonite and 10 mg/l of cationic polyelectrolyte were added to the aeration basins. Continuous feed operation with sludge recycle was begun. The feed COD and TOC, effluent COD and TOC and MLVSS were measured at approximately 24-hour intervals. The percent organic removal efficiencies, organic loading rate in pounds per day of COD per 1000 ft^3 and the food to microorganism ratio (lb/day/lb MLVSS) were computed for these parameters and are shown in Tables 10 through 16. From these data it can be seen that removal efficiencies at 5°C were not satisfactory and the MLVSS remained low, reaching only approximately 500 mg/l at equilibrium. Operational problems were also encountered at this temperature. The polyelectrolyte coagulant was less effective and

Table 10. SUMMARY OF START-UP OF A MODEL ACTIVATED SLUDGE SYSTEM AT 5°C

Time (days)	COD of feed (mg/l)	COD of effluent (mg/l)	COD removed (%)	TOC of feed (mg/l)	TOC of effluent (mg/l)	TOC removed (%)	COD loading ($lb/1000\ ft^3$)	MLVSS (mg/l)	F/M ratio
0	415	161	56	83	31	63	77.0	--	--
1	294	190	54	63	34	59	54.7	270	2.49
2	421	117	62	104	13	64	78.4	294	1.80
3	460	105	75	115	32	69	85.7	316	3.00
4	296	101	78	63	23	80	54.3	417	2.58
5	314	56	81	75	11	83	58.7	384	1.89
6	216	82	74	41	18	76	40.2	516	1.35
7	394	30	86	91	8	80	73.6	474	1.20
8	417	63	84	109	14	85	77.4	394	2.52
9	216	88	79	46	10	79	40.2	486	2.01
10	318	63	71	71	12	74	59.0	471	0.96

Table 11. SUMMARY OF START-UP OF ACTIVATED SLUDGE MODEL AT 10°C

Time (days)	COD of feed (mg/l)	COD of effluent (mg/l)	COD removed (%)	TOC of feed (mg/l)	TOC of effluent (mg/l)	TOC removed (%)	COD loading ($lb/1000\ ft^3$)	MLVSS (mg/l)	F/M ratio
0	408	240	41	90	48	46	76.0	--	--
1	294	91	77	67	19	79	54.7	282	3.37
2	421	99	66	101	25	63	78.4	362	1.62
3	274	39	91	56	5	95	80.8	445	2.57
4	415	43	84	90	11	80	77.0	547	1.26
5	224	77	81	54	19	79	41.7	552	1.83
6	376	29	87	87	5	81	69.7	617	0.94
7	419	41	89	89	9	90	78.0	629	1.59
8	318	42	90	63	76	86	53.9	625	1.81

Table 12. SUMMARY OF START-UP OF ACTIVATED SLUDGE MODEL AT 15°C

Time (days)	COD of feed (mg/l)	COD of effluent (mg/l)	COD removed (%)	TOC of feed (mg/l)	TOC of effluent (mg/l)	TOC removed (%)	COD loading ($lb/1000\ ft^3$)	MLVSS (mg/l)	F/M ratio
0	418	255	69	101	31	69	72.1	--	--
1	276	88	79	64	16	84	47.9	480	2.06
2	376	36	87	78	10	84	65.3	521	1.38
3	421	45	88	97	7	91	73.1	634	1.57
4	318	34	92	79	10	90	55.2	763	1.52
5	219	19	94	44	2	97	37.8	804	1.11
6	196	20	91	40	4	91	33.9	936	0.64
7	441	6	97	111	4	90	76.5	1014	0.56
8	306	21	93	67	6	95	52.8	941	0.91

Table 13. SUMMARY OF START-UP OF MODEL ACTIVATED SLUDGE SYSTEM AT 20°C

Time (days)	COD of feed (mg/l)	COD of effluent (mg/l)	COD removed (%)	TOC of feed (mg/l)	TOC of effluent (mg/l)	TOC removed (%)	COD loading ($lb/1000\ ft^3$)	MLVSS (mg/l)	F/M ratio
0	249	54	78	61	19	69	46.5	--	--
1	282	37	85	66	15	75	52.3	590	1.08
2	344	28	91	81	7	91	73.6	715	1.06
3	455	28	92	93	5	94	84.7	537	1.76
4	497	30	94	104	9	90	44.5	852	1.50
5	313	47	91	68	5	95	58.6	860	1.57
6	292	19	94	67	2	97	54.2	998	0.88
7	347	26	91	81	7	90	64.9	975	0.82

Table 14. SUMMARY OF START-UP OF A MODEL ACTIVATED SLUDGE SYSTEM AT 25°C

Time (days)	COD of feed (mg/1)	COD of effluent (mg/1)	COD removed (%)	TOC of feed (mg/1)	TOC of effluent (mg/1)	TOC removed (%)	COD loading ($lb/1000\ ft^3$)	MLVSS (mg/1)	F/M ratio
0	224	70	69	51	18	64	41.6	--	--
1	196	49	78	40	11	78	36.3	467	1.12
2	337	27	86	75	6	85	62.4	694	0.73
3	313	24	93	71	8	89	58.6	517	1.82
4	347	28	91	77	10	89	64.4	667	1.28
5	320	21	94	71	13	86	59.5	936	1.04
6	416	13	96	94	8	91	77.4	1014	.98
7	219	29	93	51	8	91	40.7	939	1.24
8	315	13	94	64	6	91	58.6	956	0.64

Table 15. SUMMARY OF START-UP OF A MODEL ACTIVATED SLUDGE SYSTEM AT 30°C

Time (days)	COD of feed (mg/1)	COD of effluent (mg/1)	COD removed (%)	TOC of feed (mg/1)	TOC of effluent (mg/1)	TOC removed (%)	COD loading ($lb/1000\ ft^3$)	MLVSS (mg/1)	F/M ratio
0	473	212	63	110	45	59	88.1	--	--
1	294	36	92	67	14	87	54.7	370	3.54
2	421	51	83	90	11	84	78.4	521	1.39
3	274	35	92	75	9	90	50.8	482	2.40
4	415	35	87	89	11	85	77.4	547	1.31
5	224	27	94	45	9	90	41.2	707	1.65
6	280	25	89	62	4	91	51.8	721	0.83

Table 16. SUMMARY OF START-UP OF ACTIVATED SLUDGE MODEL AT 35°C

Time (days)	COD of feed (mg/1)	COD of effluent (mg/1)	COD removed (%)	TOC of feed (mg/1)	TOC of effluent (mg/1)	TOC removed (%)	COD loading ($lb/1000\ ft^3$)	MLVSS (mg/1)	F/M ratio
0	364	215	41	84	52	38	67.8	--	--
1	516	167	54	121	39	53	95.8	362	1.63
2	394	191	63	81	47	61	73.6	411	2.37
3	236	199	49	54	26	51	44.0	325	1.80
4	384	87	63	81	16	70	71.6	316	1.41
5	325	149	61	72	31	62	60.5	385	1.83
6	391	104	68	91	24	67	72.6	426	1.56
7	421	172	56	90	41	55	78.4	380	1.38

the dosage had to be increased from 10 mg/l to 25 mg/l and repeated at 12-hour intervals to maintain the floc in a satisfactory condition. The floc was apparently more viscous at lower temperatures than at high temperatures and caused problems with the small scale pumping system in the laboratory model.

These problems were not apparent at 10°C. The removal efficiencies were considerably better at 10°C than at 5°C, indicating an extremely low bioreaction rate at the lower temperature.

There was very little difference in operational conditions at 15-, 20-, and 25°C. In this temperature range the system reached equilibrium in less than four days and maintained an efficiency greater than 90%. At 30°C the removal efficiency approached the maximum after only one day but was not stable.

A more motile bacterial population predominated at this temperature and the efficiency of the sludge clarification system was reduced. This trend continued in the start-up and operation of the system at 35°C where sludge clarification was difficult and removal efficiencies were substantially lowered.

Figure 2 shows the COD removal efficiencies plotted against time for the start-up of the model system at five degree intervals for 5°C through 35°C. The similarity in operation at 15-, 20-, and 25°C is readily apparent as is the poor removal efficiency at 35°C.

The maximum removal efficiencies increased as operational temperatures increased, up to 30°C. At 35°C the maximum removal efficiency was considerably lower (68%) than at 30°C (95%).

The above data indicate that high treatment efficiencies can be achieved by activated sludge systems over a wide range of operating temperatures. Special design modifications will be required for areas where there is a very wide difference in mean summer and winter temperatures.

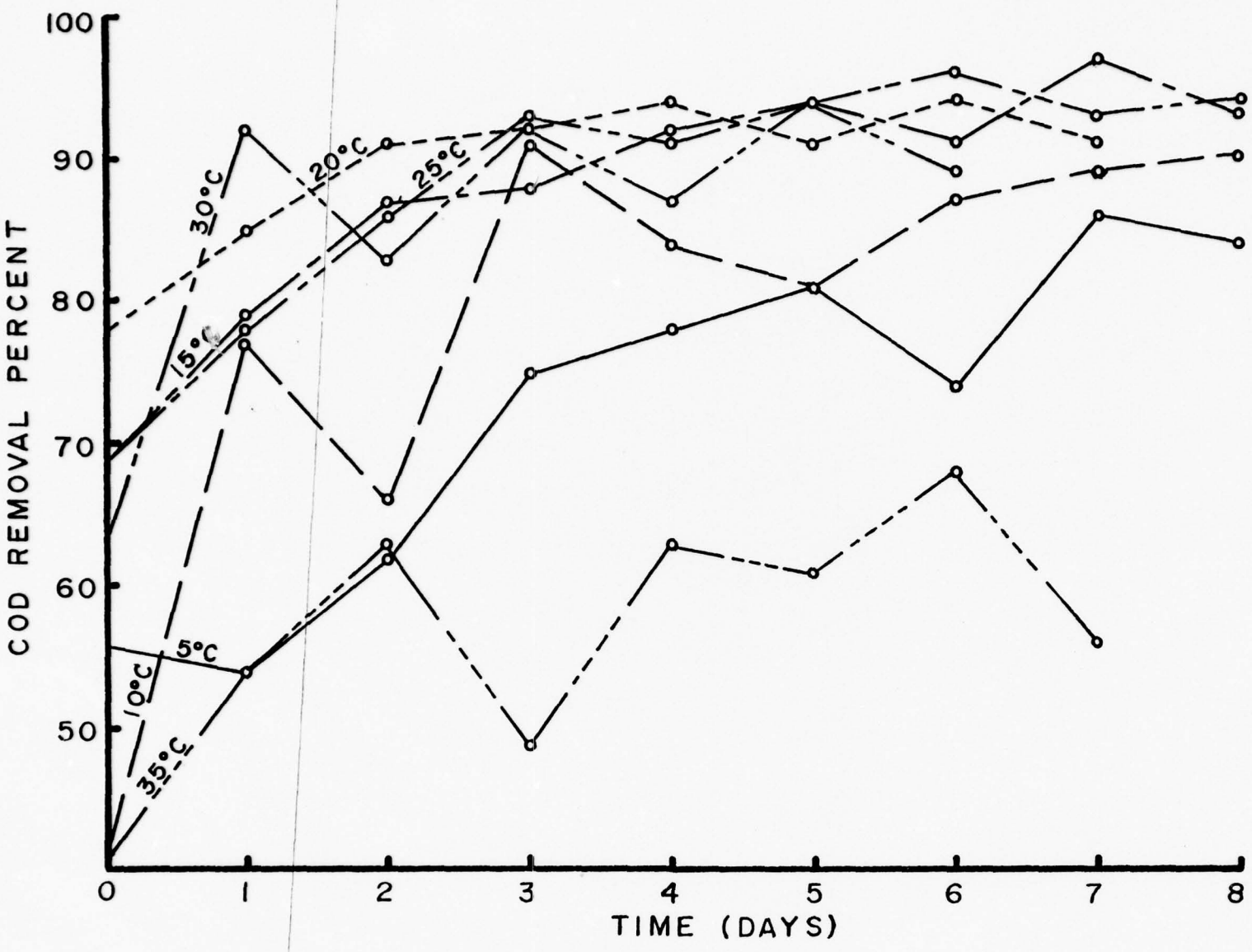

Figure 2. Effects of Temperature on COD Removal Efficiencies

Varying pH

The pH of municipal sewage tends to be lower than the pH of the domestic water supply. This is principally due to dissolved organic acids, carbon dioxide and hydrogen sulfide released through anaerobic decomposition of the wastewater in the collection system. The amount of pH depression is largely dependent on several factors, such as the alkalinity of the water supply and the age of the wastewater as it reaches the plant. Other factors such as industrial waste discharges may significantly alter the pH of sewage, especially in small systems. In a small military collection and treatment system, acid equipment washes or highly alkaline laundary wastes can substantially alter the operating pH.

Activated sludge systems normally operate over a pH range of 6-9, although pH values within that range must be fairly constant. Below this range sludge bulking is likely to occur, resulting in poor clarification and poor removal efficiencies. At a pH higher than 9, the stabilizing organisms, particularly the more complex protozoa, are inhibited, which also results in poor clarification and poor removal efficiencies.

To determine the effects of pH on the start-up characteristics of the activated sludge system, the system was operated at a constant hydraulic retention time (HRT) of 8 hours and a temperature of 20°C, 2 gm/l bentonite and 1 mg/l ST-260 was added to the system to promote settling and the pH of the wastewater was adjusted to values of 6,7,8, and 9. To adjust the pH of the collected waste, a sample of the waste was purged with air for 15 minutes to dispel hydrogen sulfide and readily volatile acids which would be driven off in an aeration system. Then a sufficient quantity of 12 N sodium hydroxide or sulfuric acid was added to produce the desired pH. This amount of sodium hydroxide or sulfuric acid was porportionally added to the daily wastewater feed.

At an influent waste pH of 6 a maximum COD removal efficiency of 89% was obtained. Maintaining a pH of 6 in the feed the pH of the system dropped steadily from a pH of 7.1 during the first day of operation to

6.8 after 5 days. At the end of this period the sludge began to bulk, and settling was very poor. The results of this study are given in Table 17 . Operation was improved by increasing the pH of the waste to 7 (Table 18). Bulking did not occur and the COD removal efficiency was over 90% within 4 days of start-up. The operation was essentially the same for the waste at a pH of 8 (Table 19), since the pH in the system was approximately 8 for the system while treating wastes at either an influent pH of 7 or 8. Less satisfactory results were obtained at a pH of 9 (Table 20). At this pH the protozoan population characteristic of a stabilized system did not develop. Free swimming ciliates were discouraged and only a few suctoria appeared after the second day. Figure 3 shows a summary of start-up for varied pH values.

Hydraulic Loading

The hydraulic loading rate on a domestic waste treatment system is a variable parameter. The variations are diurnal, with minimum early morning flows and mealtime peaks. There are also day to day variations in areas which have cyclic living habits and activities.

Peak flows will vary from two to three times the average flow rate. Minimum flow rates will be less than half the average. In the case of military installations with large portions of the populace retiring and rising simultaneously, the minimum and maximum flows are pronounced. A large number of family dwellings on a military base would decrease the ratio of minimum and maximum flows to the average.

There are conditions which vary the hydraulic loading rate from design or expected value, without significantly changing to organic loading rate. For example, during periods of limited water supply, such as in a drought during which use is sharply curtailed, the two principal sources of organics, feces and ground garbage, continue to be discharged to the collection system. Thus, the decrease in organic loading may not be proportionately as large as the decrease in hydraulic loading. During periods of wet weather, infiltration

Table 17 START-UP OF ACTIVATED SLUDGE SYSTEM (pH = 6)

Time (days)	COD of Feed	COD of Effluent	Percent Removal	lb/day / 1000 ft.^{3}	lb/day / lb. MLVSS	MLVSS mg/l	O_2 Uptake mg/l/hr.	pH in Reactor
0	224	97	57	42	---	---	---	---
1	196	54	76	36	1.8	327	8.0	7.1
2	232	38	81	45	1.9	365		7.2
3	337	44	81	63	2.1	478	14.4	7.1
4	351	37	89	65	1.6	660	13.6	6.9
5	306	48	86	57	1.3	693	11.4	6.8

Table 18 START-UP OF ACTIVATED SLUDGE SYSTEM (pH = 7)

Time (days)	COD of Feed	COD of Effluent	Percent Removal	lb/day / 1000 ft.^{3}	lb/day / lb. MLVSS	MLVSS mg/l	O_2 uptake mg/l/hr.	pH in Reactor
0	224	100	55	42	---	---	---	7.0
1	196	42	81	36	1.7	330	6	7.8
2	232	38	81	43	1.8	375	15	8.1
3	337	53	77	63	2.3	437	19	8.0
4	351	32	91	65	1.7	590	36	7.9
5	306	36	90	57	1.2	763	21	7.9

Table 19 START-UP OF ACTIVATED SLUDGE SYSTEM (pH = 8)

Time (days)	COD of Feed	COD of Effluent	Percent Removal	lb/day / 1000 ft.^{3}	lb/day / lb. MLVSS	MLVSS mg/l	O_2 uptake mg/l/hr.	pH in Reactor
0	320	141	56	60	--	---	---	8.0
1	320	39	88	60	1.9	512	13	8.3
2	291	68	79	54	2.8	493	14	8.2
3	287	44	85	53	1.4	613	18	8.1
4	313	30	90	58	1.1	836	16	8.1
5	349	29	91	65	1.2	910	---	8.1
6	198	35	90	37	.6	1050	19	8.2

Table 20 START-UP OF ACTIVATED SLUDGE SYYTEM (pH = 9)

Time (days)	COD of Feed	COD of Effluent	Percent Removal	lb/day / 1000 ft.^{3}	lb/day / lb. MLVSS	MLVSS mg/l	O_2 uptake mg/l/hr.	pH in Reactor
0	320	154	52	60	---	---	---	9.0
1	320	126	61	60	2.6	364	5	8.7
2	291	93	71	54	1.9	451	7	8.8
3	287	87	70	53	1.9	462	8	9.0
4	313	51	82	58	1.8	519	7	8.8
5	349	63	80	65	2.0	520	10	8.8
6	198	59	83	37	1.2	490	8	8.8

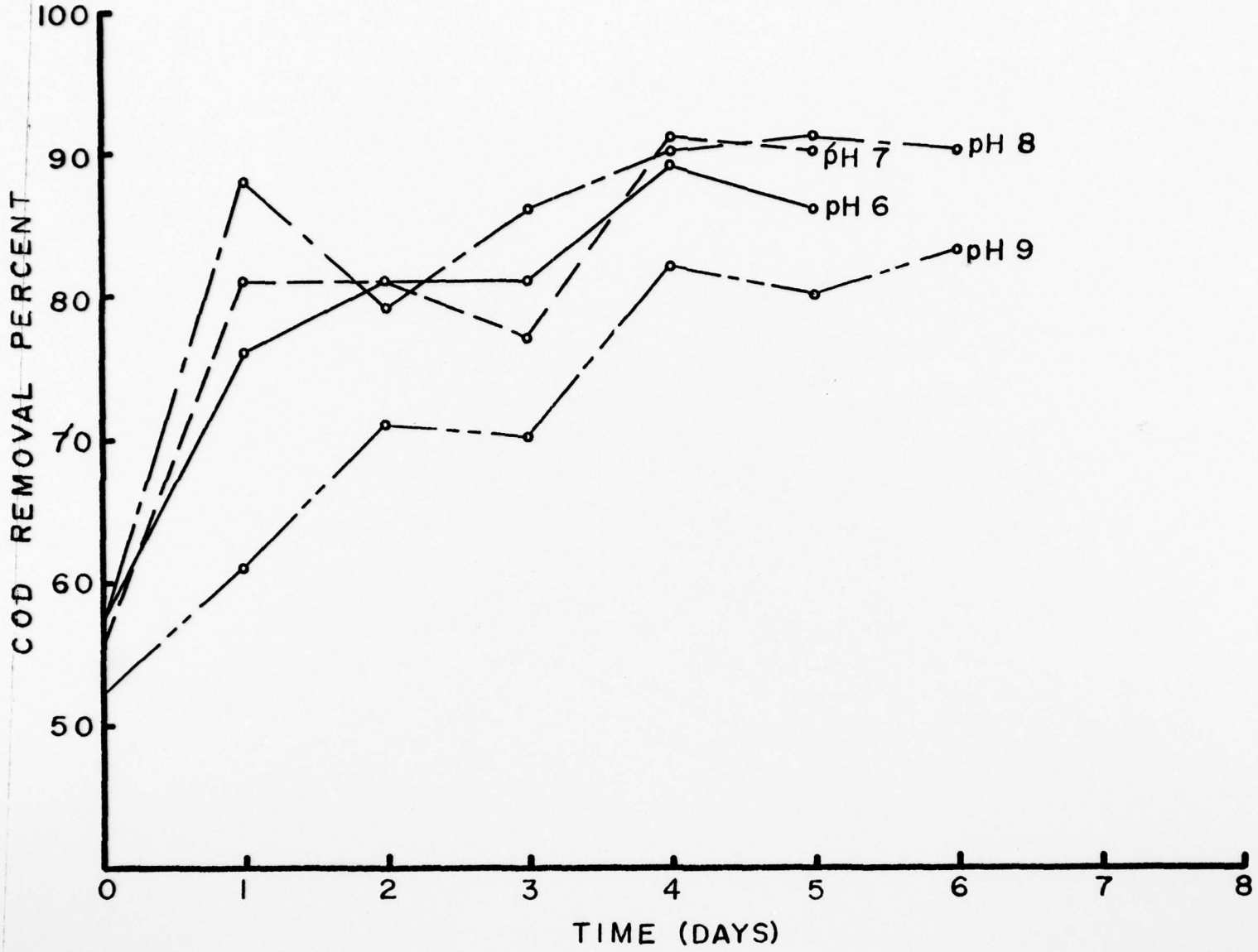

Figure 3. Effects of pH on COD Removal Efficiency

and inflow can greatly increase the hydraulic loading on a system especially one that is old and poorly maintained. Inflow is generally considered to be that storm water which enters a collection system through manholes, breaks and openings in the line, and combined storm and sanitary sewers. Inflow causes a rapid rise in sewage flow rates shortly after rainfall or when snow melt begins. Inflow ceases when surface runoff ceases. Infiltration is transportation of water through the soil to the collection system via leaking joints and cracks. It lags behind inflow by several hours and may last for several days. Although the first flush of inflow into a collection system may be quite high in suspended organic material, the organic content of other inflow and infiltration water will be considerably lower than the normal domestic load. A very high hydraulic loading rate can occur, then, without a large increase in organic loading.

The model activated sludge system was operated from start-up to test the conditions under varying hydraulic but constant organic loading rates. The previous hydraulic retention time for conditions tested was eight hours. Hydraulic retention times of 4-, 8-, 12-, and 16 hours were used in this study. An organic loading rate of 75 lb of COD/1000 ft^3/ day was selected for each run. To obtain this loading rate, the COD of the feedwater was adjusted to 200 mg/l for the 4-hour retention time, 400 mg/l for the 8-hour retention time, 600 mg/l for the 12-hour retention time and 800 mg/l for the 16-hour retention time.

The start-up remained the same with 2 g/l of bentonite and 10 mg/l of calgon ST-260. However, in this procedure the ST-260 was added periodically to maintain good floc settleability. Tables 21 through 24 summarize operations for the various hydraulic retention times tested.

The weakest point in an activated sludge system operating under hydraulic overload is in the clarifier, where a doubled flow rate creates a doubled overflow rate, possibly reducing the efficiency of the clarifier. This can be compensated for by maintaining the activated sludge

Table 21. SUMMARY OF START-UP OF ACTIVATED SLUDGE MODEL AT HYDRAULIC RETENTION TIME OF 4 HOURS COD OF FEED = 200

Time (days)	COD of Effluent (mg/1)	COD Removal (%)	MLVSS (mg/1)
0	104	48	216
1	93	54	354
2	75	63	531
3	31	85	517
4	26	87	520
5	27	87	487
6	24	88	621
7	20	90	815
8	19	91	764

Table 22. SUMMARY OF START-UP OF ACTIVATED SLUDGE MODEL AT HYDRAULIC RETENTION TIME OF 8 HOURS COD OF FEED = 400

Time (days)	COD of Effluent (mg/1)	COD Removal (%)	MLVSS (mg/1)
0	296	26	208
1	216	46	415
2	121	70	526
3	74	82	621
4	38	91	666
5	25	94	754
6	27	93	905
7	24	94	891

Table 23. SUMMARY OF START-UP OF ACTIVATED SLUDGE MODEL AT HYDRAULIC RETENTION TIME OF 12 HOURS COD OF FEED = 600

Time (days)	COD of Effluent (mg/l)	COD Removal (%)	MLVSS (mg/l)
0	504	16	264
1	392	35	318
2	278	54	499
3	145	76	521
4	76	87	784
5	94	84	861
6	40	93	936
7	29	95	1140

Table 24. SUMMARY OF START-UP OF ACTIVATED SLUDGE MODEL AT HYDRAULIC RETENTION TIME OF 16 HOURS COD OF FEED = 800

Time (days)	COD of Effluent (mg/l)	COD Removal (%)	MLVSS (mg/l)
0	649	19	351
1	421	47	596
2	364	55	781
3	398	60	754
4	276	66	908
5	142	82	848
6	100	88	1160
7	70	91	947
8	34	96	1240

such that the settleability of the floc is maximized. In the initial start-up the induced bentonite floc settled easily. As bacterial growth began, it was necessary to add dosages of the ST-260 polyelectrolyte at the rate of 10 mg/l every four hours to maintain the best floc settleability. In this way satisfactory clarification efficiencies were maintained throughout the test runs.

Referring to Tables 21 to 24 it is interesting to note that as the concentration of the influent increases simulating a decrease in flow rate (increased HRT) the effluent concentration increases. However, the total pounds of COD discharged per 1000 ft^3 of aeration basin volume decreases from 7.13 lbs/day at a 4 hour retention time to 3.2 lb/day at 16 hour retention time. This indicates that establishing effluent standards based on effluent concentrations is not conducive to encouraging water conservation. Within the limits of this study increased flow resulted in increased effluent quality. Based on the results of this study it is possible to change the hydraulic load by a factor of two, i.e. decrease the retention time from 8 to 4 hours or increase it to 16 hours without adversly affecting the efficiency of the system as long as the settling characteristics of the sludge is maintained; thereby enabling the clarifier to function efficiently.

Varying Organic Loading

The organic load imposed on a wastewater treatment system is a function of the flow rate and the organic concentration of the waste stream, both of which vary significantly if the waste is from a military base. The average concentration of wastes from both military and civilian establishments are given in Table 2, page 13 of this report. The organic concentrations are usually greater, the flows less and variation of both greater for a military base than for a municipality. Therefore, it was deemed desirable to determine the affects of varying the organic loading on start-up of and the effluent quality from a waste treatment system.

To assess the start-up characteristics of the activated sludge model under varying organic loading rates, several trial runs were made with a constant 8-hour hydraulic retention time, a temperature of 20°C, and COD concentrations of approximately 125, 250, 500, and 1000 mg/l respectively. A concentration of 125 mg/l represents light organic loading, 250 and 500 mg/l normal loading, and 1000 mg/l heavy loading for a municipal waste treatment system. To obtain the desired concentration wastewater was collected and analyzed for COD and TOC concentrations and then altered as necessary. If the waste collected had a higher COD value than desired, it was diluted with tap water. If a stronger waste was required, a sufficient amount of tryptose-glucose-yeast extract (TGY) broth was added. Start-up at each COD level was the same as in previously mentioned operations, which included the addition of 2 g/l bentonite and 10 mg/l of polyelectrolyte.

Removal efficiencies for a waste concentration of 125 mg/l COD never exceeded 80% and MLVSS build-up was slow, due to a limited organic concentration (Table 25). A COD removal of 80% was reached with a COD concentration of 25 mg/l, after only 7 days of operation. This concentration approaches the effluent guidelines of 20 mg/l BOD, established by the State of Oklahoma for streams that are not water quality limited. A waste concentration maintained at 250 mg/l COD resulted in an equilibrium removal rate of about 90%. At this strength, the effluent discharge would also be approximately 25 mg/l (Table 26). Feeding the model activated sludge system at 500 mg/l of COD increased COD concentration of the treated waste to 35 mg/l resulting in a 93% removal efficiency. These data are summarized in Table 27. Feeding the model system a waste with a COD adjusted to about 1000 mg/l, resulted in a removal efficiency of approximately 92-93% and an effluent COD concentration of around 70 mg/l. which would not meet effluent guidelines (Table 28).

Tables 25 through 28 show that although removal efficiencies rise with

Table 25 START-UP OF ACTIVATED SLUDGE SYSTEM (COD_I= 125)

Time (days)	*COD_I mg/l	*COD_E mg/l	Percent Removal	lb/day / 1000 ft^3	lb/day / lb MLVSS	MLVSS mg/l	O_2 Uptake mg/l hr
0	125	36	71	23.2			
1	123	39	69	22.9	1.27	290	9.4
2	131	38	69	24.4	1.21	324	10.9
3	127	34	74	23.6	0.92	416	8.3
4	116	30	76	21.6	0.71	493	9.9
5	120	26	78	22.3	0.68	526	10.6
6	123	25	80	22.9	0.74	499	11.4
7	132	26	80	24.5	0.71	554	16.2

*COD_I = COD of Influent

*COD_E = COD of Effluent

Table 26 START-UP OF ACTIVATED SLUDGE SYSTEM (COD_I= 250)

Time (days)	COD_I mg/l	COD_E mg/l	Percent Removal	lb/day / 1000 ft^3	lb/day / lb MLVSS	MLVSS mg/l	O_2 Uptake mg/l hr
0	240	76	68	44.6	2.25	320	8.8
1	248	63	75	46.1	1.78	418	9.6
2	261	52	79	48.5	1.47	534	14.3
3	255	47	82	47.4	1.06	716	18.1
4	250	28	89	46.5	0.91	824	15.6
5	244	26	90	43.4	0.78	934	13.9
6	236	29	88	44.0	0.61	1162	16.0
7	251	22	91	46.7	0.72	1040	17.0

Table 27 START-UP OF ACTIVATED SLUDGE SYSTEM (COD_I= 500)

Time (days)	COD_I mg/l	COD_E mg/l	Percent Removal	lb/day / 1000 ft^3	lb/day / lb MLVSS	MLVSS mg/l	O_2 Uptake mg/l/hr
0	490	196	60	91.1	3.71	396	12.6
1	505	78	84	93.9	2.46	616	18.4
2	516	56	89	96.0	1.90	814	19.3
3	512	39	94	95.2	1.68	916	21.4
4	503	38	93	93.6	1.41	1073	16.6
5	493	35	93	91.7	1.15	1284	25.8
6	481	39	92	89.5	1.11	1304	30.4
7	508	36	93	94.5	1.24	1233	28.6

Table 28 START-UP OF ACTIVATED SLUDGE SYSTEM (COD_I= 1000)

Time (days)	COD_I mg/l	COD_E mg/l	Percent Removal	lb/day / 1000 ft^3	lb/day / lb MLVSS	MLVSS mg/l	O_2 Uptake mg/l/hr
0	1010	491	52	187.9	6.37	476	24.9
1	1033	324	68	192.1	4.47	693	29.6
2	984	167	84	183.0	3.22	916	30.7
3	1031	107	89	191.8	2.97	1041	29.4
4	1005	96	91	186.9	2.10	1436	37.3
5	984	114	89	183.0	1.81	1634	39.0
6	1012	86	92	188.2	1.58	1920	43.1
7	1007	74	93	187.3	1.52	1983	44.0

organic loading rates, the quality of effluent decreases. This fact substantiates the results obtained in the proceeding section entitled "Varying Hydraulic Loading." For domestic waste treatment systems the permits are usually issued based on effluent standards. These standards generally approximate an organic concentration (BOD_5) of 20 mg/l. Based on these results cities and military bases may be inadvertantly encouraged to use more water to dilute their wastes or not to renovate their collection system to decrease infiltration and inflow. Based on these results it is possible to change the organic loading rate of an aerobic biological liquid culture system by a factor of 2 from normal loading of approximately 50 lbs/day/1000 ft^3 (250 mg/l at an 8 hour retention time) to either 25 or 100 lbs/day without significantly affecting the removal efficiencies or effluent quality. Increasing the organic loading by a factor of 4 to 200 lbs/day resulted in decreased effluent quality. These data are predicated on the availability of sufficient oxygen. Again referring to tables 25 through 28 it is immediately apparent that as the organic loading rate increases the oxygen uptake increases. The equilibrium oxygen uptake rate at initial COD concentrations of 125-, 250-, 500- and 1000 mg/l was 16.2-, 17.0-, 28.6- and 44 mg/l/hr respectively. Therefore, a system designed for an influent concentration of 250 mg/l would probably have inadequate aeration capacity to treat a waste with a concentration of 500 mg/l.

START-UP AND OPERATION OF AERATED LAGOON SYSTEMS

Aerated lagoon systems are commonly used to treat wastes from military bases. Military usage of aerated lagoons extends from tropical to artic locations with operation temperatures varying from freezing to over 30°C. Since the rate of biochemical reactions is dependent upon temperature it was deemed necessary to determine the best method for start-up and the time to reach equilibrium for aerated lagoons operating at various temperatures.

To perform this study a bench scale model consisting of a thirty gallon, glass-sided aquarium fitted with an air diffuser stone in the geometric center of the liquid was used. This permitted some mixing and proper aeration in the upper part of the tank and quiescent conditions in the lower part where sludge was allowed to accumulate and digest.

To start the bench scale model aerated lagoon system, two liters of wastewater were added to the basin daily. Aeration was begun at the time of the first wastewater addition. The cross-sectional dimensions of the tank were 320 x 74.8 cm. Two liters of water gave a depth of about 0.8 cm. This addition of wastewater was continued until the system reached equilibrium, or until the maximum organic removal was obtained. Water was then withdrawn from the tank at the rate of two liters per day. Water lost through evaporation was made up with distilled water.

The tank was fitted with aquarium heaters and a refrigeration coil to vary and control the basin temperature. By insulating the tank, temperature could be varied from 10° to 35°C. To compensate for heat loss when operating at higher than ambient temperatures, electric heating pads were placed under the tank until the water was deep enough to permit use of the aquarium heaters. Under these conditions, algal growth was at times so luxuriant that the effluent BOD was higher

than the influent BOD. To limit algal growth it was necessary to block off almost all available light.

The COD of the tank influent was measured daily, as was the COD of the effluent when wasting was begun. A cumulative average influent COD was calculated. The effluent COD was compared against the cumulative average to determine removal efficiencies.

The systems were run as stated above at temperatures of 10-, 15-, 20-, 25-, 30-, and 35°C. After equilibrium was reached the system was monitored for an additional two weeks. The data are recorded in Tables 29 to 33. Table 34 summarizes the removal efficiencies, the times required to reach 80 percent removal at different temperatures, and the corresponding loading rates based on retention times corresponding to the time to reach 80 percent removal.

The time to reach 80 percent removal, which approached equilibrium, ranged from 10 days at 30°C to 32 days at 10°C. It is therefore evident that the time to reach equilibrium from start-up and the retention time required to maintain a desired removal efficiency is dependent upon the operating temperature.

Start-up of an aerated lagoon system is dependent upon the design retention time and the operating temperature. For systems with retention times equal to or greater than those listed in table 34 the system can be started by filling the basin with waste to a level sufficient to start the aerators and then operate normally. By the time the system fills the efficiency will approach the design efficiency. Prior to the start of aeration odor may be detectable therefore, if water is available and the system is close to developed areas it may be necessary to fill the basin with water to a level which the aerators can be started and then start normal operation. For systems with retention times significantly less than those listed in table 34, start-up procedures similar to those outlined for activated sludge systems can be followed.

Table 29. SUMMARY OF START-UP OF AERATED LAGOON MODEL AT 10°C

Day	COD of feed (mg/l)	Cummulative mean COD (mg/l)	COD in tank (mg/l)	COD removal (%)	Day	COD of feed (mg/l)	Cummulative mean COD (mg/l)	COD in tank (mg/l)	COD removal (%)
1	362				31	264	347	83	76
2	319	340	300	12	32	388	349	73	79
3	258	313	263	16	33	245	345	52	85
4	412	338	270	20	34	461	349	49	86
5	387	347	270	22	35	362	349	56	84
6	449	364	270	26	36	318	348	59	83
7	296	355	256	28	37	554	354	57	84
8	410	361	250	31	38	316	352	49	86
9	312	256	231	35	39	186	348	45	87
10	196	340	207	39	40	425	351	49	86
11	523	357	221	38	41	419	352	60	83
12	346	356	234	34	42	354	352	46	87
13	329	354	205	42	43	368	353	67	81
14	417	358	168	47	44	296	352	53	85
15	316	356	175	51	45	315	351	42	88
16	284	351	154	56	46	195	348	38	89
17	261	345	152	56	47	264	346	38	89
18	394	348	141	59	48	264	344	45	87
19	390	351	129	63	49	219	342	34	90
20	354	351	123	65	50	308	341	34	90
21	417	353	116	67	51	411	342	41	88
22	361	354	110	69	52	196	339	41	89
23	311	352	109	69	53	153	336	40	89
24	204	346	110	68	54	318	336	34	90
25	316	345	97	72	55	357	336	40	88
26	412	348	97	72	56	362	337	44	87
27	355	348	89	74	57	333	336	37	89
28	387	349	73	79	58	394	337	30	91
29	414	351	88	75	59	341	336	40	88
30	310	350	102	71					

Table 30. SUMMARY OF START-UP OF AERATED LAGOON MODEL AT 15°C

Day	COD of feed (mg/1)	Cummulative mean COD (mg/1)	COD in tank (mg/1)	COD removal (%)	Day	COD of feed (mg/1)	Cummulative mean COD (mg/1)	COD in tank (mg/1)	COD removal (%)
1	324				31	306	316	48	85
2	344	333	283	15	32	242	311	40	87
3	216	294	217	26	33	300	311	62	80
4	239	280	193	31	34	348	312	53	83
5	454	315	208	34	35	365	314	47	85
6	329	317	206	35	36	352	315	38	88
7	388	327	199	39	37	392	317	41	87
8	512	350	203	42	38	208	314	50	84
9	349	350	189	46	39	164	310	40	87
10	216	337	172	49	40	290	310	34	89
11	176	322	148	54	41	349	311	34	89
12	354	324	132	59	42	383	312	31	90
13	296	324	126	61	43	408	315	32	90
14	224	314	116	63	44	473	318	38	88
15	216	308	102	67	45	261	317	28	91
16	346	310	105	66	46	225	315	25	92
17	248	307	80	74	47	418	317	44	86
18	290	306	67	78	48	369	318	35	89
19	386	310	65	79	49	283	318	32	90
20	340	312	72	77	50	614	324	32	90
21	402	316	66	79	51	391	325	29	91
22	327	316	63	80	52	484	328	32	90
23	313	316	75	76					
24	295	316	66	79					
25	191	310	62	80					
26	387	310	62	80					
27	417	314	56	82					
28	309	314	50	84					
29	294	314	50	84					
30	398	317	48	85					

Table 31. SUMMARY OF START-UP OF AERATED LAGOON MODEL AT 22°C

Day	COD of feed (mg/1)	Cummulative mean COD (mg/1)	COD in tank (mg/1)	COD removal (%)	Day	COD of feed (mg/1)	Cummulative mean COD (mg/1)	COD in tank (mg/1)	COD removal (%)
1	347				24	164	344	39	89
2	359	348	310	10	25	196	338	26	92
3	317	338	286	15	26	198	332	18	94
4	385	349	260	26	27	223	329	19	95
5	296	339	219	35	28	145	322	25	92
6	216	318	194	39	29	296	321	20	94
7	194	301	176	41	30	351	322	18	94
8	310	302	155	49	31	384	324	21	94
9	416	314	141	55	32	247	321	14	96
10	354	318	124	61	33	318	322	16	94
11	409	327	104	68	34	301	322	18	94
12	380	331	96	71	35	203	313	20	94
13	300	329	89	73	36	386	314	15	95
14	210	321	73	77	37	331	315	14	96
15	294	319	71	77	38	394	317	21	93
16	386	323	70	78	39	416	320	19	94
17	516	354	66	81	40	194	317	16	95
18	426	358	57	84	41	293	316	14	96
19	339	357	49	87	42	187	313	13	96
20	154	347	40	88	43	325	313	17	95
21	227	341	39	89	44	369	315	20	94
22	418	345	35	90	45	400	316	18	94
23	516	352	37	89					

Table 32. SUMMARY OF START-UP OF AERATED LAGOON MODEL AT 30°C

Day	COD of feed (mg/1)	Cummulative mean COD (mg/1)	COD in tank (mg/1)	COD removal (%)	Day	COD of feed (mg/1)	Cummulative mean COD (mg/1)	COD in tank (mg/1)	COD removal (%)
1									
2	321	353	317	10	24	196	309	18	94
3	163	289	200	31	25	304	308	23	93
4	482	337	171	49	26	293	308	16	95
5	354	341	157	54	27	311	308	17	95
6	195	316	131	58	28	496	315	19	94
7	216	302	91	70	29	293	314	20	94
8	386	313	79	75	30	225	311	15	95
9	249	306	64	79	31	279	310	18	95
10	200	295	60	80					
11	193	286	61	79					
12	486	302	54	82					
13	358	307	58	81					
14	390	312	45	86					
15	324	313	39	87					
16	222	307	40	87					
17	160	299	33	89					
18	284	303	31	90					
19	341	305	26	91					
20	362	308	16	95					
21	370	311	19	94					
22	419	316	21	93					
23	264	314	14	96					

Table 33. SUMMARY OF START-UP OF AERATED LAGOON MODEL AT 35°C

Day	COD of feed (mg/l)	Cummulative mean COD (mg/l)	COD in tank (mg/l)	COD removal (%)	Day	COD of feed (mg/l)	Cummulative mean COD (mg/l)	COD in tank (mg/l)	COD removal (%)
1	416								
2	382	399	383	4	24	321	344	37	89
3	219	339	288	15	25	349	345	33	90
4	357	344	264	23	26	308	343	26	92
5	614	397	266	33	27	290	341	34	90
6	314	387	216	44	28	164	335	37	89
7	196	357	196	45	29	385	336	45	87
8	414	364	167	54	30	216	332	40	88
9	386	366	160	56	31	340	333	36	89
10	394	369	147	60	32	168	327	31	91
11	316	364	145	60	33	136	322	26	92
12	357	363	138	62	34	184	317	30	91
13	193	351	119	66					
14	257	344	110	68					
15	368	342	84	75					
16	321	341	86	75					
17	319	342	71	79					
18	385	345	59	83					
19	412	348	56	84					
20	219	342	44	73					
21	263	338	38	89					
22	505	346	31	91					
23	318	331	35	89					

Table 34. SUMMARY OF START-UP RESULTS FOR A MODEL AERATED LAGOON

Temperature (^{o}C)	Time to reach 80% removal (days)	Loading Rate $\frac{\text{lbs COD}}{1000\ ft^3}$	Maximum Removal Efficiencies (%)
10	32	0.07	89
15	25	0.09	90
22	17	0.13	95
30	10	0.22	95
35	18	0.12	90

SHOCK LOADING OF ACTIVATED SLUDGE SYSTEM OPERATING AT EQUILIBRIUM CONDITIONS

Studies were performed to determine the effects of transient hydraulic loads on a previously equilibrated system by using wastewater adjusted to a fairly constant organic concentration (COD near 300 mg/1), diluting it by a desired dilution factor and then feeding it to the equilibrated system at a hydraulic retention time of eight hours divided by the dilution factor. This way a constant organic loading rate was maintained while the hydraulic loading rate varied. Dilution factors of 1.5, 2, and 3 were used.

In this procedure two units were run in parallel under similar equilibrium conditions. One system was treated at hydraulic overload to obtain a better effluent and the other was not. The treatment method was the periodic addition of the cationic polyelectrolyte, Calgon ST-260, at a rate of 10 mg/l of reactor volume at a time interval of one hydraulic retention time of the system.

The control unit (unit "a") with a hydraulic load of 1.5 times the normal load, had a slightly reduced organic removal efficiency. The removal rate expected at normal loading rates was 91 to 94 percent, based on removals already reported. The removal rate at the 1.5 times the normal hydraulic load was 86-90 percent.

A second unit (unit "b"), while being subjected to the same hydraulic overload as unit "a" received a 10 mg/l dose of the cationic polyelectrolyte Calgon ST-260 every 6 hours. The organic removal efficiency of unit "b" stayed near the level reached prior to hydraulic overload, indicating that the addition of the polymer would be an effective corrective measure.

At a hydraulic overload factor of two, the organic removal efficiency of unit "a" was reduced even more to 76-84%. The addition of the polymer to unit "b" resulted in a removal efficiency only slightly less than

before the transient overload. The polymer in this case was added at four-hour intervals and improved the settleability of the sludge to a degree that the greatly increased hydraulic load had little effect on the loss of biosolids. In fact, the MLVSS was only slightly depressed during the overload.

At a hydraulic overload factor of three the changes were much more dramatic. Solids washout increased. In unit "a", the MLVSS was reduced from 1096 mg/l to 659 mg/l in 72 hours. Organic removal efficiency dropped from 92 to 63 percent, and then rose to 75 percent.

The addition of 10 mg/l of the polymer to unit "b" improved the operation, but removal efficiency decreased from 91 to 80 percent. However, the MLVSS was maintained at a fairly constant level.

These results indicate that a transient hydraulic load of two times the design flow is not detrimental to the system if the solids settling rates are increased consistant with the increase in overflow rates. This can be accomplished by the addition of a cationic polyelectrolyte. A transient hydraulic load of three times the design flow decreases the efficiency significantly.

Organic Overloading of Equilibrated Systems

The operation of an activated sludge type system under organic overload conditions was investigated by shock-loading a system operating at equilibrium. An organically enriched feed was used to simulate shock-loading. The 12-liter model systems were maintained at an eight-hour hydraulic retention time and the wastewater feed was supplemented with nutrient broth. Organic overload factors of 1.5, 2, and 3 were simulated.

The operation at an organic overload factor of 1.5 was not significantly different from that of the preceding equilibrium operation. The COD removal efficiency increased slightly, and the COD of the effluent

remained fairly constant. There was a slight rise in the MLVSS concurrent with increased organic loading.

An organic overload factor of 2 resulted in a concomitant rise in MLVSS while maintaining a high COD removal efficiency of 96-97 percent. The sludge produced was readily settled in the clarifier and solids washout was not a problem.

Very high removals were also realized at an organic overload factor of three. The effluent COD was, however, quite high and certainly would not meet the EPA NPDES effluent standards of 20 mg/l BOD. The above data indicate the bench model system was very resilient and was not drastically affected by either hydraulic or organic shock loads. To apply the results to a full scale system it must be realized that the model had very high mixing and aeration capacities and consequently was capable of meeting the demands put upon it by increased solids and oxygen requirements. It is unlikely that a plant scale system would have this capacity for excess aeration and mixing. Therefore, the magnitudes of the transient loads which may cause undesirable conditions may be less for a full scale system than for the model.

EFFLUENT CHLORINATION UNDER NON-EQUILIBRIUM CONDITIONS

In the past, chlorination for reduction of bacterial levels of raw or primary level wastewater discharges was a common practice. Effluents from secondary waste treatment systems were rarely chlorinated. Recent standards established by the U.S. Environmental Protection Agency for fecal coliform discharge (generally set at 200 fecal coliforms/100 ml) requires some form of disinfection prior to discharge.

Principally due to the universal acceptance of chlorine in the disinfection of potable water, most of the disinfection of wastewater effluents is being done with the use of chlorine. The equipment and technology for wastewater chlorination is readily available, and most municipalities have experience in chlorination.

There are many ways in which potable water and wastewater chlorination differ. The amount and variety of chemically active residual materials in secondary quality effluent far exceeds that of potable water. Ammonium compounds capable of forming mono-, di-, or trichloramine compounds may be present in abundant quantities in wastewater but not in potable water. Particulate matter capable of protecting a large portion of the fecal coliforms from the disinfecting agent may also be present in wastewater. The level of fecal coliforms in wastewater effluent is far higher than that allowed in a raw water supply. The consequences of inadequate disinfection of effluents are potentially much less severe than the consequences of inadequate water supply disinfection.

Several factors affect the disinfection capabilities of chlorine. These include chlorine dosage, temperature, pH, contact time and interfering chemicals. It is therefore desirable to determine the efficiency of chlorination as a function of these factors to assure adequate disinfection, to minimize costs and to prevent possible discharge of toxic chlorine residuals into the receiving stream.

The chlorine requirements for bacterial reduction of effluents from a well-operated secondary wastewater discharge have been studied. Chlorine requirements of effluents for systems not in equilibrium have not been determined. It is the purpose of this part of the research study to assist in the development of chlorination technology for effluents from systems not in equilibrium. To accomplish this, samples of effluents produced under the selected non-equilibrium states discussed in previous chapters were chlorinated to varying degrees. The effect of chlorination on the fecal coliform level of the samples was then noted.

Test Procedures

The chlorination tests were performed during start-up of the model activated sludge system under varying temperature, organic loading and hydraulic loading conditions. One-liter samples of the effluent were taken from the units operating at varying temperatures as explained on page 46 of this report. These samples were maintained at the operating temperature, divided into 100 ml units and a sufficient amount of a calcium hypochlorite solution was added to obtain chlorine dosages of 0, 1, 2, 4, 8, 16, 32, and 64 mg/l. After 20 minutes, sodium thiosulfate was added to deactivate the chlorine residual and a fecal coliform test was performed using the membrane filter method as outlined in Standard Methods for the Examination of Water and Wastewater. All membrane-filter-fecal-coliform tests were performed in duplicate and the average reported in Tables 35 to 41.

Chlorine requirements to meet a standard of 200 fecal coliforms per 100 ml of effluent reported in Table 42 range from 8 to 32 mg/l. The requirements are both temperature and concentration dependent.

Trends in chlorine effectiveness can be detected from the results. First as the system approached equilibrium the chlorine dosage required to meet a standard such as 200 fecal coliforms/100 ml decreases. This was expected since the chlorine demand decreases as the concentration of the organics and ammonia decrease to a minimum at equilibrium.

Second, as the temperature increases from 5 to 30°C the required dosage decreases, since the chlorine reaction rates increase with temperature. At 35°C however, the trend reverses. This is probably due to higher concentrations of fecal coliforms at 35°C than that at lower temperatures due to the more favorable environmental conditions or to higher COD's in the effluent which increases the chlorine demand and concomitantly decreases the efficiency for disinfection. Temperatures of 35°C and above may also enhance the escape of chlorine to the atmosphere.

Table 35. SUMMARY OF EFFECTS OF CHLORINATION OF EFFLUENT PRODUCED BY AN ACTIVATED SLUDGE MODEL OPERATING AT 5°C, OPERATING FROM START-UP

Time (days)	COD of Effluent (mg/l)	COD Removal Efficiency (%)	Fecal Coliform in Effluent After Indicated Dosage (no./100 ml)						
			0 mg/l	2 mg/l	4 mg/l	8 mg/l	16 mg/l	32 mg/l	64 mg/l
0	161	56	110,000	97,000	46,000	3,900	560	120	31
1	190	54	54,000	26,000	34,000	11,000	1,000	26	35
2	117	62	76,000	54,000	31,000	3,600	1,100	52	100
3	105	75	34,000	13,000	6,400	3,400	110	54	6
4	101	78	119,000	57,000	2,100	2,600	520	34	12
5	56	81	86,000	21,000	8,600	390	26	18	26
6	82	74	29,000	26,000	3,600	190	120	6	39
7	30	86	77,000	84,000	870	21	3	0	0
8	63	84	21,000	1,200	130	44	21	11	7
9	88	79	36,000	17,000	1,200	100	21	7	2
10	63	81	62,000	58,000	1,600	56	94	87	2

Table 36. SUMMARY OF EFFECTS OF CHLORINATION OF EFFLUENT PRODUCED BY AN ACTIVATED SLUDGE MODEL OPERATING AT 10°C, OPERATING FROM START-UP

Time (days)	COD of Effluent (mg/l)	COD Removal Efficiency (%)	Fecal Coliform in Effluent After Indicated Dosage (no./100 ml)						
			0 mg/l	2 mg/l	4 mg/l	8 mg/l	16 mg/l	32 mg/l	64 mg/l
0	240	41	86,000	76,000	61,000	24,000	3,900	618	214
1	91	77	49,000	38,000	16,000	1,200	220	41	28
2	99	66	37,000	30,000	18,000	1,000	361	8	45
3	39	91	62,000	58,000	39,000	6,200	390	14	2
4	43	84	59,000	63,000	6,200	1,100	660	290	14
5	77	81	21,000	12,000	11,000	860	110	40	0
6	29	87	31,000	14,000	1,500	67	621	14	20
7	41	89	20,000	1,700	600	200	36	9	0
8	42	90	13,000	1,400	800	100	21	13	6

Table 37. SUMMARY OF EFFECTS OF CHLORINATION OF EFFLUENT PRODUCED BY AN ACTIVATED SLUDGE MODEL OPERATING AT 15°C, OPERATING FROM START-UP

Time (days)	COD of Effluent (mg/l)	COD Removal Efficiency (%)	Fecal Coliform in Effluent After Indicated Dosage (no./100 ml)						
			0 mg/l	2 mg/l	4 mg/l	8 mg/l	16 mg/l	32 mg/l	64 mg/l
0	418	69	46,000	32,000	1,600	380	43	8	0
1	276	79	39,000	21,000	3,200	670	120	360	41
2	376	87	86,000	75,000	8,400	2,100	390	42	11
3	421	88	14,000	15,000	1,300	150	61	3	0
4	318	92	13,000	4,400	1,600	340	35	18	2
5	219	94	26,000	6,000	2,000	1,100	96	20	0
6	196	91	41,000	41,000	13,000	1,800	21	3	0
7	441	97	3,800	14,000	120	360	88	12	3
8	306	93	19,000	14,000	1,100	630	26	7	1

Table 38. SUMMARY OF EFFECTS OF CHLORINATION OF EFFLUENT PRODUCED BY AN ACTIVATED SLUDGE MODEL OPERATING AT 20°C, OPERATING FROM START-UP

Time (days)	COD of Effluent (mg/1)	COD Removal Efficiency (%)	Fecal Coliform in Effluent After Indicated Dosage (no./100 ml)						
			0 mg/1	2 mg/1	4 mg/1	8 mg/1	16 mg/1	32 mg/1	64 mg/1
0	54	78	100,000	34,000	6,200	360	36	4	0
1	37	85	46,000	21,000	3,400	890	214	16	3
2	28	91	37,000	26,000	4,100	1,100	61	14	2
3	28	92	41,000	24,000	2,800	220	18	29	15
4	30	94	27,000	18,000	1,900	64	11	4	8
5	47	91	20,000	9,000	1,600	860	42	61	0
6	19	94	19,000	12,000	1,400	420	13	9	0
7	26	91	31,000	17,000	1,100	550	110	86	2

Table 39. SUMMARY OF EFFECTS OF CHLORINATION OF EFFLUENT PRODUCED BY AN ACTIVATED SLUDGE MODEL OPERATING AT 25°C, OPERATING FROM START-UP

Time (days)	COD of Effluent (mg/l)	COD Removal Efficiency (%)	Fecal Coliform in Effluent After Indicated Dosage (no./100 ml)				
			0 mg/l	2 mg/l	4 mg/l	8 mg/l	16 mg/l
0	70	69	86,000	36,000	1,800	120	14
1	49	78	49,000	21,000	1,400	80	3
2	27	86	37,000	29,000	690	47	16
3	24	93	54,000	18,000	800	62	29
4	28	91	51,000	26,000	290	34	6
5	21	94	43,000	34,000	1,900	111	44
6	13	96	39,000	16,000	2,200	49	181
7	29	93	52,000	25,000	3,600	16	34
8	13	94	46,000	32,000	1,800	120	3

Table 40. SUMMARY OF EFFECTS OF CHLORINATION OF EFFLUENT PRODUCED BY AN ACTIVATED SLUDGE MODEL OPERATING AT 30°C, OPERATING FROM START-UP

Time (days)	COD of Effluent (mg/l)	COD Removal Efficiency (%)	Fecal Coliform in Effluent After Indicated Dosage (no./100 ml)				
			0 mg/l	2 mg/l	4 mg/l	8 mg/l	16 mg/l
0	212	63	36,000	20,000	900	54	3
1	36	97	37,000	17,000	260	180	9
2	51	83	21,000	8,000	540	150	0
3	35	92	43,000	3,800	300	26	14
4	35	87	8,000	9,400	290	38	26
5	27	94	20,000	8,600	1,100	170	100
6	35	89	14,000	1,700	600	240	8

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Table 41. SUMMARY OF EFFECTS OF CHLORINATION OF EFFLUENT PRODUCED BY AN ACTIVATED SLUDGE MODEL OPERATING AT 35°C, OPERATING FROM START-UP

Time (days)	COD of Effluent (mg/l)	COD Removal Efficiency (%)	Fecal Coliform in Effluent After Indicated Dosage (no./100 ml)				
			0 mg/l	2 mg/l	4 mg/l	8 mg/l	16 mg/l
0	215	41	130,000	94,000	4,100	360	29
1	167	54	160,000	120,000	10,000	150	7
2	191	63	89,000	61,000	5,400	68	11
3	199	49	100,000	76,000	11,000	120	28
4	87	63	64,000	54,000	7,600	1,100	3
5	149	61	86,000	21,000	500	64	96
6	104	68	75,000	58,000	3,800	250	41
7	172	56	110,000	62,000	14,000	180	60

TABLE 42 . REQUIRED RANGE OF CHLORINE DOSAGE FOR DISINFECTION OF EFFLUENTS FROM ACTIVATED SLUDGE SYSTEMS AT VARYING TEMPERATURES.

Temperature of Operation (^{o}C)	Time from Start-up (days)	Chlorine Dosage Requirements (mg/l)
5	0 - 4	16 - 32
	5 - 10	8 - 16
10	0 - 5	16 - 32
	6 - 8	8 - 16
15	0 - 2	16 - 32
	3 - 8	16
20	0 - 7	16
25	0 - 8	8
30	0 - 5	8
35	0 - 7	16

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