AERATED LAGOON TECHNOLOGY

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Aerated lagoon technology, especially that of high-performance systems, is one of the most misunderstood technology in wastewater treatment. This misunderstanding is largely the result of its evolution from the technology of facultative lagoons, in which algae play a vital role and hydraulic retention times are long. In fact, the technology of high-performance aerated lagoons has much in common with that of activated sludge. With proper design and operation, aerated lagoons can deliver effluents that meet limits of 30 mg/L, both for TSS and $CBOD_5$. Furthermore, with modification or with the addition of low-tech process units, they can be designed to nitrify. The major advantages of aerated lagoon systems are their low cost and their minimal need for operator attention.

The performance of aerated lagoon systems, as well as the diagnosis and remedies of their operational problems, will be the focus of a series of technical notes that will appear on this website. The notes should be of considerable value both to engineers and operators. The development of the technical notes will be a continuing activity on the part of the author and will be added to the list on this page. The notes presently available are below.

TECHNICAL NOTES

Technical Note Number 1

EFFLUENT BOD5 – A MISLEADING PARAMETER FOR THE PERFORMANCEOF AERATED LAGOONS TREATING MUNICIPAL WASTEWATERS

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In spite of the fact that effluent BOD_5 is a key parameter in many discharge permits for aerated lagoons, it is the most misleading. Most effluent BOD_5 data are flawed as the result of being inflated by nitrification that occurs in the BOD_5 test itself. It has been reported that as many as 60 percent of the BOD_5 violations nationally may have been caused by nitrification in the BOD_5 test rather than by improper design or operation (Hall and Foxen 1983). Consequently, millions of dollars may have been spent needlessly on new treatment facilities.

The total BOD of a wastewater is composed of two components – a carbonaceous oxygen demand and a nitrogenous oxygen demand. Traditionally, because of the slow growth rates of those organisms that exert the nitrogenous demand, it has been assumed that no nitrogenous demand is exerted during the 5-day BOD_5 test. Although, such assumption is valid when the test is performed on untreated municipal wastewaters, it is not valid when performed on secondary effluents, especially those from aerated lagoons. The BOD_5 of effluents from the latter are almost always inflated by a nitrogenous component. Moreover, unlike the carbonaceous demand which is proportional to the concentration of the biodegradable carbon constituents in the effluent, the nitrogenous demand exerted during the 5-day test is proportional to the number of nitrifying organisms that happen to be caught in the sample being tested. Thus the argument that the test provides insight on the impact that the effluent will have on the receiving water can not be defended. Neither can the practice of making waste-load allocations from models that contain both a BOD_5 (assumed to be a measure of the carbonaceous demand) and a nitrogenous demand.

The severity of the problem is illustrated in Figs. 1 and 2. Figure 1 compares the effluent BOD_5 with the $CBOD_5$ (carbonaceous component of the BOD_5). The $CBOD_5$ is determined by using a nitrification suppressant in the BOD_5 test. Figure 2 compares the two parameters in filtered samples. Note should taken of the magnitude of the nitrification factor in the 5-day test. Similar magnitudes are observed in effluents from aerated lagoons in warmer climates.

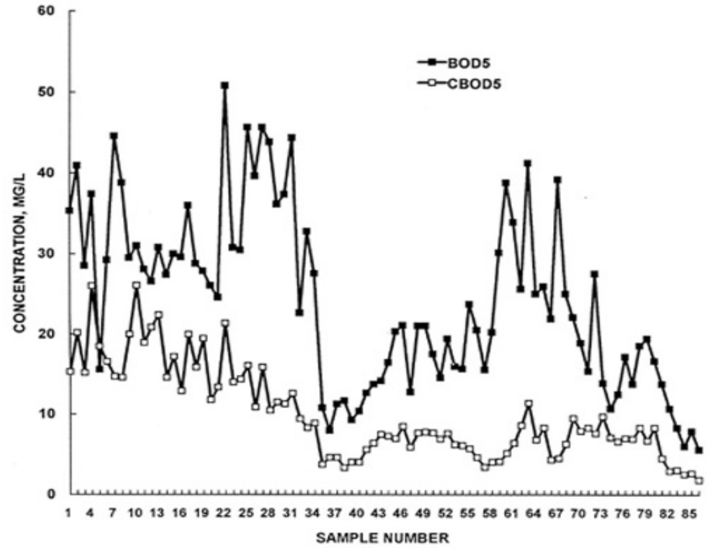

Figure 1. Effluent BOD_5 and $CBOD_5$ data from an aerated lagoon system in Maine that treats a domestic wastewater. (Courtesy of George Bloom, Woodard and Curran, Engrs. Taken from Rich (1999))

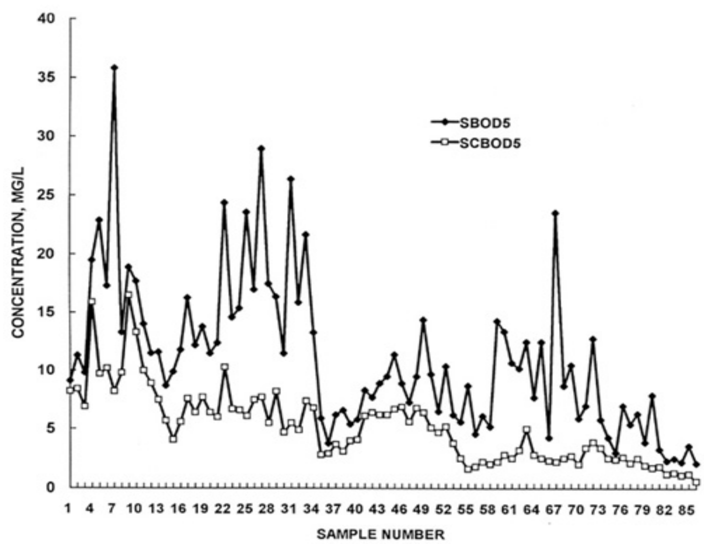

Figure 2. Effluent $SBOD_5$ and $SCBOD_5$ data from an aerated lagoon system in Maine that treats a domestic wastewater. (Courtesy of George Bloom, Woodard and Curran, Engrs. Taken from Rich (1999))

Nitrification in the BOD_5 test has been thoroughly researched and documented (Young 1973; Dague 1981; Barth 1981; Carter 1983; Chapman et al. 1991). Such nitrification can be eliminated by the use of commercially available nitrification inhibitors, a practice recommended by Standard Methods (1995). Chapman et al. (1991) demonstrated that by cleaning the sampler tubing weekly with chlorine bleach, nitrification in the BOD_5 test can be reduced. The U.S. EPA has given their approval to the use of a nitrification inhibitor, provided that the effluent permit states the limit in terms of the $CBOD_5$ instead of the BOD_5. Arguing that secondary BOD_5 limits were initially established on the basis of values flawed by nitrification, the EPA has suggested that the $CBOD_5$ limit for secondary treatment be 25 mg/L rather than the 30 mg/L allowed when the limit is stated in terms of BOD_5 (Hall and Foxen 1983). Considering the fact that the nitrification component of the BOD_5 is generally at least 5 mg/L and frequently as high as 50 mg/L, the 25 mg/L limit appears to impose no handicap.

In summary, BOD_5 is an ambiguous parameter when applied to secondary effluents, especially those of aerated lagoons, and should not be used. Instead, use should be made of the $CBOD_5$ test which specifically measures the concentration of the biodegradable carbonaceous materials.

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Technical Note Number 2

AERATED LAGOON EFFLUENTS

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There are two great myths in aerated lagoon technology. The first myth is that effluent BOD_5 measures the biodegradable carbonaceous material in the effluent. The fallacy of this myth was discussed in Technical Note Number 1. Practically all effluent BOD_5 values are inflated by nitrification that occurs in the 5-day BOD_5 test itself. Such inflation is avoided by using the $CBOD_5$ test in which nitrification is suppressed. The second myth is that the effluent BOD_5, or $CBOD_5$, is the residual of the BOD_5 in the influent to the lagoon. In fact, most of the effluent $CBOD_5$ is the result of algae that grows in the lagoon. By discarding these two myths, one is in a much better position to understand the performance of aerated lagoon.

For domestic wastewaters, the BOD_5, or $CBOD_5$, in the influent to a lagoon system consists of two fractions – a particulate fraction consisting of 70 to 80 percent of the total, and a soluble fraction, making up the remainder. In an aerated lagoon cell in which all settleable solids are maintained in suspension by aeration, the removal of the particulate fraction is very rapid, probably no more than 4 to 5 hours. Removal is the result of the physical capture and adsorption by the suspended floc. The time required for the removal of the soluble fraction is somewhat longer, but still quite rapid. The mechanism involved here, is the assimilation of the organic materials for growth.

Attention is directed to Fig. 1. The data points shown there illustrate the soluble BOD_5 remaining in the effluent of a full-scale aerated lagoon treating a domestic wastewater at different hydraulic retention times for a range of temperatures varying from 16° to 20° C (Fleckseder and Malina 1970). The curve in Fig. 1 is a plot of an equation that predicts the effluent BOD_5 (Rich 1991) using coefficients determined by Jorden et al. (1971) for domestic wastewaters at 20° C. The equation, and, hence the prediction, was developed on the assumption that the BOD_5 values truly represented the carbonaceous demand. As was discussed in Technical Note Number 1, the BOD_5 values in the plot were most likely to have been inflated by nitrification in the 5-day test. Therefore, the curve is seen to provide a conservative estimate of the soluble BOD_5, let alone the soluble $CBOD_5$. From the figure, it is obvious that the residual of the $CBOD_5$ in the influent that is found in the effluent will be quite small.

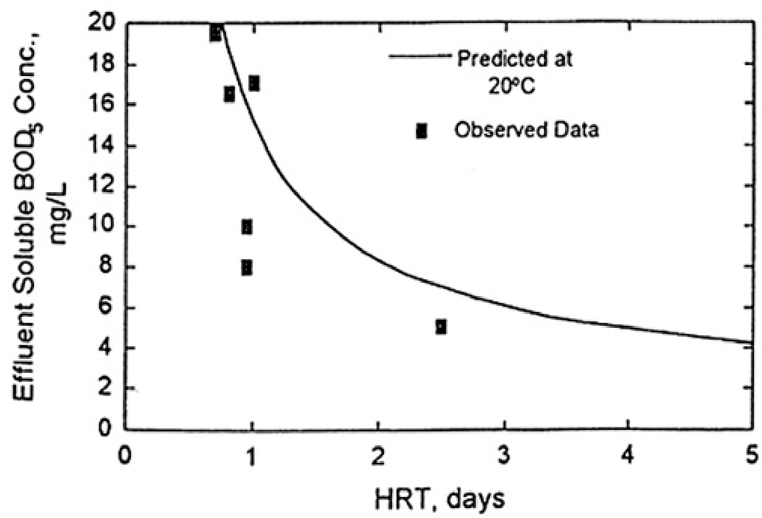

Figure 1. Effluent soluble BOD $_5$ as a function of the hydraulic retention time. (Taken from Rich (1993))

Municipal wastewaters have an abundance of nitrogen and phosphorus, and, thus, when treated in lagoon systems with excessive hydraulic retention times, provide an optimal environment for the growth of algae. The concentration of algae in the effluent is reflected in the magnitude of the total suspended solids (TSS). In the absence of algae, the TSS of the effluent of a lagoon system with a terminal settling cell will normally be less than 10 mg/L. Not only will algae increase effluent TSS, they will also increase the CBOD $_5$. Such increase is the result of the respiration of algae during the 5-day test. On the average (Toms et al. 1975),

$$CBOD_5 = 0.5\ TSS \qquad (1)$$

In summary, most of the TSS and CBOD $_5$ in the effluents of lagoons is caused by algae growing in the lagoon. Very little, if any, TSS and CBOD $_5$ in the effluents are residuals of the TSS and CBOD $_5$ that enter the lagoon. Figure 2 illustrates the impact that nitrification in the BOD $_5$ test and algae have on the effluent BOD $_5$ of an aerated lagoon system located in South Carolina. The values with the legend "ABOD5" were the effluent CBOD $_5$ values whereas those with the legend "NBOD5" were derived from the differences in the BOD $_5$ and the CBOD $_5$ values.

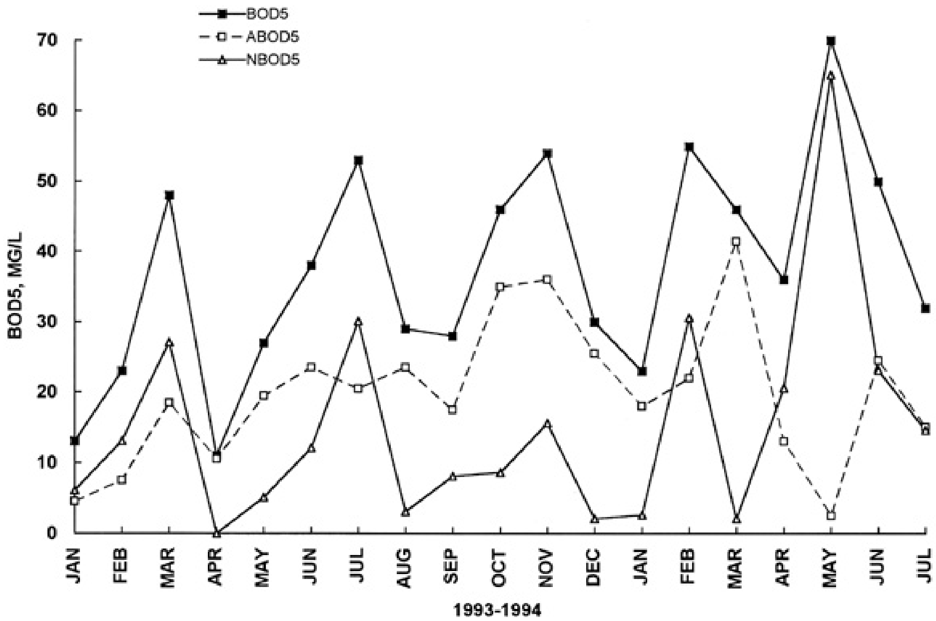

Figure 2. Effluent BOD_5 and its components - BOD_5 caused by algal respiration (ABOD5) and BOD_5 caused by nitrification in the BOD_5 (NBOD5).

Reconstructing Performance Records

As discussed in Technical Note Number 1, effluent BOD_5 is worthless as a performance parameter. Consequently, most historical records in terms of this parameter are of little value in determining performance, especially for aerated lagoons. However, approximate performance records in terms of $CBOD_5$ can be reconstructed using effluent TSS data.

The effluent $CBOD_5$ can be estimated, using

$$CBOD_5 = SCBOD_5 + 0.5\ TSS \qquad (2)$$

where $SCBOD_5$ is the soluble $CBOD_5$. It is seen in Fig. 2 of Technical Note Number 1 that the $SCBOD_5$ component of $CBOD_5$ is generally less than 10 mg/L. Thus, if an aerated lagoon had an effluent TSS of 50 mg/L, the effluent $CBOD_5$ can be estimated roughly as being

$$\begin{aligned} CBOD_5 &= 10 + 0.5(50) \\ &= 35 \text{ mg/L} \end{aligned} \qquad (3)$$

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Technical Note Number 3

CONTROL OF ALGAE

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The problem with algae was discussed in Technical Note 2. Algae growing in an aerated lagoon system will increase both the TSS and the $CBOD_5$ of the effluent. In systems treating municipal wastewaters, the effluent TSS and $CBOD_5$ will often be many times that which would occur if the algae had not been present. Effluent values of these parameters inflated by algae offer no clew as to how well the lagoon is removing the influent TSS and $CBOD_5$. Consequently, engineers mistakenly assume that because the effluent TSS and $CBOD_5$ are approaching, or exceeding, the limit, additional treatment capacity is required when in fact the current capacity may be (and probably is) excessive. Since algae are a distinct liability and play no beneficial role in aerated lagoons, a consideration of ways to prevent, or control, algal growth should be of interest to those responsible for the design and operation of these systems. Such consideration is the focus of this technical note.

Hydraulic Retention Time (HRT)

Retention time is the most influential factor controlling algal growth. In a lagoon basin with a depth of at least 3 m and fitted with mechanical surface aerators that provide a power intensity of about 1.6 W/m^3 (8 hp/10^6 gal of basin volume) or less, algal growth can be expected to occur if the HRT exceeds about 2 d (Fleckseder and Malina 1970; Toms et al. 1975). If, however, the lagoon basin is divided into two or three cells in series by curtain walls, algal growth can be expected to occur only if the total HRT exceeds about 3 d, and 3.6 d. respectively (Rich 1999). Thus, the post fitting of a lagoon basin with curtain walls may reduce effluent algae. At greater aeration power intensities, shading provided by the suspension of settable solids reduce algal growth. At an intensity of 6 W/m^3 (30 hp/10^6 gal), very few algae will grow.

Depth

As photosynthetic organisms, algae require light to grow. Per unit volume of lagoon basin, the quantity of light energy available for such growth is proportional to the surface area. For a basin with vertical sides, an increase in the depth will decrease the surface area proportionally. However, because of the trapezoidal cross section typical of lagoon basins, an increase in depth does not always decrease the surface area. Figure 1 illustrates the relationship between the two variables for a basin with a volume of 2840 m^3 (750,000 gal) and with side slopes of 1 (vertical) 3 (horizontal). For such a basin, an increase in depth will decrease the surface area up to a depth of about 3 or 4m. Beyond which depths the surface area begins to increase.

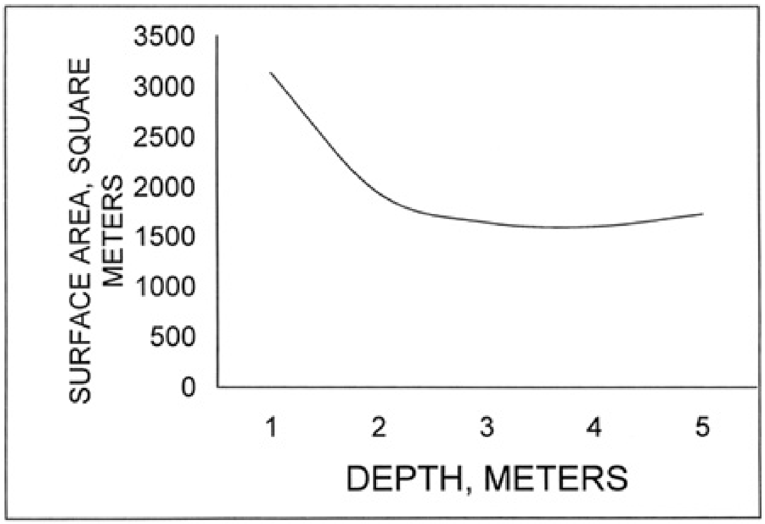

Figure 1. Surface area vs. depth for a lag oon basin with a volume of 2840 m3 and side slopes of 1(vertical):3(horizontal)

Lagoon depths of 3 or 4 m will also create a more favorable geometry for mixing with surface aerators. Reduced surface areas will position the mixing zones in closer proximity.

Mixing

As was discussed above, if a lagoon basin tr eating a domestic wastewater is fitted with mechanical surface aerators that provide a power intensity of at least 6 W/m^3 of basin volume (30 hp/10^6 gal), the turbidity of suspended so lids is sufficient to minimize algal growth. At lower mixing intensities, algae will grow providing the HRT is sufficient. However, all lagoon basins, including those that are used for sedimentation (polishing), should be mixed a level of about 1 W/m^3 of basin volume (5 hp/10^6 gal). Such mixing is beneficial from several points of view. Without mixing thermo stratification will occur, thereby permitting the retention of undisturbed surface layers for relatively long periods of time. Such conditions provide an excellent environment for algae to become established and grow.

Mixing will also exhaust the carbon dioxide from the system. For wastewaters, such as those from domestic origin in which ther e is an excess of ni trogen and phosphorus, carbon dioxide can be growth limiting during a portion of the diurnal cycle. During the night hours when light is not available, carbon dioxide accumulates as the result of respiration of the microorganisms in the lagoon. At dawn, when light does become available, the rate of consumption of carbon dioxide through photosynth esis exceeds that of respiration and, as a result, the store of carbon diox ide is depleted and algal grow th becomes limited. In other words, the carbon dioxide accumulated during the night hours is stored for use in the daytime hours. Carbon dioxide concentrations as high as 25 mg/L have been observed at night in lagoons (Williford and Middlebrooks 1967). Since at sea level the saturation concentration of carbon dioxide is only about 0.42 mg/L at 20° C, mixing by aeration will remove significant quantities of carbon dioxide from the system during the night hours, thus ensuring that carbon dioxide becomes growth limiting earlier in the day. During the day, when carbon dioxide is growth limiting, aeration does not significantly replace carbon dioxide in the system because the concentration gradient is too low. As will be discussed in later notes, aeration in settling basin is a must, not only because of the mixing that is created, but also, for the maintenance of dissolved oxygen in the water column. Such maintenance reduces feed back of CBOD and nitrogen from the benthal deposits.

Cover

Cover of any type, artificial or natural, that will prevent light from entering the water column of a lagoon will prevent the growth of algae. Commercially available floating polyester fabrics have been used to shade aerated lagoons. Such shades should not cover the entire lagoon surface, leaving sufficient room for mechanical surface aerators.

Natural cover can be provided by surface-growing plants such as duckweed. Duckweed, if kept from the effluent by inserting surface baffles in front of the effluent weir, is very effective toward reducing algae in the lagoon. Furthermore, experience in South Carolina has shown that for aerated lagoons, it is not necessary to periodically harvest the duckweed, nor does the duckweed appear to result in significant accumulations in the bottom of the lagoon. Floating grids placed across the lagoon surface have been used to ensure surface coverage. However, several aerated lagoons covered with duckweed have operated successfully without grids. Regardless of the type of cover used, provision must be made for aerating the lagoon. Otherwise, the lagoon will become anaerobic.

Intermittent Discharge

Algae respond to the diurnal variation in light by moving vertically through the water column. King et al. (1970) found that during the afternoon hours, the particulate COD at 8 inches below the surface of a facultative lagoon was about four times that at the same depth during the night hours. Such vertical migration suggests that effluent quality might be improved if the daily flow is released only during the night, or from two different depths over the diurnal cycle.

Chlorination

Several studies have shown that chlorination will kill algae. The focus of most of these studies has been on the impact that algae have on the chlorine demand of plant effluents. In these studies, the chlorine doses used have been large (5-20 mg/L) and the contact periods short (15 min to 2h), conditions under which algae are killed and lyse. At least two authoritative studies, however, have shown that much lower chlorine doses (2-4 mg/L) over much longer contact periods (>10h) will impair algal growth (Echelberger et al. 1971; Kott 1971). This suggests that by continually adding chlorine in a relatively low dose in a aerated lagoon settling basin, effluent algae reduction would occur as a result of a lower growth rate.

Copper Sulfate

Copper sulfate has long been used by waterworks personnel to control algal growth in reservoirs. Some waterworks personnel use a standard dose of 1 mg/L of copper sulfate which is sufficient to kill most types of algae. However, care must be taken to protect fish in the receiving stream. Trout, which appear to be the most sensitive of the fish, do not tolerate copper sulfate in concentrations greater than about 0.14 mg/L (Steel and McGhee 1979). It has been reported that the combination of chlorination and copper sulfate has given excellent results (Courchene et al. 1975).

Water Soluble Dyes

Certain non-toxic, organic water soluble dyes that blocks out the specific light rays utilized in photosynthesis are used for killing algae. Some of these dyes, which leave the water a natural teal blue, have been used to kill algae in sewage lagoons. Such dyes are marketed commercially.

Effluent Treatment

During the late 1960's and early 1970's, much research was conducted on the removal of algae in the effluents of lagoon. At least three review papers describes the scope of such research (Kothandaraman and Evans 1972; Middlebrooks et al. 1974; Parker 1975). A wide range of wastewater treatment processes were investigated in the hope that effluent treatment would be economically feasible. With a single exception, it appears that none of the processes are at the present considered to be feasible, especially for the treatment of aerated lagoon effluents. The exception is intermittent sand filtration, which is used primarily to achieve nitrification, the removal of algae being an added benefit. The performance of intermittent sand filtration in the treatment of aerated lagoon effluents will be discussed in a future technical note. Rapid sand filtration has two disadvantages. The removal of some algal species is marginal, and there is always the problem of what to do with the back-wash water. If the back-wash is simply recycled to the lagoon, algae accumulates in the lagoon, causing more frequent back wash.

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Technical Note Number 4

NITRITES AND THEIR IMPACT ON EFFLUENT CHLORINATION
(Revised February 11, 2002)

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Chlorination of secondary treatment effluents is sometimes impaired by an immediate chlorine demand exerted by nitrites. Such a demand is erratic and can be so large as to prevent maintaining a chlorine residual regardless of the dosage. In most instances, the condition appears to be transitory and soon disappears. However, in some instances, especially in the case of aerated lagoons with long retention times, the condition lasts long enough to result in fecal coliform violations of the effluent discharge permit. Remedial measures to remedy the problem depends primarily on the understanding of the conditions that favor nitrite production.

Cool-Water Accumulation of Nitrites

Nitrification (oxidation of ammonia to nitrate) is a two step process.

$$NH_3 \rightarrow NO_2^- \rightarrow NO_3^- \qquad (1)$$

Under aerobic conditions, with sufficient alkalinity, and a favorable temperature, ammonia (NH3) is oxidized to nitrite (NO2-) which in turn, is oxidized to nitrate (NO3-). At temperatures above approximately 17°C, the first step, the oxidation of ammonia to nitrite, is the slowest step. Consequently, when nitrite is formed, it is rapidly oxidized to nitrate, resulting in a relatively low ambient concentration of nitrite (< 1-2 mg/L) being found in the effluent. At temperatures below 17°C, the rate of nitrite oxidation to nitrate begins to decrease until, at a temperature of from 12°-14°C, the rate of nitrite oxidation to nitrate becomes the rate controlling step in nitrification. Under such temperature conditions, significant nitrite accumulation can occur (>15 mg/L) (Randall and Buth 1984). In addition, there can be specific compounds introduced by industrial discharges that exert a differential toxicity on the organisms responsible for the two steps, thereby resulting in nitrite accumulation.

About the only remedy that can be suggested for cold water nitrite formation is to limit the nitrification process by reducing the level of aeration. Usually, accumulations of nitrite that occurs in the spring are transitory and will disappear with warmer weather. Accumulations that occurs in the fall will dissipate and disappear as nitrification ceases with falling temperature.

Warm water Accumulation of Nitrites

Nitrite accumulation can also occur under conditions in which temperatures are above 17°C. When oxygen is limiting in parts of the aerated lagoon system, any nitrates that have been produced in other parts of the system that have been aerobic will become reduced by denitrification. Denitrification can be simplified as a two-step process.

$$NO_3^- \rightarrow NO_2^- \rightarrow N_2 \qquad (2)$$

However, unlike in nitrification, the second step appears to be the slowest step (Dawson and Murphy 1972), particularly if the carbon is limiting growth (Freedman 1999) or if the carbon source is complex (McCarty et al. 1969). Furthermore, it has been found that the second step, nitrite reduction, is inhibited by the presence of nitrates (Kornaros et al. 1996), and is more sensitive to oxygen (Kornaros and Lyberatos 1998). Since the second step is the slowest step, nitrite can accumulate in significant concentrations.

When the condition of excessive nitrites in the effluent of an aerated lagoon operating at a temperature >17°C persists, the best remedy is to attempt to nitrify the nitrites to nitrates by ensuring a completely aerobic environment (all aerators on all the time) along with at least an effluent alkalinity of 150 mg/L.

Impact of Ammonia Addition on Chlorine Demand

When chlorine is added to effluents with nitrites but with little ammonium nitrogen, the chlorine reacts as free chlorine and is removed quickly by a chemical reaction with the nitrites, thereby increasing significantly the amount of chlorine that must be used to meet the required limit of coliform concentration. Each mg/L of nitrite nitrogen reacts with 5 mg/L of chlorine. Thus, a nitrite concentration of only 10 mg/L will exert a chlorine demand of about 50 mg/L.

It has been shown, however, that when chlorine is added to effluents with nitrites and with a high concentration of ammonium ion (>20 mg/L), the chlorine reacts preferentially with the ammonium ion forming chloramines (Phoenix 1995). This suggests that, if insufficient ammonium is already present in the effluent, the chlorine demand exerted by effluent nitrites can be minimized by adding ammonia along with chlorine in a well-mixed chlorine contact basin. Chloramines, while effective as disinfectants, will act only slowly with nitrites (Chen and Jenson 2001).

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Technical Note Number 5

AERATED LAGOONS FOR SECONDARY TREATMENT

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Historically, effluent data for aerated lagoons treating municipal wastewaters have been expressed in terms of BOD_5 and TSS. This is unfortunate. As was discussed in Technical Note 1, the BOD_5 parameter is faulty because of its inflation by nitrification taking place in the BOD_5 test. Arguing that secondary BOD_5 limits were initially established on the basis of values flawed by nitrification, the U.S. EPA has suggested that the $CBOD_5$ limit for secondary treatment be 25 mg/L rather than the 30 mg/L allowed when the limit is stated in terms of BOD_5. For this reason, secondary treatment will be defined here as treatment that will consistently meet effluent limits of 25 mg/L for $CBOD_5$ and 30 mg/L for TSS.

Typically, the general configuration of aerated lagoon systems used in the past to treat domestic wastewaters takes the form of an aerated basin followed by an unaerated polishing pond. Effluent from this type of system rarely meets, on a consistent basis, the secondary limits stated above. Figure 1 illustrates the effluent performance of such a system in Georgia. As was discussed in Technical Note 2, practically all of the effluent TSS and $CBOD_5$ is caused by algae that grows in the lagoon. This being the case, why not design the lagoon system in such a way that algal growth is minimized?

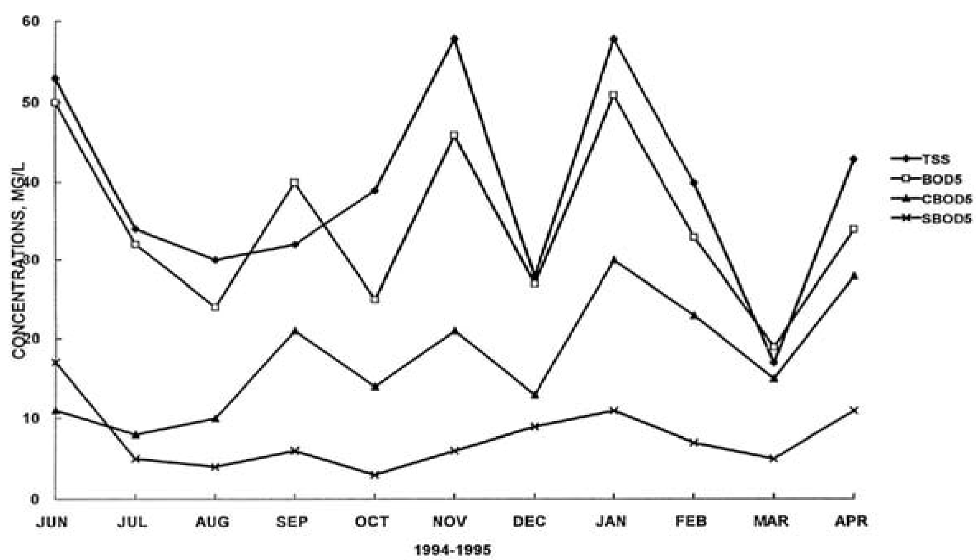

Figure 1. Effluent characteristics of an aerated lagoon system in Georgia treating a domestic wastewater (Courtesy of Bruce Henry)

Figure 2 illustrates in a conceptual way how algal growth can be minimized through control of the hydraulic retention time (HTR). There is a minimum HTR (point a) required to reduce the influent $CBOD_5$ to an acceptable level. There is also an HRT (point b) beyond which algae become established and grow. The key to the design of a system that will produce an effluent with minimal algae is to design a system where the effective HRT falls between points a and b, preferably close to point b considering the sludge storage function of the system. Also, it must be kept in mind that the effective HRT should be based on a consideration of the initial flow rate as well as the design rate. It is just as important for the system to perform well the day that it goes into operation as it is at the end of its design life. One such system is found in Figure 3.

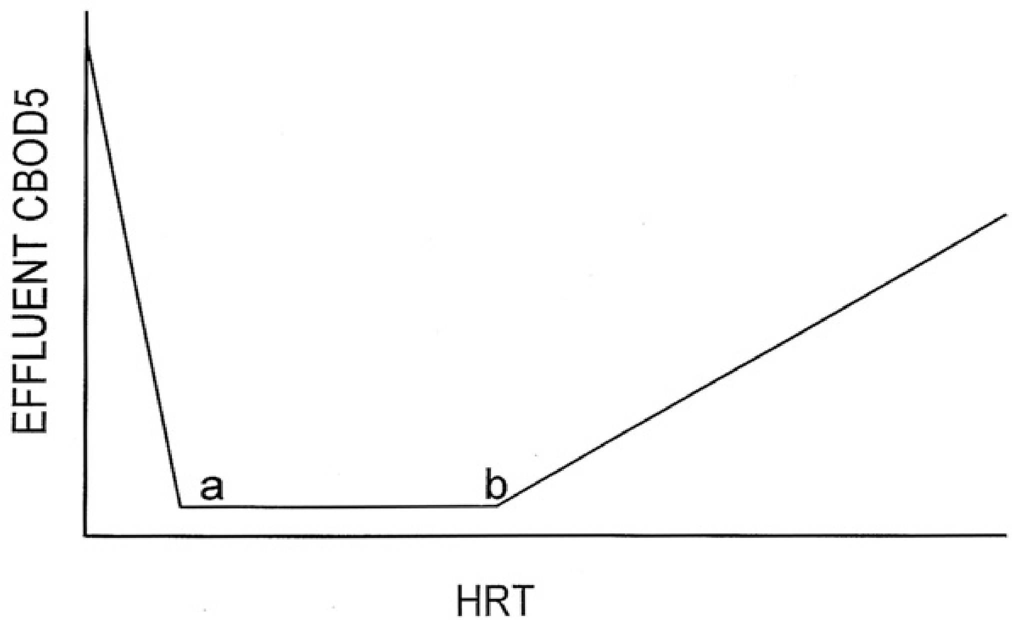

Figure 2. Conceptual sketch of influence of hydraulic retention time (HRT) on lagoon effluent $CBOD_5$

Figure 3. Photograph of a DPMC aerated lagoon system

DPMC AERATED LAGOON SYSTEMS

Figure 3 is a photograph of a dual-power, multicellular (DPMC) aerated lagoon system. The DPMC systems were considered innovative as recent as a decade ago. Now, however, many such systems are operating successfully in the southeastern United States and elsewhere. Design details are found elsewhere (Rich 1999). Essentially, the system consists of four cells in series. For municipal wastewater treatment in the southeastern United States, the system will have, at design flow, a total HRT of 4.5 to 5 days, and a depth of at least 3 m. The first cell (HRT = 1.5 - 2 d) is aerated at 6 W/m^3 of volume (30 hp/mgal), a level that will 1) maintain all solids in suspension, and 2) provide oxygen sufficient for the conversion of the influent CBOD to carbon dioxide and biomass. The following three cells, each with a HRT of approximately 1 d, serve the functions of sedimentation, solids stabilization, and sludge storage. Each cell is aerated at 1 W/m^3 of volume (5 hp/mgal), a level that permits the settleable solids to settle, but, is sufficient to maintain a thin aerobic layer at the top of the solids deposit. The aerobic layer reduces feed-back of nitrogen and CBOD to the water column, and maintains a stable deposit. Aeration also reduces the dead-space volume of the cells.

Since the control of algal growth is crucial in the reduction of effluent suspended solids, careful attention is paid to factors influencing such growth. The turbidity created in the first cell by maintaining all settable solids in suspension reduces light in the water column to the extent that very little algal growth occurs in that cell. The focus of concern, therefore, centers on factors in the remaining three cells. Those factors include HRT, multicellular configuration, surface area, and mixing.

Solids stabilization rates are another consideration in the design of DPMC systems. Benthal stabilization occurs as the result of a combination of aerobic and anaerobic mechanisms. The bottom surface area of the cells must be large enough that the solids loading will not exceed that which will result in all biodegradable solids being stabilized over the annual temperature cycle. The frequency at which the sludge must be removed from the system is also a consideration. One DPMC system treating a domestic wastewater for over twelve years at 40 percent of the design hydraulic load has not required sludge removal.

Control of the HRT in DPMC systems is critical to good performance. Where initial flow rates are significantly lower than design flow rates, three options are available to the designer. One option is to divide the system into parallel trains, additional trains to be brought into operation as the wastewater flows increase. The second is to design the system to operate at multiple depths. Thanks to the shallow slopes of typical lagoon basins, a volume increase of almost 50 percent can be attained by increasing the operating depth from 3 m to 4 m. The third option is to install effluent weirs in all three settling cells, and discharge from the cell giving the best effluent.

PERFORMANCE

Since the DPMC system is in the public domain and no warranties apply to ensure proper design, many engineers inject their own biases and irrational reasoning when they design these systems. One of the biggest mistakes is to add additional HRT as a safety factor. Another is to omit aeration in the last settling cell. Furthermore, even if all settling cells are provided with aeration, operators, to save on power costs, will often operate the aerators intermittently. Of course these mistakes result in more algae, and hence, higher effluent TSS and $CBOD_5$ values. For these reasons, the performance data presented here are for systems in which the author is confident that they have been designed and operated correctly.

Figure 4 illustrates the performance record of a DPMC system located in Berkeley Co., SC. The record was taken from the monthly discharge monitoring reports submitted to the state regulatory agency. Typically, BOD_5 was determined rather than $CBOD_5$. In the 9.5 year record presented in the figure, the TSS and BOD5 values never reached 30 mg/L. Keeping in mind that the $CBOD_5$ in the effluent would be less than the corresponding value of the TSS, one can see that the system can clearly meet the 25/30 limits on a consistent basis. The average of the parameter values, both BOD_5 and TSS, measured during this period of time was about 12 mg/L.

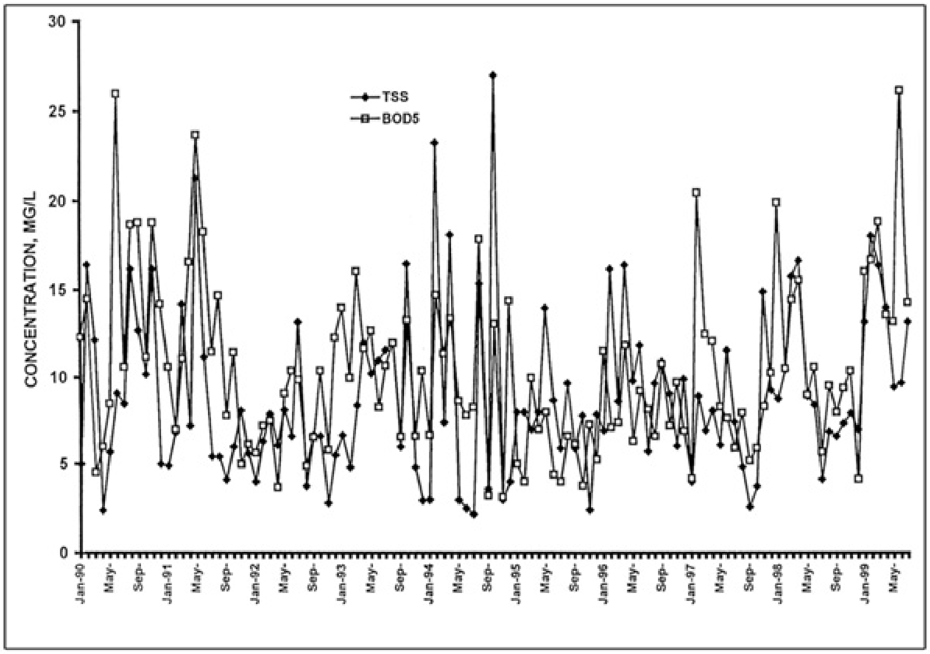

Figure 4. Effluent TSS and BOD5 of a DPMC aerated lagoon system in Berkeley Co., SC

Figure 5 illustrates the performance of a DPMC lagoon (2 mgal/d) located at Hampton, SC. The lagoon is followed by an intermittent sand filter for nitrification. Consequently, effluent data is normally collected for the entire system, rather than just for the lagoon. However, a short study was conducted to evaluate the performance of just the lagoon. The first point to be made about Fig. 5 is that the lagoon effluent TSS values were amazingly low, this in spite of the fact that the system was operating at a flow rate of 25 percent of the design rate. The second point is that the BOD_5 values are all grossly inflated by nitrification occurring in the BOD_5 test. The corresponding $CBOD_5$ values, had they been determined, would have been less than the TSS values.

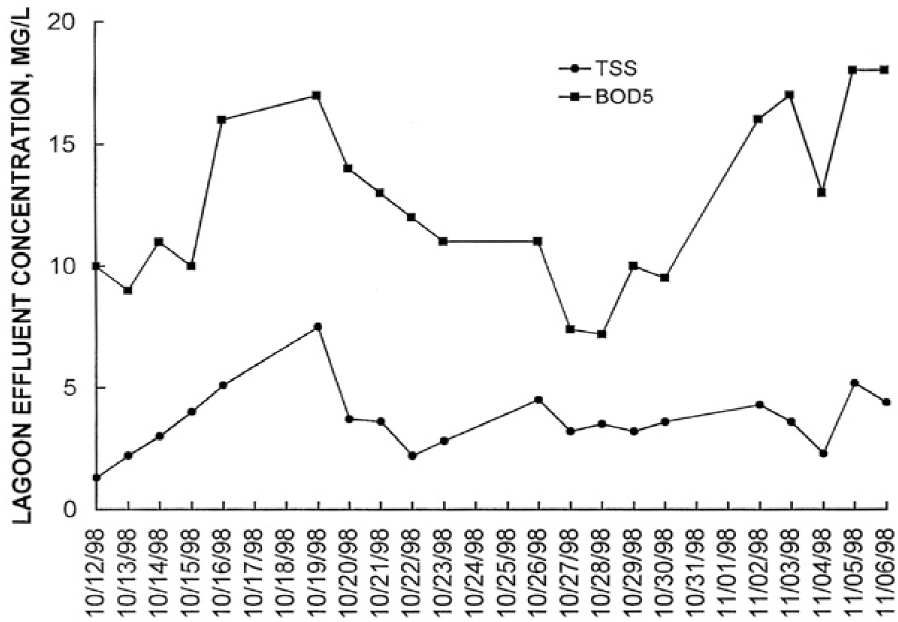

Figure 5. Effluent TSS and BOD5 of a DPMC aerated lagoon system at Hampton, SC

Figure 6 is a sketch of another DPMC lagoon - intermittent sand filter system. This system, with a design capacity of 3.4 mgal/d, is located at North Myrtle Beach, SC, a resort community where the flows during the summer months are about 3 to 4 times the flow in the winter months. For this reason, the system was designed with two DPMC lagoons in parallel, both discharging to one of nine intermittent sand filters. The system has been in operation for about 12 years, and, for this period of time, only the effluent from the entire system has routinely been evaluated. However, in October 1997, the U. S. EPA, Region 4, Enforcement and Investigations Branch conducted an intensive three day, on-site study of the plant followed by a six-month post evaluation to confirm its performance and to study its costs. Table 1 tabulates the results in terms of the means of two, 24 hour composite samples. The sampling points are indicated in Fig. 6. Flow through the plant during the studies was approximately 50 percent of the design flow.

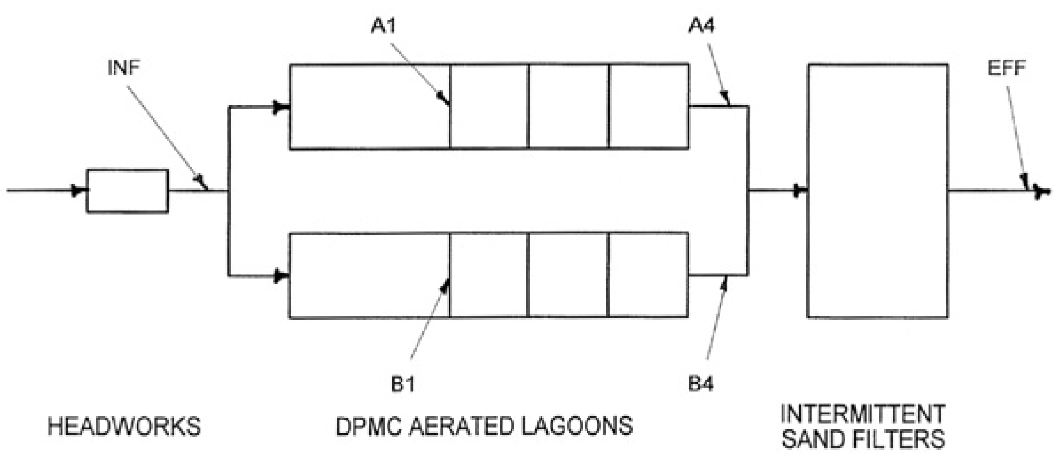

Figure 6. Sketch of a DPMC aerated lagoon -in termittent sand filter system at North Myrtle Beach, SC

Table 1. Effluent characteri stics of the DPMC aerated lagoons at North Myrtle Beach, SC

	INF	A1	B1	A4	B4	EFF
BOD_5[a]	160	21	23	10	12	2
$CBOD_5$[b]	165	16	20	8	6	1
$SCBOD_5$[c]	62	5	5	4	4	1
TSS[d]	185	79	77	8	4	4
ALK.[e]	195	190	190	210	220	17
NH_3-N[f]	25	25	28	31	30	1
NO_3-N[g]	0.07	0.05	0.05	0.09	0.44	32
TKN[h]	37	35	40	34	33	2
TP[i]	5.9	2.8	3.3	0.6	1.2	0.8
CHLOR A[j]	-	-	-	0.056	0.043	-

[a] Standard 5-day, 20°C biochemical oxygen demand
[b] Carbonaceous BOD_5
[c] Soluble carbonaceous BOD5
[d] Total suspended solids
[e] Alkalinity as $CaCO_3$
[f] Ammonia nitrogen
[g] Nitrate + nitrite nitrogen
[h] Total Kjeldahl nitrogen
[i] Total phosphorus
[j] Chlorophyll *a*

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Technical Note Number 6

NITRIFICATION IN AERATED LAGOONS AND WITH INTERMITTENT SAND FILTERS

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Effluent ammonia exerts an oxygen demand on the receiving body of water. Furthermore, ammonia in water exists in two forms – the ammonium ion (NH_4^+) and unionized ammonia (NH_3). At a high pH, most of the ammonia in solution is in the unionized form, whereas at a low pH the ammonia is mostly in the ionic form. Since the unionized form is toxic to the aquatic organisms in the receiving body of water and since both the ionized and unionized forms exert an oxygen demand, effluent limits now often include a maximum limit on the total ammonia. This technical note considers the fundamentals of nitrification and the removal of ammonia in aerated lagoons.

Nitrification is defined as the oxidation of ammonia to nitrate. The oxidation occurs in two steps – the oxidation of ammonia to nitrite by the bacterium Nitrosomonas followed by the oxidation of nitrite to nitrate by the bacterium Nitrobactor. The stoichiometric equations for nitrification are

$$NH_4^+ + 1.5O_2 \rightarrow 2H^+ + H_2O + NO_2^- \tag{1}$$

$$NO_2^- + 0.5O_2 \rightarrow NO_3 \tag{2}$$

Being chemosynthetic autotrophs, nitrifying bacteria derive their energy from ammonia and nitrite and their carbon from carbon dioxide.

Below a pH of 8.5, almost all of the ammonia in solution will exist as the ammonium ion. The conversion of ammonium to nitrite results in the formation of hydrogen ions (Eq. 1). If the pH of the wastewater is less than 8.3, which is typical for domestic wastewaters, the hydrogen ions produced are neutralized by bicarbonate ions in the wastewater.

$$H^+ + HCO_3^- \rightarrow CO_2 + H_2O \tag{3}$$

This reaction results in the decrease in bicarbonate alkalinity as well as an increase in the carbon dioxide concentration, both occurrences of which lowers the pH. If the wastewater has a relatively low alkalinity, the change in pH can be dramatic. In turn, the low pH can significantly reduce the rate of nitrification. Below a pH of 7.2, the rate falls precipitously, approaching zero at a pH of 6. Based on Eqs. 1 and 3, approximately 7.2 mg of bicarbonate alkalinity (as $CaCO_3$) are required to neutralize the hydrogen ions produced by the oxidation of 1 mg of ammonium nitrogen to nitrite. Thus, wastewaters with low alkalinity require the addition of alkalinity to support uninhibited nitrification. In addition to pH, nitrification is very sensitive to temperature, the dissolved oxygen concentration, and toxic materials. The literature on nitrification is extensive. Papers by Sharma and Ahlert (1977) and Barns and Bliss (1983) offer excellent reviews of factors influencing nitrification.

NITRIFICATION IN AERATED LAGOONS

During warm weather months, some nitrification generally occurs in most aerated lagoons treating domestic wastewaters. However, such nitrification is usually unpredictable and cannot be depended upon to meet effluent limits. The reason is that the organisms responsible for nitrification are slow growers and more sensitive to environmental factors than are those that remove BOD_5. Figure 1 illustrates the impact that temperature has on nitrification. The curves, which was prepared by using the kinetics used by Downing et al. (1964), Downing and Knowles (1966), and Parker (1975), predicts the hydraulic retention time (HTR) required in an aerated lagoon to achieve an effluent ammonia nitrogen concentration of 2 mg/L if all the biomass is maintained in suspension, if sufficient alkalinity is present to meet the requirement for nitrification, if the dissolve oxygen is maintained at 2 mg/L, if there are no toxic materials present, and if the influent conditions do not vary. Obviously, these limitations are not met on a consistent basis in the real world; hence even longer retention times would typically be required. Since cold weather temperatures in lagoons in the Southeast may drop to as low as 8 to 10°C, an HRT of at least 6 to 7 days would be required for year-round nitrification. Considering the fact that the completely-suspended biomass conditions require aeration power of about 6 W/m^3 of basin volume (30 $hp/10^6$ gal), such retention times are excessive from the stand point of power usage. Power for solids suspension would be about three times that required to meet the oxygen demand. Therefore, for aerated lagoons to be considered as viable processes for nitrification, the lagoon process must be modified so that the solids age can be uncoupled from the HRT. This can be accomplished either through sedimentation in clarifiers with solids recycle or through the retention of solids by use of sequencing batch reactor (SBR) technology. An example of the latter will be discussed in a later technical note. However, there are available add-on processes that can be used to nitrify effluents of aerated lagoons. These include the intermittent sand filter, a process for which long term performance records are available that demonstrate its success as a nitrifier and polisher with respect to TSS and $CBOD_5$.

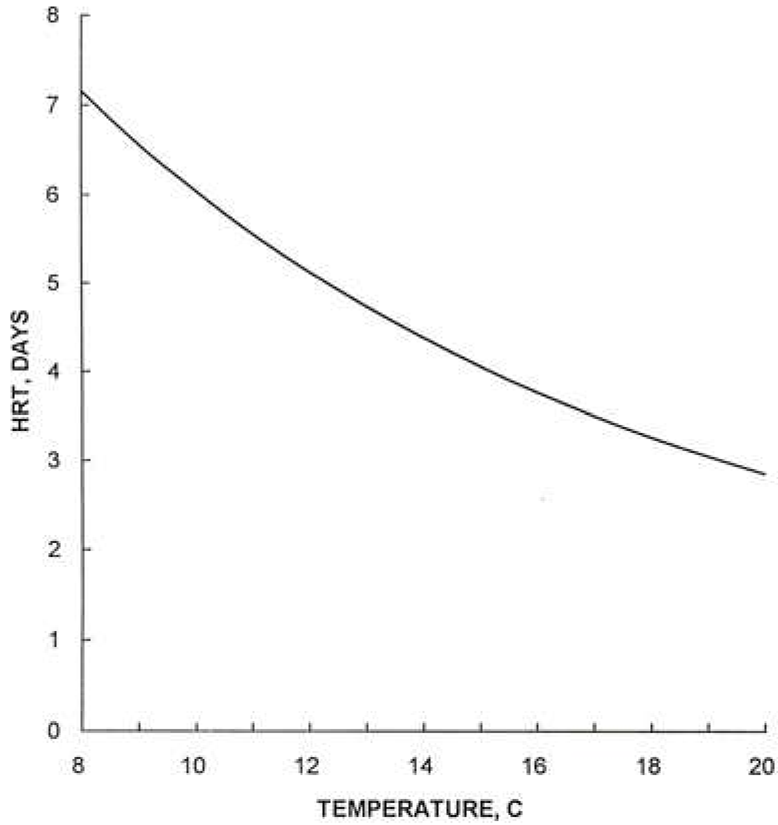

Figure 1. Influence of temperature on hydr aulic retention time required to achieve nitrification in a completely-suspended aerated lagoon under optimum conditons.

NITRIFICATION WITH INTERMITTENT SAND FILTERS

The design of intermittent sand filters can be found be found elsewhere (USEPA 1983, Rich 1999). Intermittent sand f ilters have been used to polish effluents of both facultative and aerated lagoon systems. Their use with the dual-power, multicellular aerated lagoon discussed in Technical Note 5 has been particularly success ful. These systems produce effluents low in TSS, a factor that not only reduces the frequency at which the filters have to be cleaned, but also ma kes possible higher loading rates and smaller filters.

Table 1 lists the effluent characteristics of three DPMC lagoon –intermittent sand filter systems in South Carolina. The Ocean Drive and Crescent Beach plants have been in operation for about for about 13 years; the Loris plant for about 8 years. The table tabulates the mean and 90 percentile values of the TSS, BOD_5, and the ammonia nitrogen in the effluents of the plant. From a regulato ry standpoint, 90 percentile compliance with effluent limitations is adequate (Rott 1996). All three systems re quire alkalinity addition for nitrification. In the operation of the Ocean Drive and Crescent Beach plants, it has been found that if the pH of the lagoon effluent is maintained between 7.5 and 8.0, the ammonia nitrogen of filter effluent will generally be less than 0.5 mg/L. However, since the two plants are meeting their ammonia nitrogen limits of 2 and 4 mg/L, March through October, there is no incentive to achieve the lowest concentrations possible. Figures 1 and 2 illustrate 8.5 years of the monthly reco rds at these two plants. Table 1 in Technical Note 5 shows the effluent ammonia nitrogen found in the intensive three day study of the Ocean Drive plant by EPA described in that technical note.

Table 1. Aerated Lagoon - Intermittent Sand Filter System Performance			
	OCEAN DRIVE	CRESCENT BEACH	LORIS
Size, mgd	3.4	2.1	0.7
Length of record, yr	8.5	8.5	4.6
TSS, mg/L			
50 percentile	4.5	2.8	3.1
90 percentile	7.5	5.4	5.2
BOD_5, mg/L			
50 percentile	2.4	3.0	1.8
90 percentile	3.4	5.3	3.0
NH_3-N, mg/L			
50 percentile	1.1	1.0	1.3
90 percentile	1.8	2.8	3.0

Figures 2 and 3 illustrate the monthly effluent records of the Ocean Drive and Crescent Beach over the 8.5 year period. Such stability combined with the minimal skills and attention required to operate the systems, makes the systems viable alternatives to the activated sludge process, especially in those areas where land costs are not excessive and cheap sand is available. A copy of a paper entitled A Cost Comparison of a Low-Tech Alternative to Activated Sludge by Mike Bowden and Bruce Henry is available upon request at bowden.mike@epamail.epa.gov or henry.bruce@epamail.epa.gov.

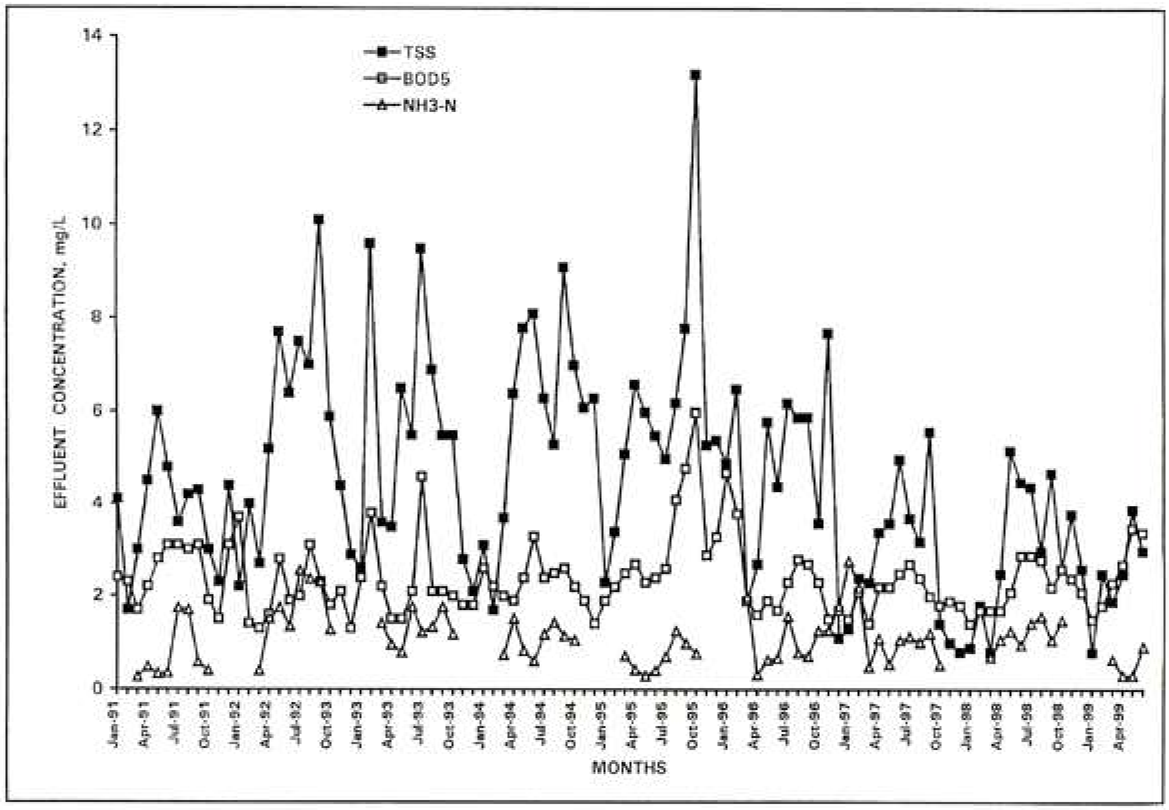

Figure 2. Monthly average effluent TSS, BOD_5, and NH_3-N at Ocean Drive plant, North Myrtle Beach, SC.

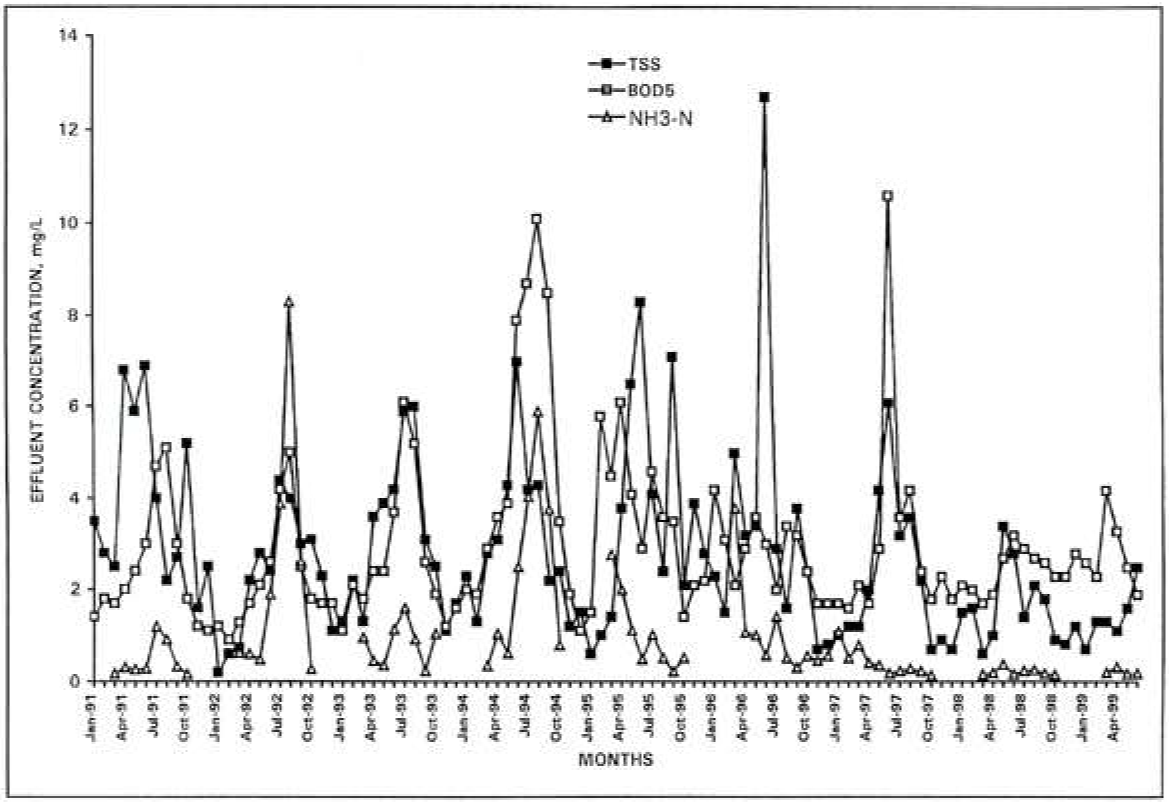

Figure 3. Monthly average effluent TSS, BOD_5, and NH_3-N at Crescent Beach plant, North Myrtle Beach, SC.

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Technical Note Number 7

MIXED-LIQUOR RECYCLE (MLR) LAGOON NITRIFICATION SYSTEM

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As was discussed in Technical Note 6, during warm summer months, some nitrification generally occurs in most aerated lagoons. However, such nitrification is usually unpredictable and cannot be depended upon to meet effluent limits, especially during the winter months. Therefore, for aerated lagoons to be considered as viable processes for nitrification, the lagoon process must be modified so that the solids age is uncoupled from the hydraulic retention time (HTR). This can be accomplished either through sedimentation in clarifiers with solids recycle or through the retention of the solids in the aeration basin by use of sequencing batch reactor (SBR) technology. The latter approach has been used in single basin, continuous-feed, intermittent discharge (CFID) treatment systems for many years both in Australia and the United States. Their performance has been well documented (Goronszy 1979, Arora et al., 1985, Deeney et al. 1991). Although generally successful, some of the CFID systems have had major operational problems as the result of short-circuiting and/or sludge bulking. However, such problems can be minimized, or even eliminated, by modifying the design to deal directly with conditions that promote the problems. The mixed-liquor recycle (MLR) lagoon system incorporates such modifications. The MLR nitrification system, which is not proprietary, is illustrated in Fig. 1. The system consists of two earthen basins in series – a reactor basin for ammonia and $CBOD_5$ removal and a sludge basin for solids stabilization and storage. The overall configuration and size of such a system are discussed in the following paragraphs. Design details are to be found in Rich (1999).

REACTOR BASIN

The reactor basin is divided into two cells by a floating curtain wall or a hard wall. The first cell is aerated continuously, whereas the second cell is aerated intermittently in a controlled cycle that includes sedimentation and supernatant decant. Mixed liquor is recycled from the second cell to a manhole just upstream from the headworks of the plant. There it is mixed with the incoming sewage. The mixture flows through the headworks and into the first cell in a continuous stream. The two cell configuration eliminates short circuiting through the reactor basin as well as modulates the peaks of the diurnal loading pattern. Furthermore, by designing for nitrification to occur in the second cell, the system separates the oxygen demand of nitrifiers from that of the heterotrophs creating more favorable conditions for nitrifiers. The mixed liquor recycle promotes the contact of the biomass with the soluble $CBOD_5$ in the sewage, thus reducing the tendency for filamentous bulking.

The nitrification process is controlled through the continuous wasting of the mixed liquor from the reactor basin to the sludge basin. Such control can be accomplished by diverting a portion of the recycle flow. The flow rate of the diverted mixed liquor will determine the solids retention time at which the process is operated. The rate can be controlled simply by adjusting a hand valve inserted in the piping for the diverted flow.

In addition to aeration equipment and mixers to ensure solids suspension, equipment required includes a programmable logic controller (PLC), liquid level sensors, a low-head recycle pump, and a decant device. The latter can be as sophisticated as a decanter designed for sequencing batch reactor technology, or as simple as pumps or fixed pipes with automatic valves.

The design retention time in the reactor basin should be such that sufficient volume is provided to dilute peak organic and ammonia loads, yet low enough to reduce the power requirements for solids suspension. Therefore, it is recommended that the retention time at design flow rate, HRT_D, be a function of the ratio of the initial flow rate, Q_I, and the design flow rate, Q_D. If $Q_I/Q_D \geq 0.5$, then HRT_D should be 2d. If $Q_I/Q_D < 0.5$, then HRT_D should be 1d.

SLUDGE BASIN

The sludge basin is designed for solids stabilization by benthal processes and multiyear sludge storage. Furthermore, the basin serves as an effluent balance tank to attenuate the intermittent decants from the reactor basin. Two floating curtain walls are used to divide the basin into three cells, each with a hydraulic retention time of about one day at the design flow rate. All cells are aerated, but not at a rate that interferes with sedimentation. Aeration is required in the sludge basin to prevent ammonia feed back from the bottom solids, and to eliminate dead spaces in the water column where algae can become established and grow. If $Q_I/Q_D \geq 0.5$, then the HRT_D should be 3d. If $Q_I/Q_D < 0.5$, then the HRT_D should be 2d.

EXISTING SYSTEMS

Although the MLR nitrification system must still be considered innovative, two such systems are currently in operation and a third is under design. The two systems in operation are located close to Liberty, SC. The Cramer lagoon initially was a dual-power, multicellular aerated lagoon system consisting of a separate reactor basin followed by a sludge basin divided into three cells in series. The upgrade consisted of dividing the reactor basin into two cells in series with a hard wall, fitting both cells with aerators and mixers, providing a decant pump and line to the sludge basin, and a mixed liquor recycle pump and a line to a manhole above the headworks of the plant. A programmable logic controller (PLC) was installed to control aerators, mixers, and pumps. At the present, discharge from the second cell of the reactor is on a six-hour cycle. The Cramer system is permitted at 157,000 gal/d. Cost of the upgrade was $312,000. Photographs of the Cramer lagoon system are shown in Fig. 2.

Initially, the Roper lagoon was a single-basin, facultative aerated lagoon. The upgrade consisted of dividing the existing basin into two cells in series with a floating curtain wall, fitting both cells with aerators and mixers, a decant pump and line to a new sludge basin, and a recycle pump and line to a manhole above an enlarged headworks. The upgrade included also a new three-cell sludge basin, with aerators, and an enlarged chlorination-dechlorination facility. A PLC was installed for control of the aerators, mixers and pumps. The Roper system is permitted at 500,000 gal/d. Cost of the upgrade was $1,100,000.

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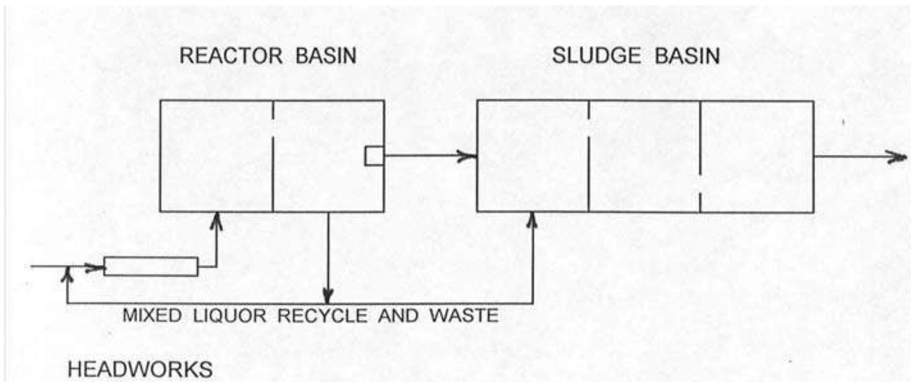
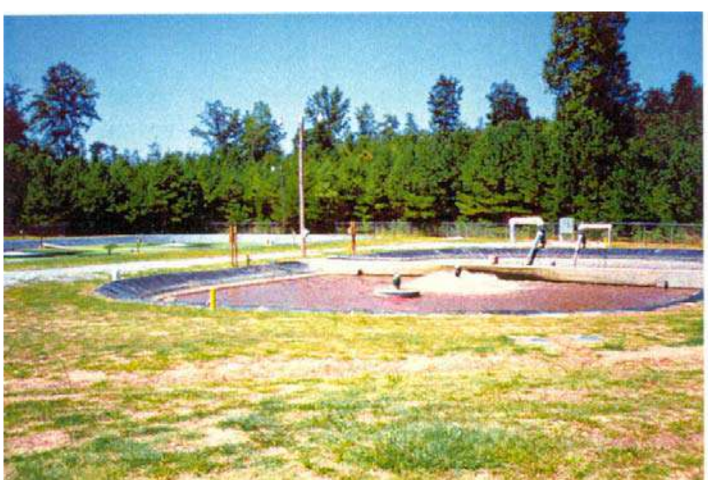


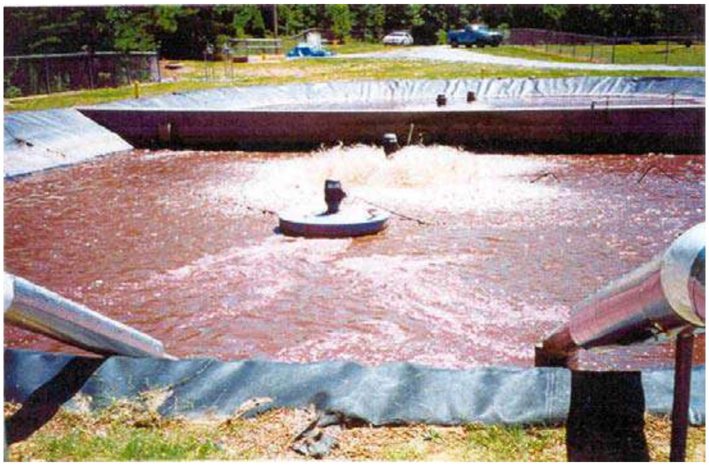
Figure 1. Mixed-liquor recycle (MLR) nitrification system.

MIXED-LIQUOR RECYCLE NITRIFICATION SYSTEM
CRAMER LAGOON, LIBERTY, SC



Reactor basin with first cell in foreground

Closer view with decant and mixed-liquor recycle pumps in left background



Second cell of reactor basin in foreground with headworks in far background

Three-cell sludge basin with chlorination facilities in far background

Figure 2. Photographs of the Cramer mixed-liquor nitrification system at Liberty, SC.

Technical Note Number 8

FACULTATIVE LAGOONS: A DIFFERENT TECHNOLOGY

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It is useful to consider the differences between the technology and performance of facultative lagoons as they have been used to treat domestic wastewaters with those of the high performance aerated lagoons discussed in Technical Note Number 5. First, the oxygen needed for aerobic treatment in facultative lagoons is supplied primarily by algae, the cultivation of which is a major factor in the lagoon design. On the other hand, in high performance aerated lagoons, oxygen is provided by mechanical aeration (usually with mechanical surface aeration), and features to minimize algal growth are incorporated in their design. In fact, the technology of high performance aerated lagoons is more closely related to the activated sludge process than it is to that of facultative lagoons. Second, because algae is a major component of the effluent suspended solids and BOD, facultative lagoons can not, on a consistent basis, meet secondary effluent limits. Third, there is a spectrum of differences in technology and performance existing between the extremes of the facultative lagoon and the high performance aerated lagoon depending upon the extent of aeration and the size and physical configuration of the system. For this reason alone, it is useful to exam in detail the facultative lagoon. The technology and performance of aerated lagoons approach those of the facultative lagoon as the aerated lagoon becomes larger, shallower, and less aerated.

Facultative lagoons used to treat domestic wastewaters provide an example of a highly stressed aquatic ecosystems. These lagoons consist of a shallow basin in which settleable solids introduced by the wastewater settle to the bottom to for a sludge layer that decomposes anaerobically. If oxygen is present in the water column, the biodegradable organic materials that do not settle are degraded aerobically. The term facultative describes the aerobic-anaerobic nature of the lagoon - an anaerobic bottom region covered by an aerobic top layer. The depth of the latter is in a state of constant fluctuation as the result of changing meteorological conditions. The dominant organisms in the system are algae and bacteria which function in a mutually beneficial relationship.

Attention is directed to Fig. 1. Shown there is a diagram of the relationship between algae and bacteria in a facultative lagoon. Biodegradable organic carbon is introduced by the influent wastewater and converted by bacteria to biomass and carbon dioxide. The latter is utilized photosythetically by algae to form algal biomass and oxygen. The oxygen thus produced becomes available to the bacteria for the degradation of more organic carbon. Algal biomass, unlike bacterial biomass, resists gravity sedimentation. As a result, the effluent of a facultative lagoon generally consists of a high concentration of organic carbon in the form of algal biomass. Due to the presence of detergents and urinary products of humans, nitrogen and phosphorus are present in the lagoon in concentrations in excess to the needs of the growing organisms. Consequently, carbon is the element which limits growth in lagoons. Furthermore, the extreme fluctuations in environmental conditions that occur in facultative lagoons tend to minimize the development of higher organisms such as fish.

ROLE OF DIURNAL CYCLE

Because of their shallow depths and large surface areas, meteorological conditions play a major role in determining the characteristics and behavior of facultative lagoons. Such conditions are especially important during the summer in influencing the changes taking place over a twenty-four hour period. A classic study conducted on a facultative lagoon located in the piedmont area of South Carolina revealed the extent to which these changes take place. The lagoon treated domestic wastewater and was loaded at 38 lb BOD/acre-day, a loading typical in the Southeast. The study was unique in that several parameters were monitored at different depths within the lagoon continuously over the 24-hour diurnal cycle (Meenaghan and Alley 1963).

Figure 2 illustrates the type of changes that took place during the summer months. During the daylight period, solar radiation heated the lagoon causing thermal stratification, similar in some respects to the seasonal stratification of lakes. Photosynthesis was very active close to the water surface and, as a result, carbon dioxide was stripped from the top layers and the dissolved oxygen concentration increased to super saturation. The low carbon dioxide concentration in the upper layers of the lagoon was reflected by the abnormally high pH values. It is to be noted, however, that even when super saturation existed at the surface of with respect to dissolved oxygen, the lower portion the water column was devoid of dissolved oxygen. The depth where oxygen depletion occurred is labeled in the figure as the oxypause.

After sundown photosynthesis ceased and cooling of the top layers of the lagoon occurred. Surface cooling imparted a greater density to the surface water causing instability and mixing in the water column. By early hours in the summer, the lagoon was unstratified and no gradient existed with respect to most of the water parameters. At sunup, stratification again occurred and the diurnal cycle repeated itself.

Nuisance conditions in facultative lagoons generally occur during the summer period when high temperatures accelerate the oxygen-uptake activities taking place within the lagoon. For such a period, the information revealed in Fig. 2 leads to the following conclusions:

1. Over a time frame of 24 hr or longer, shallow facultative lagoons can be considered as mixed systems.
2. Facultative lagoons are primarily anaerobic systems with an aerobic top layer that all but disappears for at least a few hours during the night.
3. The fact that dissolved oxygen virtually disappears after photosynthesis ceases indicates that the surface oxygenation rates are low compared with the oxygen demand in the water column.
4. The fact that a high pH is sustained during photosynthesis indicates that the rate of carbon dioxide transport from the atmosphere is low compared to the rate at which algae are utilizing the carbon dioxide.

Distinct seasonal variations occur. During the winter months the diurnal changes in the lagoon were minimal compared to those observed during the summer.

NUISANCE BEHAVIOR

Facultative lagoons are often the source of odors. The odors are primarily caused by two factors - mats of dead algae decomposing at the surface and along the sides of the lagoon, and hydrogen sulfide evolving from the lagoon.

Decaying algal mats give rise to a pig pen odor. Such mats are caused by the periodic occurrence of excessive quantities of filamentous, blue-green algae. This group of algae which flourishes in facultative lagoons during the summer months does not settle like green algae but floats and accumulates at the surface where it decays in the sun, giving off noxious odors. As stressed ecosystems, facultative lagoons are subject to periodic algal blooms.

Hydrogen sulfide is formed from sulfates in wastewaters that become reduced in the anaerobic environment in the lagoon, as well as from the decomposition of proteinaceous materials in the bottom solids. Hydrogen sulfide, which has a rotten egg odor, ionizes in solution to form the equilibrium

H_2S " $H^+ + HS^-$ (1)

Inasmuch as hydrogen ions are one of the ionic species involved in the equilibrium, the particular form in which the sulfide exists is dependent upon the pH of the system. This dependence is illustrated in Fig. 3. The combined form, H_2S, is the only form of the sulfide that is not ionic and can be released from solution as a gas. From the figure it is noted that H_2S exists in significant concentrations only at pH values less than 8. Since the top layer of a facultative lagoon during daylight hours in the summer is greater than 8, the layer functions as a lid to retain the sulfide in solution. Only during the night hours, when the pH of the top layer falls below 8 can hydrogen sulfide escape from the lagoon.

DISCUSSION

Facultative lagoons have three major disadvantages: odors, variable effluent quality, and large land-area requirement. The sources of odors have been discussed above. Variable effluent quality is a characteristic of all facultative lagoons. The removal of BOD_5 will vary from 50 to 95 percent, depending on how much algae is in the lagoon at the time. Only with additional treatment such as intermittent sand filtration can facultative lagoons consistently achieve BOD_5 effluent limits now being imposed. Furthermore significant concentrations of nitrogen and phosphorus are found in the effluents. As for land area requirements, a wastewater flow of 1 mgd will require about 30 acres of lagoon, if the latter is loaded at 50 lb BOD_5/acre-day.

Consideration of the characteristics of facultative lagoons and their performance provide insight to the behavior of aerated lagoons that do not have the aeration, mixing, size, or depth that they should have. Such insight will provide a guide for design modifications that should be made to the aerated lagoon to achieve optimal performance.

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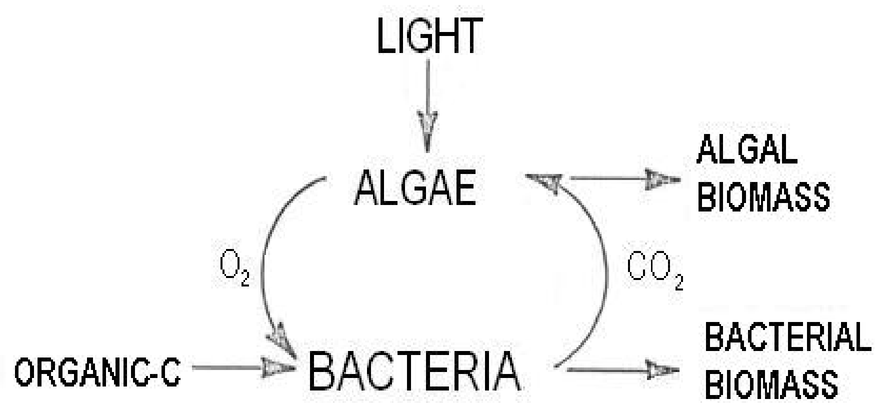

Fig. 1. Conversion of organic-C to biomass in facultative lagoons.

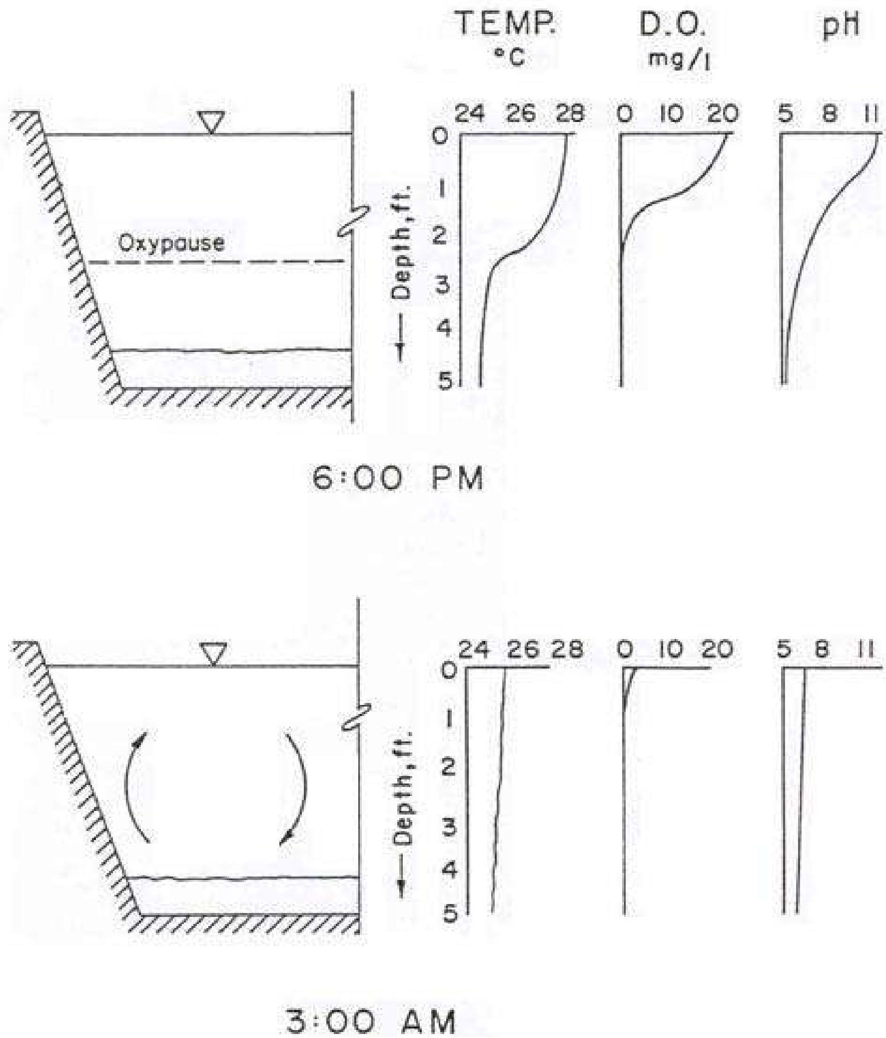

Fig. 2. Profiles typical of summer conditions.

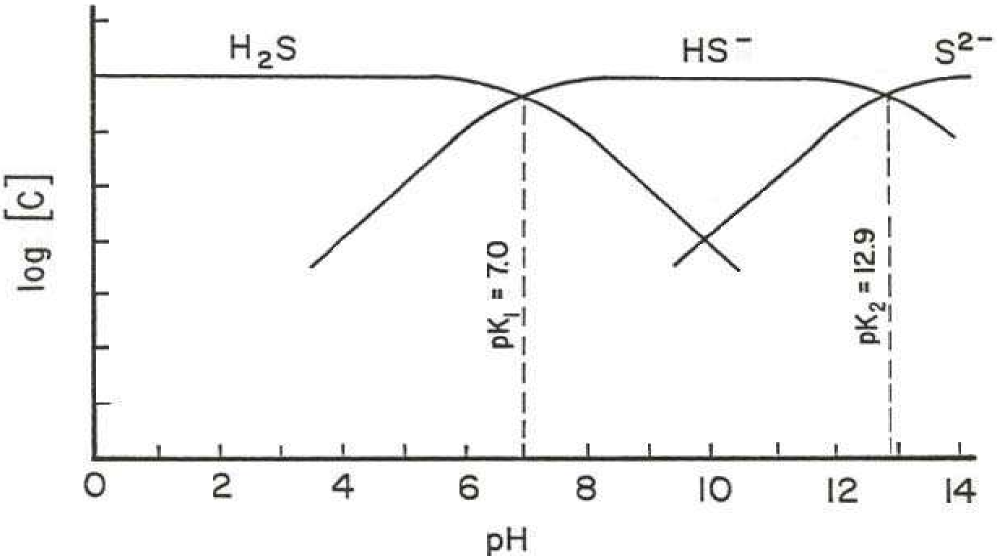

Fig. 3 Hydrogen sulfide system.

Technical Note Number 9

SLUDGE ACCUMULATION IN HIGH-PERFORMANCE AERATED LAGOON SYSTEMS

by
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High-performance aerated lagoon systems are defined here as aerated lagoon systems that can, on a consistent basis, meet both a TSS and a BOD5 effluent limit of 30 mg/L. Since most of the TSS and BOD5 in the effluents of lagoons treating domestic wastewaters are the result of algae growing in the lagoons, the design of the lagoons must include features that minimize such growth. One of the features, a limited hydraulic retention time, conflicts, however, with required sludge storage capacity. As a result, the high-performance lagoon system will have a much smaller foot print and greater sludge depth than do most systems for which sludge accumulation data has been determined. Furthermore, little information is available as to the solids content of these sludges. Both sludge accumulation rates and solids contents are important factors in the rational design of high-performance lagoons.

DPMC LAGOONS

Dual-power, multicellular (DPMC) aerated lagoons can be classified as high-performance systems. The evolution and design of DPMC lagoons are discussed elsewhere (Rich 1982a, 1982b, 1985, 1996, 1999). Basically, the lagoons consist of a reactor cell with a retention time of 1.5 to 2.5 d which is aerated at a level that will maintain most of the solids in suspension, followed by three settling cells in series, each with a retention time of 1 d and aerated at a level that will permit the settleable solids to settle yet maintain dissolved oxygen in the water column. By aerating the first cell at a level that will keep most solids in suspension, sufficient turbidity is maintained in the cell to minimize the growth of algae. Consequently, growth potential for such organisms exists primarily in the settling cells. Many DPMC systems are presently in operation in South Carolina and other regions with similar climates. Although nitrification will occur in most DPMC systems, especially in the warmer months, such nitrification is erratic and should not be depended upon to meet an effluent limit. The lagoons are strictly for carbon removal in temperate climates.

Early in the acceptance of DPMC lagoons as secondary treatment systems, concern was expressed as to the frequency at which sludge would have to be removed from the systems and how would the accumulating sludge with the resulting reduction in retention time impact performance. In absence of long-term operating performance, reliance had to be placed both on an assumed fraction of nonbiodegradable solids in the influent wastewater and an estimate of the sludge solids percent of the accumulated sludge in order to predict accumulation rates (Rich 1985). No attempt was made to predict the effect that reduced retention time would have on performance.

SLUDGE ACCUMULATION

One of the earliest DPMC lagoons to be constructed is located in Berkeley County, South Carolina. As is typical for these lagoons, the lagoon basin is divided into cells by floating curtain walls, and the water depth in the lagoon is 3 m (10 ft). Placed in operation in 1986, the flow into the lagoon has remained at about 40 percent of the design flow of 17.52 L/s (0.400 MGD). Sludge has never been removed from the system. Figure 1 illustrates the sludge accumulation depths in the four cells as of June 2002. Each depth is the average of five in situ measurements at different locations within the cell. The accumulation of sludge found in the first cell can probably be attributed to a combination of factors: regions of lower turbulence levels between the surface aerators, influent sand, and leakage from the second cell under or around the floating curtain wall separating the first and second cells. Distribution in the settling cells is what one would expect, accumulation varying with distance from the first cell. Based on cell size, estimates indicate that sludge now occupies approximately 58 percent of the lagoon volume. The long-term effluent record for the lagoon is shown in Table 1. The effluent statistics are based on a log-normal distribution of data obtained from monthly discharge monitoring reports submitted to the state regulatory agency. The statistics at the top of Table 1 extend from January 1990 to June 2002, a period of 12.5 years. For comparison, averages of the effluent TSS and BOD5 are given below for the recent 2.5 year period extending from January 1999 to June 2002. It appears that sludge accumulation has had little or no effect on performance.

Long-term sludge accumulation rates for two DPMC systems are presented in Table 2. For the Berkeley County lagoon, the rate measured for the 16 year period is less than that for the 7 year period. Such reduction can be explained by the further consolidation of the sludge resulting from additional stabilization and increasing weight of sludge on the bottom solids. For domestic wastewaters, these accumulation rates provide estimates with which the sludge storage requirements can be determined in the design of DPMC lagoons. Alternatively, storage requirements can be determined from estimates of typical sludge solids percentages and estimates of the nonbiodegradable solids in the settling sludge.

SLUDGE SOLIDS PERCENT

Properties of sludges measured at the bottom of four South Carolina DPMC lagoons are presented in Table 3. Values in the table were obtained from samples taken from the bottom of the lagoons with sludge samplers. All four lagoons treat domestic wastewater. The variation observed in sludge solids percentages probably is due, for the most part, to sand that is carried into the lagoons by the sewage. This is reflected in the variation observed in the volatile solids percentages.
Sludge properties will vary from top to bottom of the sludge layer. Bryant (1983) found that the percent solids of a deposit of activated sludge which was added to on a weekly basis increased from 1.2 to 2.4 percent in 15 weeks. Rich and Conner (1982) in another study involving activated sludge solids added intermittently to a deposit found that the percent solids at the bottom of the deposit was 3.25 percent at the end of 42 weeks. Evidently, for such deposits the percent solids varies in a parabolic relationship with time.

DISCUSSION

At the present, insufficient information is available to estimate sludge accumulation rates based on percent sludge solids and percent volatile solids. Needed are these properties at the top of the sludge and at least one intermediate depth. However, sludge accumulation rates based on volume can be estimated using the information given in Table 2. Furthermore, in spite of the relatively short hydraulic retention time provided for in DPMC lagoons, it appears that the performance is unaffected by significant sludge accumulation.

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TITLES OF FIGURES

FIGURE 1. Sludge accumulation in DPMC lagoon.

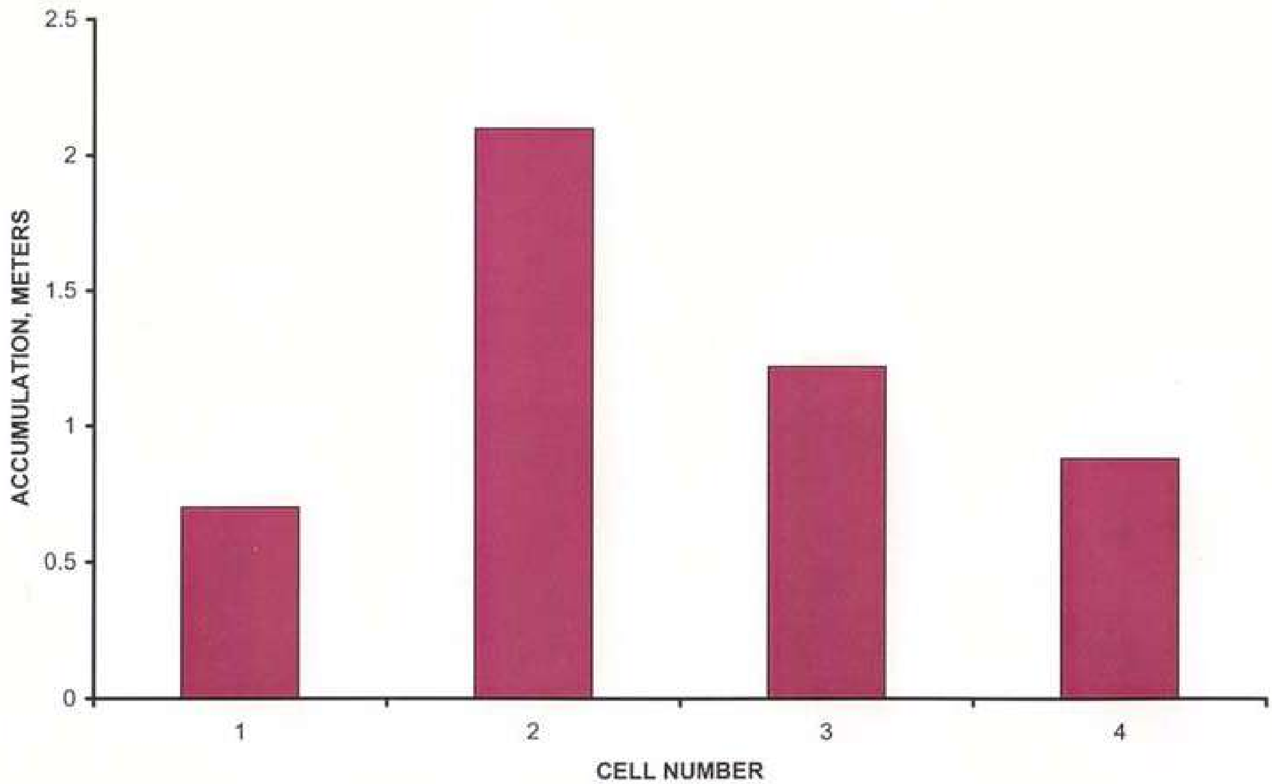

TABLE 1 Long-term performance characteristics of a dual-power, multicellular aerated lagoon located in Berkeley County, South Carolina

	FLOW L/s	FLOW (MGD)	TSS mg/L	BOD_5 mg/L
January 1990-June 2002				
Average	7.23	(0.165)	7.85	10.02
95 percentile	11.57	(0.264)	19.95	22.47
Maximum	14.15	(0.323)	28.68	27.50
January 2000-June 2002				
Average	7.12	(0.163)	8.03	12.93

TABLE 2 Observed sludge accumulation rates in two dual-power, multicellular aerated lagoons located in South Carolina

	Accumulation period Years	Accumulation rate $(m^3 \text{ sludge/y})(L/s)^{-1}$
Berkeley County	7	44
	16	39
Hampton	7	38

TABLE 3 Sludge properties measured at the bottom of four dual-power, multicellular aerated lagoons located in South Carolina

	Accumulation, years	Sludge solids, percent	Volatile solids, percent
Saint Matthews[a]	≈ 15	5.9	50.3
Lugoff[a]	≈ 15	7.4	38.3
Newberry[a]	≈ 10	5.8	50.7
Berkeley County[b]	16	4.8	57.0

[a] Matthews, J. (2002)
[b] Ouzts, C. (2002)

Technical Note Number 10

AMMONIA FEED BACK IN THE SLUDGE BASIN OF A CFID NITRIFICATION SYSTEM

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The discharge of the reactor basin through the sludge basin in a continuous flow, intermittent discharge (CFID) nitrification system is predicated on the assumption that there is no feed back of ammonia from the bottom sludge deposit to the over lying water column (See Technical Note Number 7). Such an assumption is supported by some early research by Mortimer (1971) and Fillos and Molof (1972). They found that as long as the dissolved oxygen in the water column is maintained at 2 to 4 mg/L, very little, if any, ammonia escaped from the deposit. Recent investigations conducted on an aerated lagoon by Jim Matthews of On Line Environmental, Inc. provide additional support to the assumption. The aerated lagoon studied is a 3 mgd dual-power multicellular (DPMC) aerated lagoon system located at Allendale, SC. The system, which is currently operating at about one-third of its capacity, consists of one fully-suspended aerated cell (29 hp/106 gal of volume), followed by two partially-suspended aerated cells (10 hp /106 gal of volume) in series. Although such systems are not recommended for dependable nitrification, some nitrification does occur in such systems, especially during summer months in under-loaded systems such as the one in Allendale.

The monthly average ammonia nitrogen concentration in the cell effluents are illustrated in Figure 1. The data shows that in all months (14 months), the average ammonia concentration in the effluent of the third cell was equal to, or less than, the ammonia found in the effluent of the first cell. The figure does not establish that there was no feed back, but is does indicate that if feed back does occur, nitrification in the two cells equals, or is greater than, the ammonia feed back. As a matter of general interest, Figure 2 illustrates the concurrent total HRT in the partially-suspended cells (cells 2 and 3) along with the final effluent TSS and BOD5.

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Figure 1. ALLENDALE SC AERATED LAGOON SYSTEM

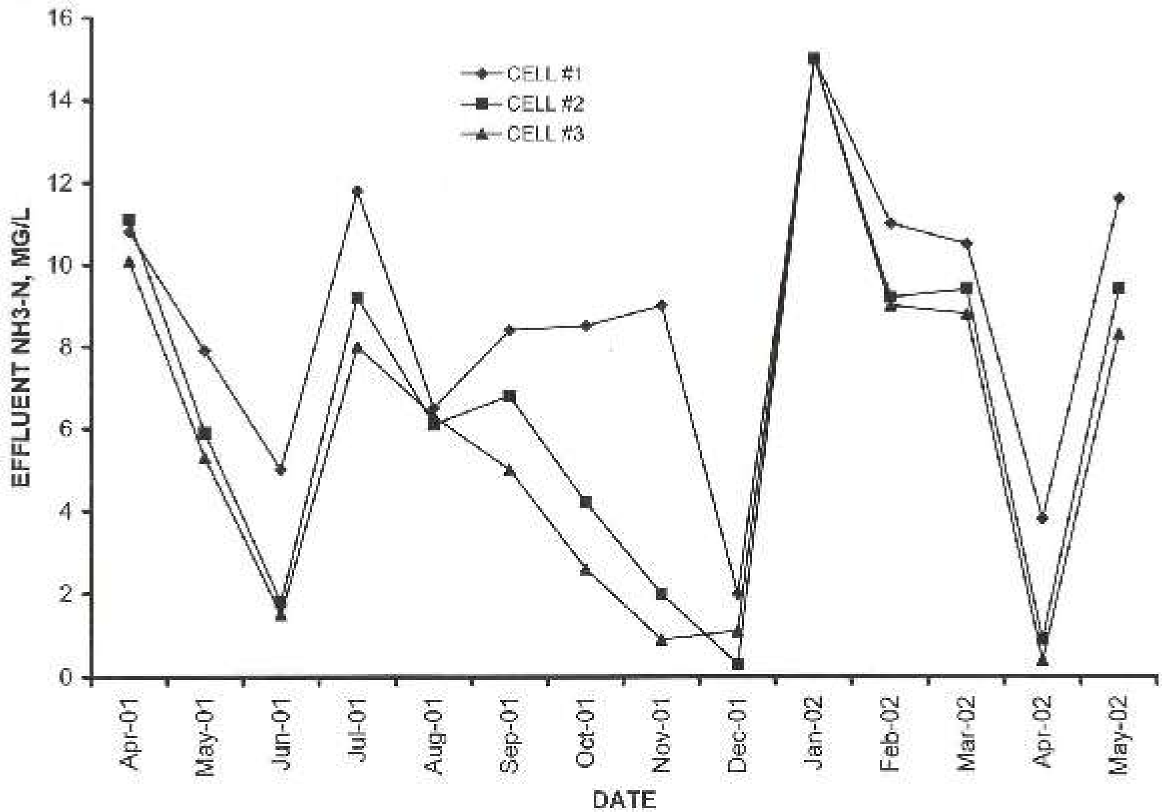

Figure 2. ALLENDALE SC AERATED LAGOON SYSTEM