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Intermittent Sand Filtration to Upgrade Existing Wastewater Treatment Facilities

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INTERMITTENT SAND FILTRATION TO UPGRADE EXISTING WASTEWATER TREATMENT FACILITIES

by

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ABSTRACT

Laboratory and field scale intermittent sand filtration of wastewater lagoon effluents was found to be a promising and economically feasible means of upgrading wastewater effluents to meet more stringent water quality standards of today. While the process was found to be very efficient at oxidizing applied nitrogen compounds, it was also found to be inefficient at removing applied phosphorus compounds. The process was found to be capable of removing appreciable amounts of applied algae, still some algae were found to pass the entire filter depth. When operating under applied BOD concentrations typical in a properly operated secondary biological treatment plant effluent, the process was found to consistently meet present Utah Class "C" water quality standards for BOD ($\leqq$ 5 mg/l). Very high total coliform removal efficiency was exhibited by the process of intermittent sand filtration. The number of consecutive days of operation before cleaning was required was found to be related to the hydraulic loading rate and the filter influent suspended solids concentration. It was estimated that an effluent polishing intermittent sand filter process can be constructed and operated at a cost ranging from \$15 to \$47 per million gallons of filtrate.

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INTRODUCTION

Nature of the Problem

Water, today, has become much more than a basic necessity of life. It has become a tool used in nearly every facet of American society to a point that its availability controls the quality of our lives. Industry, public service, and agriculture are all directly affected by the quantity and quality of the source of water. With such dependence upon one natural resource, it has become apparent that present sources of supply may not be enough to meet the future demands.

Water quality and quantity problems in the State of Utah are similar to those in other parts of the country. In Utah, there are many rural communities that are still fortunate to be surrounded by large areas of open and relatively inexpensive land. It was originally due to this reason that many of these communities adopted waste stabilization lagoons as a means of wastewater treatment. Although this treatment scheme requires large tracts of land, the important consideration was that it gave a satisfactory effluent for minimum cost and maintenance. But now Utah, along with the rest of the nation, recognizes that a better quality effluent is necessary. If small cities and towns are to economically produce a higher quality effluent, some form of treatment must be utilized that will continue to take advantage of the large areas of relatively inexpensive land surrounding these communities. One method of treatment that capitalizes on the availability of large land areas is intermittent sand filtration.

In most areas of the country where intermittent sand filtration has been used, the lack of large inexpensive tracts of land was a major factor contributing to a decline in use. Thus, the relatively inexpensive tracts of rural land available in Utah are a definite asset. Intermittent sand filtration becomes even more economically attractive if filter media are available locally.

Objectives

The objective of this study was to evaluate on a laboratory and pilot field scale the performance of the intermittent sand filter as a polishing process that would upgrade existing wastewater treatment facilities. Particular attention was directed toward ascertaining the effectiveness of the intermittent sand filter as a means of removing the highly variable quantities of algae present in stabilization ponds during the warmer months of the year. These results will be used to develop design criteria for intermittent sand filters that would consistently produce an effluent of a quality that would meet stringent water quality standards.

REVIEW OF LITERATURE

Related Study of Slow Sand Filters

History of slow sand filtration

The basic idea of sand filtration of water is by no means a recent discovery. For nearly a century and a half, man has relied on the fundamental process of allowing water to pass through a bed of sand to improve the quality of his water supply. Historical accounts have recorded the first attempt to define the construction of the first slow sand filter and to label it as such as early as 1828 (12).

Much of its use and further development during the years immediately following its introduction took place in Europe. Slow sand filtration soon was adopted as a necessary water treatment process in countries such as England, France, and Germany. It was used as the water treatment process for small cities in these countries as well as large cities such as Paris.

Through use and experience, changes were developed to meet the varying conditions of each application. But still in every case, the fundamental components were a suitable sand and a means of controlling water to and from the filter. This simplicity of design and operation was one important factor that kept the slow sand filter popular.

At approximately the turn of the twentieth century, American cities began to see a need to improve the quality of many of their culinary water supplies. Thus, the popularity of the slow sand filter spread to the United States. The need for such treatment was in the higher populated areas of the eastern coast. Around 1905, accounts of slow sand filter plants in the U.S. began appearing in various literature sources. An indepth look will now be taken at the characteristics and various operational accounts of slow sand filters.

Design of slow sand filters

Design procedures for slow sand filters appear to have remained unchanged over the years. Differences are normally limited to plant size. Most designs call for adequate underdraining of the bed, usually by a tile-gravel combination, to prevent backup losses in the filter. The filter bottom was usually of a permanent material such as concrete. Upon this, the tile underdrains, which were usually an open-jointed type, were placed and covered with a layer of gravel approximately 18 inches in depth usually applied in three to four coarses. A typical description of the placement of gravel is shown in Table 1.

Table 1. Gravel sizes and thickness of layers normally used in slow sand filter underdrains (23).

Size	Thickness, inch
Coarser than filter sand	3 (7.5 cm)
1/8 to 3/8 inch max. diameter	3 (7.5 cm)
1/2 to 1-1/2 inch max. diameter	4 (8.0 cm)
2 to 3 inch max. diameter	8 (16.0 cm)

Filter sand is placed on the gravel at a depth that varies from 24 inches (0.6 m) to 60 inches (1.52 m). The sand itself is normally a well graded material and must have the proper effective size and uniformity coefficient. The effective size of a sand is defined as the diameter of sieve through which 10 percent of the sample will pass. For slow sand filters, this value is usually greater than .15 mm (.0059 inch) and less than .35 mm (.0137 inch) (7,30,34). The uniformity coefficient is defined as the diameter of sand at 60 percent passing divided by the diameter of sand at 10 percent passing. For most slow sand filters, this value is usually less than 3.0 (14,30,34).

Once the sand is selected, it must be decided as to how the water will be supplied to the filter, collected after filtration, and the number of filter units necessary to handle the volume of water to be filtered. Hydraulic loading rates for slow sand filters are usually dependent upon the effective sand size and the amount of suspended matter and turbidity in the raw water. Loading rates of 1.5 million gallons per day per acre (2297.7 m^3/hectare-day) to approximately 3 mgd per acre (4595.4 m^3/hectare-day) were commonly reported (41,23,29,4). Most of the values reported for any one plant were highly variable because of the length of time necessary to plug a filter (i.e., a run) was highly variable. The variability in the run length was directly related to sand size, raw water turbidity and the hydraulic loading rate (4).

There have been attempts to increase the capacity of slow sand filters and filter plants. In most cases, increased filter capacity was obtained with more frequent and more efficient sand washing made possible by the development of power-driven washing machines (17). It was also then necessary to enlarge and improve the water controlling devices of the plant to afford the higher filter rates (42). The capacity of the McMillian plant in Washington, D.C., was increased by installing power-driven washing mechanisms and increasing the control devices. These changes increased the maximum capacity of the plant to 5.5 mgd per acre (8426.0 m^3/hectare-day), with an average output of 3.98 mgd per acre (6110.0 m^3/hectare-day). Even higher filter rates were obtained in Springfield, Massachusetts, by adding four new filters plus improving cleaning methods and water carriage systems of the plant (30). These improvements, which were added in 1952, gave the plant an operating capacity of 5 mgd per acre (7659.0 m^3/hectare-day) with a peak capacity of 15 mgd per acre (22977.0 m^3/hectare-day).

As noted earlier, the design of a slow sand filter has not changed much in over a century of use. Slow sand filter construction is well defined and limited by sand specifications and water control systems. Plants with the capacity to supply one residence (23) up to plants to supply cities (30,42) show the flexibility of the slow sand filter. Plants producing water for large populations require many acres of land to meet the demand, and in marginal areas where populations may increase rapidly, it may be wise to utilize other methods of treatment in anticipation of the pressures that may force changes shortly after constructing slow sand filters.

Operational information in the literature should prove to be applicable to the operation of sand filters to polish biologically treated wastewaters. One of the most influential factors in the operation of sand filters is the condition of the influent or raw water. The amount of turbidity, algae, and suspended matter in raw water directly influences the length of filter run.

Operation of slow sand filters

Slow sand filters are usually operated by applying raw water continuously until a predetermined headloss, limited by the available freeboard, is reached. When this point is reached, the bed is drained and cleaned. Loading can begin again as soon as the cleaning operation is completed. Once a sand is selected and placed, the rate of production by a slow sand filter is directly related to the concentration of algae and suspended matter in the influent.

Madely (31) noted additonal problems associated with algae in the operation of flow sand filters. When algae grew on the sand surface, they formed large spongy masses, portions of which floated to the surface. These floating masses carried parts of the filtering skin or "schmutzdecke" with them, causing temporary periods of inefficient filtration.

Algae have also been associated with problems of tastes in culinary water. This problem was noted by Story (47) in the operation of the Ludlow filter plant. He noted, too, that taste problems were not prevalent during periods of high concentrations of algae but were noticed after such concentrations diminished. He attributed the tastes to an increase in the amount of decaying organic matter caused by the accumulation of dead algae on the surface of the filter. Mechanical plugging of the filter was also reported as an operational problem. Raw water for the Ludlow plant was obtained from the Ludlow Reservoir which frequently had high concentrations of algae during the warmer months of the year. Mechanical plugging of the filters was a frequent problem and to overcome this difficulty the plant was designed to operate intermittently. However, Story (47) reported that intermittent loading did not always solve the plugging problem. Many attempts were made to develop operating schemes that would eliminate the plugging problem. Most attempts could be grouped as filter cleaning methods.

When the loss of head through the filters became nearly equal to the depth of water on the bed, the bed was removed from service, dried and raked. Raking was usually accomplished by hand, and the greatest improvement was accomplished by raking in two directions. At the Ludlow plant, Story (47) found that raking, followed by a thorough drying period, provided a satisfactory means of restoring the filtering ability of the sand. Smith (42) reported that after raking, the filter was restored to near original filtering characteristics. Motor driven raking machines were produced to speed up the time consuming process that involved many men.

Scraping was the process in which approximately the top 2 inches (5 cm) of sand were removed and washed. Scraping was usually performed in addition to raking after the economic number of rakings were reached (41,42). Sand was not removed every time a bed became plugged, although it could have been done this often. Saville (41) reported that it required five men approximately 2 hours to rake the same bed that it took 11 men approximately 16 hours to scrape and wash. Therefore, it is necessary to optimize the number of rakings and scrapings because it becomes an economic factor.

Two procedures were commonly followed to recondition a filter. One was to scrape the sand, wash it, and return it to the bed immediately maintaining the design depth of sand at all times. Another method was to scrape sand off periodically, wash it, and store it for future use. When the bed depth was reduced to 18 inches (46 cm), the remaining sand was removed, washed and replaced in addition to the sand that had been stored from earlier scrapings (38).

Bacterial removal

The value of the slow sand filter to remove bacteria from raw culinary water has long been recognized. Jordan (29) reported total bacterial removal efficiencies of 98 to

99 percent for 2 consecutive years of operation. Flu (15) reported similar removals by slow sand filters operated in a tropical climate. Average bacterial reductions of 89 to 93 percent were reported for two filters operated over a 2 year period at Camp Perry, Ohio (4).

Madely (31) reported that bacterial removal by slow sand filters was affected significantly by algae growths on the filter surface. As the algae masses broke loose from the filter surface and floated to the surface of the liquid, irregular intervals of deterioration in the filtrate quality was noticed. Madely (31) felt that this deterioration was largely due to the breaking of the filtering skin by the bouyant algae masses.

Powell (38) showed that deeper sand beds afford much greater bacterial removals. Beds of 18 inches (46 cm) and 36 inches (92 cm) in depth were analyzed as to the ability of each to remove bacteria. Although Powell (38) conceded that much of the efficiency of the sand bed is due to the "schmutzdecke" on the surface rather than the sand itself, his work showed clearly how unsatisfactory it would be to have a bed of less than 18 inches (46 cm) in depth. If bacterial counts are known to be low in the raw water, the method of removing sand until a depth of 18 inches is reached may produce a filtrate of satisfactory quality. But should maximum efficiency of bacterial removal be desired, the procedure of periodic raking, scraping and washing, and immediate replacement of sand to restore original bed depth would be more appropriate.

Other granular medias used

There has been some attempts to determine the capabilities of media other than sand to produce effluents of equal quality. As an example, Bailey (6) showed athrafilt to be a media of promising capabilities. In his study, he replaced the top 4 inches (10 cm) of an existing 0.18 mm (.0071 inch) effective size sand with anthrafilt of an effective size of approximately 0.45 mm (.0176 inch). This mixed media filter was then subjected to typical operation, and compared with a filter containing sand with an effective size of 0.18 mm (.0071 inch). In most measures of performance, the filter containing anthrafilt produced the same quality effluents as the sand alone. Bacteria removals greater than 99 percent were obtained with both types of filters.

Also, this study showed that athrafilt resists abrasion associated with cleaning operations. Anthrafilt was found to be chemically stable and contributed no soluble salts to the effluent. In addition, it was concluded that deeper depths of athrafilt could be used in mixed media beds and still obtain the same quality effluents as obtained with the same overall depth of a bed containing only sand.

Economics

Engineering News Record (ENR) Cost Indices were used to update reported costs to 1972 values. Costs reported in the literature are reported and followed by the updated 1972 value in parenthesis.

The report on the intermittent water filters at Ludlow by Story (47) gave an entire construction cost of $50,724 ($1,008,151) for 4 acres of filters. This plant was simply designed with the filter beds constructed upon the existing ground with no special precaution against water loss such as a concrete floor or embankment. The one unusual cost in this case was an aerator that was used prior to application of the raw water to the beds. The sand used in the filters was of a bank run variety with an effective size of 0.30 mm (.0117 inch) and placed to a depth of 5 feet (1.54 m) above the gravel underdrain. From this, a present day cost of approximately $252,000 per acre of sand bed could be foreseen for construction costs only. The above figure does not include land costs and was calculated on the assumption that the aerator costs were not a substantial portion of the overall plant construction costs. Also, this value does not include any depreciation value for any specified length of time.

Operation costs are usually reported as the cost to produce one million gallons (MG) of filtrate. Saville (41) reported an operating cost of $4.23 per million gallons for 1922 ($40.82) and $3.69 per million gallons for 1923 ($29.01) at the slow sand plant in Hartford, Connecticut. Powell (38) reported an operating cost of $2.74 per million gallons for 1909 ($50.74) at the slow sand filter plant in Baltimore County, Maryland. Story (47) reported an average cost per million gallons for 1906, 1907, and 1908 of $5.73 ($99.25) for the intermittent filter plant in Springfield, Massachusetts. Based upon these cost figures, it appears that the operating costs for 1972 for slow sand filters should range from $30 to $100 per million gallons.

From a report on the 10 year (1906-1916) operation costs of the slow sand water plant in Washington, D.C. (5), it was determined that in all years reported, labor costs composed approximately 80 percent of the total filter operation. Also, through a subjective look at the report, it was noted that in nearly every year listed, labor costs for incidentals and repairs, plus filter attendants were approximately equal to the total cost of raking, scraping, sand washing, resanding, and smoothing. Although slow sand filtration is relatively simple in operation when basics such as loading, raking, scraping, etc., are considered, it is essential that a regular system of maintenance and operation be conducted. In this particular case, the cost of operation and maintenance was about two-thirds of the total labor charges. Thus, since inflationary trends today show labor costs rising much more rapidly than material costs, it can be seen that labor could comprise an even greater portion of the operating costs of such a plant today.

Related Study of Intermittent Sand Filters for Sewage

History of intermittent sand filters

Use of intermittent sewage filtration began in this country in the late nineteenth century. The first intermittent sewage filters were put in use in 1889 in Massachusetts. For many years their use was centered in the New England area. By 1945, approximately 450 intermittent filter plants were in operation in this country. But later reports showed a decrease to 398 in use by 1957. It was also noted (2) that 94 percent of those still in use by 1957 were serving communities with populations under 10,000.

The intermittent sewage filter has long been known to have the ability to produce effluents of relatively high quality as did the slow sand filter for culinary waters. The decline of intermittent sewage filters was related to the same factors that caused the decline of slow sand filters—an increase in quantity of water to be filtered due to a growing population, and to the rising costs of land. There are other factors that will be mentioned later that compounded the problems that caused the decline.

Intermittent sand filtration, as noted earlier, began in the New England area of this country. Located in Lawrence, Massachusetts, was the Lawrence Experiment Station at which many of the first studies on intermittent sand filtration were accomplished. This region of the country was ideal for the application of such a process as intermittent sand filtration. Many small rural communities were developing to the point that it was necessary to treat their wastewater at a central plant which was economical for the small town. Land to build the filters upon was readily available at reasonable rates and there was also abundant quantities of well graded bank run sand available. These conditions encouraged efforts in research at the Lawrence Experiment Station to improve the intermittent sand filter. As a result of this experimentation and success, the use of intermittent sand filters increased.

Following World War II, many people found the mild climate and sparsely populated land of Florida an ideal place to live following retirement. Large numbers of small residential centers such as isolated tourist courts, motels, trailer parks, drive-in theaters, consolidated schools, and housing developments began to spring up all over Florida. It was soon realized that an economic method of sewage treatment would be necessary for these small communities. Thus, the study of intermittent sand filtration was undertaken at the University of Florida at Gainesville (10,20,22). Much of the modern day knowledge on intermittent sand filters has come out of the studies carried out at Gainesville.

Design of intermittent sand filters

Design of intermittent sewage filters is similar to that of slow sand filters. Preparation of the land areas on which the filters are to be placed consists of clearing the land, followed by stripping of the top soil. The top soil and any other wasted soil can then be used for embankments around the filters. The embankments of sod are the cheapest form to build, but may be the most costly to maintain. The grass and weeds which grow upon them must be mowed to prevent encroachment upon the filter beds. More expensive means of building and maintaining embankments would be to pour concrete aprons on the embankments or construct vertical concrete side walls. Although these are initially more expensive to construct, they require less maintenance for longer periods of time which could justify their initial cost.

The filter bottom of natural soil is then carefully graded toward the lines of the underdrains. The main underdrain is usually vetrified clay tile of the bell and spigot type usually 6 to 8 inches (15 - 20 cm) in diameter, and is laid with joints cemented together (2). Laterals are usually solid or perforated sections of tile or other piping material (2). If solid sections of tile are used, the joints are left 1/4 to 3/8 (6 - 9 mm) apart and covered with tar paper. If perforated pipe is used sections are butted together (2). The laterals are carefully graded also to drain to the main underdrain which in turn drains to the main drain, ditch, or receiving water. Underdrains must be designed to carry the sewage away at a flow at least equal to the rate of percolation to insure proper aeration of the bed (2).

With the underdrains in place, they are then carefully covered with coarse stone or gravel placed in three layers of varying sizes. Nearest the underdrains, a layer from 3 to 5 inches (7.6 - 12.7 cm) of 1-1/2 to 2 inch (3.8 - 5 cm) aggregate is usually placed. Then a 3 to 5 inch (7.6 - 12.7 cm) layer of 3/4 to 1 inch (1.9 - 2.5 cm) aggregate is placed. This is then topped off by a 3 to 4 inch (7.6 - 10 cm) layer of 1/4 inch (6 mm) aggregate of pea gravel with the maximum depth of underdrain rock or gravel approximately 12 inches (30.5 cm) (2.27). This grading of the filter bottom prevents sand from being washed into the underdrain tile. Thus, except for the usual concrete bottom in the slow sand filter, the underdrain systems of each case are nearly identical.

After the final course of gravel has been placed and leveled, the filter is ready for the placement of the filter sand. The sand should be free from roots and cementing materials, relatively insoluble and practically devoid of clay or loam and have an effective size not less than 0.2 mm (.0078 inch) and not greater than 0.5 mm (.0196 inch) (2,24,25). The uniformity coefficient should generally be less than 5.0 with some preferring coefficients under 3.5. Salvato (40) reported that studies performed by Allen Hazen have shown that for any one given effective size of sand, the hydraulic characteristics for that sand with a uniformity coefficient of 1.0 will have practically the same hydraulic characteristics as that sand with a uniformity coefficient of 5.0. For this reason, it would not seem justified to place a more stringent specification on the uniformity of the sand than that the coefficient be less than 5.0. Sand deposits found in New England usually had uniformity coefficients between 5.0 and 10.0 (27).

One of the first intermittent sewage filters studied in this country had a sand depth of 60 inches (152 cm) (32). This depth corresponds to the maximum design depth recommended for slow sand filters. Being one of the first attempts at intermittent sand filtration, it was probably felt that greater depth afforded better treatment and that the maximum depth used in slow sand filters was necessary for sewage. To some extent this was true, but later designs calling for depths not less than 24 inches (61 cm) and not greater than 36 inches (91 cm) were found to be satisfactory (2,24,12).

Plant size is directly related to the volume of sewage to be treated and this defines the number of filters necessary. For small plants, a minimum of three filters is necessary for smooth day to day operation (24). Large plants will naturally require a greater number of filters, but there is a limit in the number due to more complex dosing apparatus, sand costs, and land availability.

Distribution of sewage upon the beds can be accomplished in many ways. Centrally located manholes surrounded by splash aprons, wooden and concrete overflow troughs, and perforated pipes are some of the common methods used on larger beds. For smaller beds, corner or corner-side apron methods are usually satisfactory if sand scouring is prevented. To help aid in proper distribution, the sand bed surface should be kept as level as possible. Also, as suggested by Hansen (24), a maximum of 2,500 square feet (232.25 sqm) is all that should be served from one point of discharge.

Hydraulic loading rates have been established primarily through experience. The rate of loading is related to the size of sand and the condition or state of the sewage applied. As a rule of thumb, raw sewage was applied at the rate of 20,000 to 75,000 gallons per acre per day (30.64 to 114.88 m^3/hectare-day) (hereafter noted as gpad), settled sewage (septic tanks, Imhoff tanks, and settling basins) was applied at the rate of 40,000 to 150,000 gpad (61.27 to 229.77 m^3/hectare-day), and biologically treated sewage was applied at the rate of 200,000 to 800,000 gpad (306.36 to 1225.44 m^3/hectare-day) (2,27).

In conjunction with the hydraulic loading rate of a filter, the dosing period must also be determined. In each dosage, a sufficient volume of sewage must be applied to cover the bed to a depth of one to four inches (2.5 - 10 cm), and the dosage should be regulated such that the time required to reach the desired depth of flooding is reached in 7 to 20 minutes (24,27). It is assumed that the noted depths of flooding are directed more towards raw and primary settled sewage since the suggested greater hydraulic loads for biologically treated sewage applied in the same amount of time would develop greater flooding depths.

Operation of intermittent sand filters

The operation of intermittent sand filters is quite similar to the operation of slow sand filters except for the main difference of intermittent dosages. Intermittent sewage filters are realtively simple to operate in most cases, but it must be remembered that the water applied to them in the case of raw sewage and settled sewage is in an unstable state. Daniels (12) stated that sand filters are not "fool proof" and unless they are carefully and intelligently operated, they can become the source of great nuisance or even total failure. Daniels (12) also noted that one of the greatest problems in operation arises due to the fact that the term "intermittent" is many times entirely overlooked. Continuous application of sewage for periods exceeding 24 hours will have damaging effects upon the effluent.

Just as it is poor practice to continuously load a sand bed, it is as poor to not apply enough sewage to give the bed an even dosage. Such poor dosages will not provide enough of the nutrients necessary for the organisms occurring in the sewage to develop or thrive. Thus, it is necessary to have an attendant see that proper dosages are applied in short duration and the bed is allowed to rest between applications. Such operation will allow the bed to drain and draw air into the bed which keeps the system aerobic. These practices will insure maximum effectiveness and minimum cleaning. Daniels (12) cited several advantages to using preventative measures rather than resorting to corrective measures.

Even under proper operation, it will become necessary to periodically remove the solids that have been strained out by the upper portion of the bed. The time between cleaning can depend greatly on the proper operation of the filter and the type of waste being applied. Records show that one experimental filter at the Lawrence Experiment Station (32) has operated 23 years without any sand being removed from its surface. It was calculated that this filter of 1/200 of an acre has received a total of 2,395,532 gallons (9067.09 m^3) of sewage which contained a total of about 6,000 pounds (2727.27 kg) of organic matter (32). This record emphasizes the potential of the intermittent sewage filter when properly operated.

In most applications cleaning will be necessary at regular intervals and can be accomplished by allowing the bed to dry and the surface mat of solids to curl and crack.

This mat can then easily be scraped off the sand and wasted. The bed can then be resanded to restore the original depth, or the cleaning process may be repeated until a minimum depth is achieved before resanding. This practice is similar to the methods used in cleaning slow sand filters.

Clean sand should never be applied until the plugged portion of the sand has been removed down to clean sand to prevent stratification. The existing layer of dirty sand would act as the fine particles in sand of poor uniformity, i.e., a coarse layer overlying a fine one, which will cause clogging below the surface. Stratification would also restrict proper aeration of a bed which is vital to proper operation. In all cases, the beds must be dry, especially on the surface, before cleaning operations begin. If the bed is wet, the time and cost of cleaning the sand will be greatly increased (12).

Findings of the University of Florida

The study of intermittent sand filters at the University of Florida was designed to determine permissible loadings and the resulting degrees of treatment afforded using native Florida sands as filter media. The specifications of the sands studied and the depths of bed in each case are noted in Table 2 (20). These sands were dosed with primary effluent at rates from 75,000 to 175,000 gpad (114.88 to 268.06 m^3/hectare-day) for approximately 12 months.

The filter units were 7.4 feet by 7.4 feet (2.25 m by 2.25 m) (1/800 acre) with walls and floor constructed of concrete. The primary effluent was bar screened, settled in either an Imhoff tank or sedimentation tank for approximately 2 hours, pumped to a holding tank, then stirred just prior to dosing.

The study period was subdivided into two phases to determine the effects of single daily dosings when compared with multiple daily dosings. During the first phase of the study, single daily dosings were applied for each hydraulic loading rate studied. During the second phase, hydraulic loads were applied in two equal loadings, one at 9:00 a.m. and the second at 3:00 p.m. (20).

Table 2. Specifications of the intermittent sand filters studied at the University of Florida (20).

Filter Number	Effective Size (mm)	Uniformity Coefficient	Depth (inches)
8	0.46 (.0181 in.)	2.79	18 (45 cm)
9	0.25 (.0097 in.)	2.22	30 (76 cm)
10	0.25 (.0097 in.)	2.24	30 (76 cm)
11	0.44 (.0172 in.)	2.78	18 (45 cm)
12	0.31 (.0122 in.)	3.26	18 (45 cm)
14	1.04 (.0410 in.)	1.70	30 (76 cm)
15	0.29 (.0113 in.)	3.27	30 (76 cm)

Special efforts were made to minimize the errors associated with grab sampling. Continuous samplers were constructed such that a small portion of the flow was delivered directly into a sample bottle (20). Samples of the effluent were taken using the special devices and raw water influent samples were taken from the influent trough during loadings. During the first phase, samples were taken three times weekly. During the second phase, samples were taken approximately every 5 days. There were no reports made on the 1.04 mm (.0410 inch) effective size sand during the first phase, because the entire bed was not flooded at the applied loading rates. Data were available from the second phase because higher loading rates were used.

Suspended solids removal

Suspended solids analyses were made on a weekly basis in both phases. Samples were preserved with chloroform and measured as composite samples for that week. The concentration of suspended solids applied during the study ranged from 90 mg/l to 130 mg/l. Average suspended solids removals ranged from 89 to 96 percent. Suspended solids removal for the 1.04 mm sand during the second phase was only analyzed at loadings of 175,000 gpad and 250,000 gpad (268.06 to 382.95 m^3/hectare-day) although rates as high as 425,000 gpad (689.31 m^3/hectare-day) were applied. Removals were 83 percent and 43 percent, respectively (20).

BOD removal

Grantham (22) showed that the intermittent sand filter is capable of producing an effluent that is well into the nitrogenous BOD stage. Due to this, it was felt that some misleading conclusions could be made as to the progress of the carbonaceous BOD stabilization. However, Grantham (22) showed BOD removals to be related to hydraulic loadings, sand size, and bed depth. Although these trends appeared obvious, at times BOD results were erratic and could be attributed to the measurement of the BOD of a nitrified effluent. Regardless of these difficulties, the results of the study appear reasonable.

It was found that smaller effective sand size and deeper bed depths produced higher BOD removals and removal decreased slightly with increased hydraulic loading rates. The applied influent BOD ranged in value from 116 mg/l to 185 mg/l for the first phase of the study.

Table 3 shows the effect of hydraulic loading on BOD removals (22). There was no significant decrease in the BOD removals as the hydraulic loading rate was increased.

By increasing the bed depth to 30 inches (76 cm), BOD removals for the effective sizes of sand shown in Table 2 were increased approximately 5 percent. It was then concluded, that under the conditions of the study, the relationship which exists between removal efficiency and loading, whether hydraulic or organic, is small. It was felt that part of this lack of relationship was because the depths of 18 and 30 inches (45 - 76 cm) were both capable of removing 80 to 95 percent of the applied BOD even though there was sufficient amounts of organic matter present to clog the surface sand in a short period of time. Thus, it appeared that for sands smaller than .45 mm (.0176 inch) and over 18 inches (45 cm) in depth, the loading rate should depend upon the amount of organic matter present on the bed surface and the rate at which the biota of the bed could oxidize the organic material (20).

To determine the relationship of bed depth to BOD removed for the sands finer than .35 mm (.0137 inch) effective size, a bed was rebuilt with sampling points at 4 (10 cm), 6 (15 cm), 12 (30 cm), 18 (45 cm), 24 (60 cm), and 30 (76 cm) inch depths (20). It was then loaded at the rate of 150,000 gpad (230.0 m^3/hectare-day) with two equal daily doses for a period of 6 weeks. Table 4 summarizes the results.

Apparently the critical depth of a filter is approximately 12 inches (30 cm). From this, it was concluded that under special conditions minimum depths of 18 inches (45 cm) of the fine sand will produce a satisfactory effluent. Care must be taken to constantly maintain this depth at all times. Because this minimum operating depth was determined under closely controlled laboratory conditions, a minimum depth of 24 inches (60 cm) would probably be more feasible under actual field conditions.

Study of the effect of multiple loadings on the percentage of BOD removed was also conducted. Initially, two equal doses were applied to all beds. Then, four doses were applied to each bed with the final period of doses being applied every hour, or 24 per day. From this experiment, it was concluded that increasing the number of doses beyond that of two per day only slightly increased the efficiency of removal in the beds of fine sands. But, in the case of the .45 mm (.0176 inch) and 1.04 mm (.0410 inch) sands, substantial improvements in efficiency of removals could be foreseen by increasing the number of doses beyond two per day (20). In all cases, substantial increases in efficiency were noted by application of two equal daily doses in place of one daily dose.

Table 3. The effect of hydraulic loading rate on BOD removal by an intermittent sand filter containing 18 inches of sand (20).

Hydraulic loading rate	BOD Removal, %		
	Effective Size of Sand, mm		
	.25 (.0097 in.)	.30 (.0117 in.)	.45 (.0176 in.)
75,000 (114.9 m^3/hectare-day)	93.0	90.5	84.0
100,000 (153.9 m^3/hectare-day)	92.8	89.7	83.4
125,000 (191.9 m^3/hectare-day)	92.5	88.9	83.6
150,000 (230.0 m^3/hectare-day)	92.3	88.2	81.8
175,000 (268.2 m^3/hectare-day)	92.2	87.8	81.0

Table 4. The effect of bed depth on BOD removal for a .35 mm effective size sand (20).

Sand depth (inches)	BOD removal percent	Sand depth (inches)	BOD removal percent
4 (10 cm)	72	18 (45 cm)	92
6 (15 cm)	83	24 (60 cm)	95
12 (30 cm)	89	30 (76 cm)	97

Oxidation of nitrogen

Another attempt to measure the degree of stabilization of the effluents of the sand filters was made by measuring the oxidation of nitrogen. From the first phase of study it was concluded that deeper beds and finer sands afford more complete nitrification, with depth appearing to be the predominant factor (22). The finer sands (0.25 mm (.0097 inch) and 0.30 mm (.0117 inch)) produced effluents with 96 to 98 percent of the nitrogen oxidized to nitrate at a loading rate of 75,000 gpad (114.88 m^3/hectare-day). But, as loading rates were increased, oxidation decreased rapidly to the point at which nitrogen oxidation became nearly independent of loading rate at the 175,000 gpad (268.06 m^3/hectare-day) rate (22).

During the second phase, it was concluded that better nitriffication was obtained when two equal daily doses were applied. Also, it was again found that increased loading rates decreased with ability of the finer sands to oxidize nitrogen. But in the case of the 0.45 mm (.0176 inch) sand, hydraulic loading rate did not effect the ability of the sand bed to oxidize nitrogen. For this sand, it was found that oxidation was dependent upon the organic loading with greater oxidation afforded by lower organic loading rates. Also smaller differences in the ability of both the 18 inch (45 cm) and 30 inch (76 cm) beds to oxidize nitrogen were noted when doses were increased from one to two daily.

More recent work done by Pincince and McKee (37) verifies the findings in the area of nitrogen oxidation by the researchers at the University of Florida. Pincince and McKee (37) found that the ability of a sand filter to oxidize nitrogen depended largely on the aerobic condition of the top portion of the bed. It was hypothesized that when infiltration has just completed, the nitrate concentration was essentially constant for the entire depth of the filter. As time passes and oxygen enters the surface of the filter, nitrate concentration increases greatly in the upper zone of the filter. The amount of nitrate formed depended largely on the length of time between doses. Thus, as the next dose was applied, nitrate formed in the upper aerobic zone was forced out of the filter by incoming water. This continues until the water percolates below the surface at which time the process repeats itself.

This hypothesis was then shown to be valid through field and laboratory experiments. Field experiments, in which the filter was a natural sand deposit from which the effluent would directly enter the groundwater table, showed the aerobic zone and corresponding high nitrate concentrations to reach a depth of approximately 4 feet (1.22 m) 14 hours after infiltration stopped. Laboratory filter columns showed the aerobic zone to reach a depth of 4½ feet (1.37 m).

Temperature effects

Air temperature was 60°F (15°C) and 85°F (29.4°C) during two periods when the loading rate was 125,000 gpad (191.48 m^3/hectare-day). This difference was considered adequate to evaluate the general trend of the effects of temperature on filter performance (22).

As the air temperature increased, the effluent BOD decreased. It was also noticed that the effect of temperature was most pronounced in the .45 mm (.0176 inch) sand, more so in the 18 inch (45 cm) bed of that sand. Grantham (22) felt that the pronounced effect observed in that particular bed was due to the fact that air was able to more easily circulate throughout the bed. This allowed the internal bed to more closely follow the variations in air temperature. In general, it was observed that a decrease in air temperature also decreased the amount of oxidized nitrogen in filter effluent.

Biology of intermittent sand filters for primary sewage

The importance of bacteria in stabilization of wastes by intermittent sand filters was reported by the Lawrence Experiment Station as early as 1889. By 1910, a number of papers discussing the functions and importance of bacteria in filters were published by researchers at Lawrence (10,32).

Calaway, Carroll, and Long (10) realized that although a considerable amount of work had been done with bacteria associated with other treatment processes, there was a lack of modern bacterial study on sand filters. Thus, additional work was undertaken at the University of Florida to study the basic biological, physical and chemical interrelationships in intermittent sand filters.

The filter studied was Unit 12 described in Table 1. Samples were taken from the sand surface by first removing the "schmutzdecke" with a flamed spatula and then removing a portion of the exposed sand as the sample. Samples were taken at depths of 6 (15 cm), 12 (30 cm), 18 (45 cm), 24 (60 cm), and 30 (76 cm) inches using a special coring device.

With respect to general heterotrophic bacteria, extreme variability in numbers present caused considerable difficulty throughout the study. Of most importance was the finding that as loading rate was increased, the number of bacteria present in the filter also increased (9,10). At the highest loading rate of 300,000 gpad (459.54 m^3/hectare-day), members of the genus *Flavobacterium* were the most prevalent in the top 18 inches (45 cm) of the filter. However, *Flavobacterium* were absent at the 24 (60 cm) and 30 inch (76 cm) depths. At times the genus *Bacillus* was predominant. At lower loading rates, below 150,000 gpad (229.77 m^3/hectare-day), *Bacillus* usually occurred in greater numbers than *Flavobacterium.* Although in every level of the filter various species were found, in most cases *Flavobacterium aquatile* or *Bacillus cereus* was the predominant species. In addition, lesser numbers of the genus *Alcaligenes, Streptomyces,* and *Nocardia* were also noted.

It was found (9,10) that the zoogleal bacteria were present in the greatest numbers and were not detected below the 12 inch (30 cm) level. An apparent absence of these floc-forming organisms was attributed primarily to the lack of sufficient food supplies at the lower levels. Calaway (9) noted that without the presence of zoogleal organisms, very little purification would take place. This explains why Furman, Calaway, and Grantham (20) found the critical depth required to produce a satisfactory effluent to be 12 inches (30 cm).

In general, coliform removal was found to be continuous throughout the filter (10). The largest numbers of coliforms were found at the surface of the sand bed which was primarily due to the added filtration of the matted layer of organic matter formed on the surface. The number of coliforms found at the surface was significantly greater than the number noted at the 6 inch (15 cm) level of the filter studied.

Calaway, Carroll, and Long (10) attributed mechanical removal by the mat or "schmutzdecke" to be the principle factor in reduction, but noted that the adverse conditions encountered by the organisms could not be overlooked. Approximately 95 percent removal of coliforms was noted by passage of the raw water through the filter (10).

The possible increase of zoogleal masses, humus, cellulose, and similar materials in the interstices of the filter would become a great problem if it were not for the activity of the metazoa. If the slimes and masses of zoogleal organisms were not effectively broken up by the action of protozoa, they could accumulate to the point of clogging the filter (9). A state of quasi-equilibrium had to be established between zoogleal growth near the surface and surface porosity to insure proper operation of the sand filter. The operation of an experimental filter for 23 years at the Lawrence Experiment Station without cleaning the sand shows that this state of equilibrium can be established in a filter (32). Metazoa found in the bed by Calaway (9) included annelid worms, flatworms, nematodes, rotifers, water mites, insects, and insect larve. Annelid worms appeared to be the most predominant species. These members of the bristle worm or oligochaete group have an insatiable appetite for sludges and slimes. Through their feeding, digestion, and utilization of some of the materials in the slimes, the bed is kept open so that aeration and filtrability of the bed is maintained. The excreted materials of this group are also then more easily acted upon by bacteria. Without the presence of the zoogleal masses, little purification would result. Without the presence of the metazoa, plugging would occur readily. Thus, the presence of both maintains their balance and keeps them active.

Past work has shown that intermittent sand filters removes approximately 78 percent of the BOD by a mechanical mechanism (9). It is thus apparent that bacterial assimilation must account for the much higher removals reported earlier. Calaway, Carroll, and Long (10) found that a large variety of organic compounds are utilized as a source of energy by all of the bacteria encountered in the filter. Pincince and McKee (37) found that the presence of bacteria lessened the diffusivity of air due to their ability to increase moisture retention throughout the filter as compared to clean sand. But, this "disadvantage" was overshadowed by the increased oxidation of nitrogen by their presence and their ability to retain moisture within the sand bed.

Mechanical filtration and adsorption are important in the purification process of intermittent sand filters. However, without the assimilation of filtered and absorbed materials by the biological population, the unit would be much less effective.

Economics

The cost of an intermittent sand filter is governed by the same two factors faced by slow sand filters; i.e., the rising costs of necessary amounts of land and the availability of the sand of proper specifications.

Imhoff and Fair (27) reported costs for 1929 of approximately $10,000 ($87,290) per acre when constructed in natural sand deposits, exclusive of initial land purchase expense. In conjunction with this, an estimated $300 per acre ($2,439) could be expected to cover annual maintenance costs.

In Massachusetts (3), 1903 costs of $3,260 per acre ($62,738) were reported for construction. In connection with this, a cost of $7.75 ($139) per million gallons of filtered water was reported.

As noted earlier, problems associated with increased land and sand costs accelerated the decline of use of the intermittent sand filter. Problems developed due to the lack of intelligent, daily operation. The assumption that sand filters were "foolproof" and nearly maintenance free led to unexpected curative costs. Also, poor operation resulted in odor problems, especially when raw sewage was being treated. Once it was apparent that additional maintenance was required, operating costs rose proportionally.

Related Study in Other Areas of Granular Media Filtration

Algal removal

Some work has been done on the removal of algal suspensions by sand filtration; however, little has been done to evaluate an intermittent sand filter as a polishing unit for secondary effluents.

In the past decade, work in the area of algae filtration has been primarily on a theoretical and experimental basis. The procedure has been to first theoretically determine equations that would relate the variables involved in the process of filtration of algal suspensions. Once these equations have been developed, they are tested and refined through controlled experimentation dealing with the actual removal of algae by sand media.

Ives (28) used radioactive algae in filtration experiments to aid in monitoring and he developed equations which related specific deposits (volume of deposit per unit filter volume), the filter coefficient (measure of a given sand to remove suspended matter, cm^{-1}), and sand depth and size. Although this particular experimental filter was designed to operate under pressurized flows, the relationships observed could easily be applied in some degree to the operation of an intermittent filter used for the removal of algae.

Artifically cultured algae, *Chlorella sp.* and *Scenedesmus sp.*, were removed by sands ranging in effective size from .11 mm (.0043 inch) to .14 mm (.0055 inch) at a depth of 24 inches (60 cm). Ives (28) found that at the surface, the specific deposit increased rapidly with time. But, as the depth into the filter increased, the specific deposit was found to rapidly decrease. At a depth of 10 to 12 cm (4 - 5 inches), the specific deposit was found to reach a near constant state with time.

As the specific deposit increased, the filter coefficient increased to a maximum point and then diminished to zero. In other words, during a certain time interval in which algal suspensions are being applied to the filter, the filter coefficient will reach a point of maximum removal and then begin to slowly decrease in removal ability. Ives (28) also concluded that larger sand sizes have lower filter coefficients, or poorer removal efficiencies. If enough pressure could be maintained, the removal efficiency would eventually follow the same plot as the relation between specific deposit and the filter coefficient. Thus, there is the possibility that under the pressures of head associated with even slow sand filters, the removal efficiency of algae could decrease to zero. In a practical sense, though, it is most likely that either the filter will clog before zero removal is experienced or that a small percentage will continue to be removed by the filter in the case of an exceptionally long run.

Borchart (8) produced results similar to those of Ives (28). He used cultures of *Anabena* and *Ankistrodesmus* in addition to *Scenedesmus* for algal suspensions. He also found that as sand size increased, decreases were noted in removal ability. Algae removal decreased to a point of near constant removal with respect to time. This agrees to some extent with the work done by Ives (28) on the relationship between time, filter coefficient, and specific deposit. In all cases, Borchart (8) found that algae were present in filter effluent throughout the study even though in many cases, the presence of algae in filter effluents could only be detected with a microscope.

When the filter was operated at rates normally applied to slow sand filters for a period of 56 hours, the algal removal efficiency decreased. Thus, the theory proposed by Ives (28) would appear to be relevant in general concept.

More recent work on algal removal by sand filtration has reinforced the findings of Ives (28) and Borchart (8). Folkman (16) concurred with the past findings on the relationship between the filter coefficient and specific deposit under continuous operation using a dune sand and *Chlorella sp.* He described filtration as a process of three phases: 1) a ripening where filter efficiency increased; 2) a period of maximum efficiency; and 3) a period of clogging or a deterioration in filter efficiency with the available pressure determining the final phase.

Folkman and Wachs (16) also noted that algae accumulated in the upper 5 cm (2 inch) layer of sand causing the increases in head losses normally encountered during the filtration of algal suspensions. A few of the algae were found to remain in the deeper layers of the filter but these were not enough to effect the hydraulic characteristics of the lower layers of sand. This agrees with Ives (28) findings where the specific deposit became essentially constant for the lower depths of the filter bed. The remainder of the algae pass through the filter and appear in the effluent.

Folkman and Wachs (16) found that *Chlorella sp.* have the ability to divide and multiply in the dark, and the average diameter of a single cell decreases in size plus its size distribution narrows. Since the period of darkness during filtration was of long enough duration to effect the cell size, the efficiency of the filter decreased. This indicates the importance of the characteristics of the algae being filtered.

Another example of the effect of algal characteristics was noted by Borchart (8) where the stringy *Anabena* afforded consistently better removals than that obtained when filtering the smaller and more compact *Ankistrodesmus.*

Secondary treatment effluent polishing

Recently, study has been undertaken in the area of secondary effluent filtration to determine the filterability of secondary effluent and the performance obtained with mixed media high rate filters. Most work has concentrated on the filter media and has neglected the influent suspension (26). If some means were available to characterize the filterability of secondary effluents, granular filters could be better designed and more easily optimized. By optimized, it is meant that filtration would produce a good quality filtrate and not cause high headlosses to develop in the process. It can be seen that this condition is largely dependent upon the nature of the influent particles to be filtered. Thus, if indicators could be developed that would describe the effectiveness of removal of suspended solids and the headloss increases during operation for a given influent, a better matched range of sand characteristics could be studied in order that optimum conditions may be reached.

Filterability index

Some work has been done using the membrane filter to determine the filterability index of different suspensions. But, in most cases, this method did not satisfactorily relate the filterability of the suspension to its filterability through a granular media (26).

Hsiung (26) filtered secondary effluents (trickling filter and activated sludge) with a membrane filter and a specified granular media filter. Relationships between removal efficiency and solids loading for that particular suspension were determined and a dimensionless parameter, E, was obtained that would describe the removal efficiency expected for a given suspension. Removal efficiencies for particles greater than one micron increased with an increase in particle size. Since E varied directly with particle size, it could be expected to be a good estimate of the removal efficiency expected for a given suspension, independent of filter media characteristics.

To relate the effect of the suspension on headloss produced, the parameter, R, expressed in milligrams, was introduced. This parameter was found to be related to the solids loading factor of the suspension which in turn is directly related to the headloss produced (26).

By determining values of E and R for a suspension using membrane filtration, the filterability of a secondary effluent by a granular filter can be better anticipated. It is emphasized that the parameters E and R are independent of the granular media characteristics. Once E and R are determined, the removal efficiency and headloss anticipated for the influent suspension could be used to determine the appropriate specifications of granular media to be employed to filter that particular secondary effluent. This information would be of great aid when attempting to determine the sand characteristics that would produce the desired effluent quality with a reasonable headloss.

High-rate, mixed media filtration

Nebolsine, Harvey, Fan, and the Hydrotechnic Corporation (35) studied the high rate filtration of combined sewer overflows and performed a limited study to determine the effectiveness of high rate filters to remove suspended solids from secondary wastewater effluents. The secondary effluents were, in this aspect of study, filtered through combinations of sand and anthracite. Typical construction of the filters consisted of 36 inches (91 cm) of sand covered by 48 inches (122 cm) of anthracite. The specifications for the anthracite are shown in Table 5. Four different combinations of sand and anthracite were evaluated. The addition of chemical coagulations to aid filtration was also studied. Little was concluded about filtration efficiency of pressurized high rate filters because the influent suspended solids were quite low. Addition of chemicals produced an effluent with a suspended solids concentration consistently below 10 mg/l, but headlosses were increased significantly (26).

Table 5. The specifications of anthracite used in mixed media high rate filters (35).

Effective size mm	Uniformity coefficient
.66 (.0258 inch)	1.62
.98 (.0382 inch)	1.73
1.78 (.0695 inch)	1.63

Summary

A review of the literature describing the history, design, and operation of slow and intermittent sand filters has been presented. Development of sand filtration has taken place over a time period of nearly one and one-half centuries. This development began with slow sand filtration of culinary water, was eventually applied as a sewage treatment process, and is presently being studied as an effluent polishing process.

Only recently has theory entered the area of wastewater granular media filtration. Unfortunately, theoretical development has not progressed to the point that it can be relied on to solve the problems confronting intermittent sand filtration as a means to upgrade existing wastewaters. Therefore, through the use of information gained from the four related areas covered by this review, a trial and error procedure aided by recent theoretical developments will be employed during this study. This approach will maximize efforts to evaluate the time proven process of intermittent sand filtration so that basic criteria can be established for its use as a wastewater effluent polishing process.

METHODS AND PROCEDURES

Experimental Equipment

The intermittent sand filtration study consisted of both laboratory and field scale experiments. The initial plan was to complete the laboratory study (Phase I) by the end of December, 1972. Based on information gained from the laboratory results, the field prototype filters would be put into operation in the spring of 1973 and studied throughout the 1973 season. The field filters were placed in operation for a short time period during 1972 which provided time to eliminate small operating difficulties before beginning the 1973 operation. This report describes both the laboratory study (Phase I) and the operation of the field filters during 1972 and 1973 (Phase II).

Laboratory study

Nine laboratory scale filter columns were erected at the Utah Water Research Laboratory as shown in Figure 1. Each individual filter column was constructed of 6 inch diameter (15 cm) plexiglass cylinders 6 feet (1.85 m) in length. A flanged coupling was provided in the middle of each column to facilitate the filter cleaning operation.

The filter underdrain material for each laboratory filter was placed on a supporting mesh and the size distribution is shown in Table 6. A depth of 28 inches (71 cm) of filter sand was then placed upon the quarter inch diameter rock (6 mm). Effective sizes and uniformity coefficients for each of the filter sands placed in the designated columns are shown in Table 7.

The 0.17 mm (.0067 inch) effective size sand was the basic sand from which the other two sizes were produced. This sand was purchased from the Le Grand Johnson Construction Company, Logan, Utah, and was a washed bank run sand that was primarily used as the fine aggregate in concrete. The 0.35 mm (.0137 inch) sand was produced by sieving the 0.17 mm (.0067 inch) sand through a U.S. series number 50 sieve, the 0.35 mm (.0137 inch) sand being the portion remaining on the sieve. The 0.72 mm (.0283 inch) sand was produced through the use of the U.S. series number 30 sieve.

The sand was placed dry in the filter columns. It was then settled and compacted by repeated rinsing with tap water until the compacted depth was equal to the flanged coupling. The top half of the column was bolted to the bottom half, and then the filter was ready to receive wastewater effluent.

Table 6. Gravel size and depth of layers of underdrain material placed in the laboratory filter columns.

Size	Layer thickness
1/4 in. (6 mm) max. dia. rock	3 in. (7.5 cm)
3/4 in. (19 mm) max. dia. rock	3 in. (7.5 cm)
1-1/2 in. (38 mm) max. dia. rock	3 in. (7.5 cm)

Table 7. Specifications of the sands placed in the laboratory filter columns.

Filter units	Effective size	Uniformity coefficient
No. 31,32,33	.17 mm (.0067 in.)	5.8
No. 21,22,23	.35 mm (.0137 in.)	3.8
No. 11,12,13	.72 mm (.0283 in.)	2.6

Logan City wastewater stabilization pond effluent was applied once daily to each of the laboratory filters. A bucket with a sprinkler-type nozzle was used to obtain complete distribution of the effluent on the filter.

In order to control the suspended solids concentration in the lagoon effluent applied to the filters, the wastewater effluent was diluted, if necessary, with aerated tap water. Dilution factors were determined on a day to day basis by carrying out a daily suspended solids analysis on the filter influent. Also, prior to dosing, the water temperature was recorded in addition to any other observations noted that day with respect to general filter operation or lagoon performance.

Hydraulic loading rates of 100,000 gpad (153.18 m^3/hectare-day), 200,000 gpad (306.36 m^3/hectare-day), and 300,000 gpad (459.54 m^3/hectare-day) were applied throughout the experiment. Three loading periods of approximately 6 weeks in duration were employed. A loading period constituted a period of operation during which the applied algae concentration was held constant. Plugging is defined as the point in time when all of the

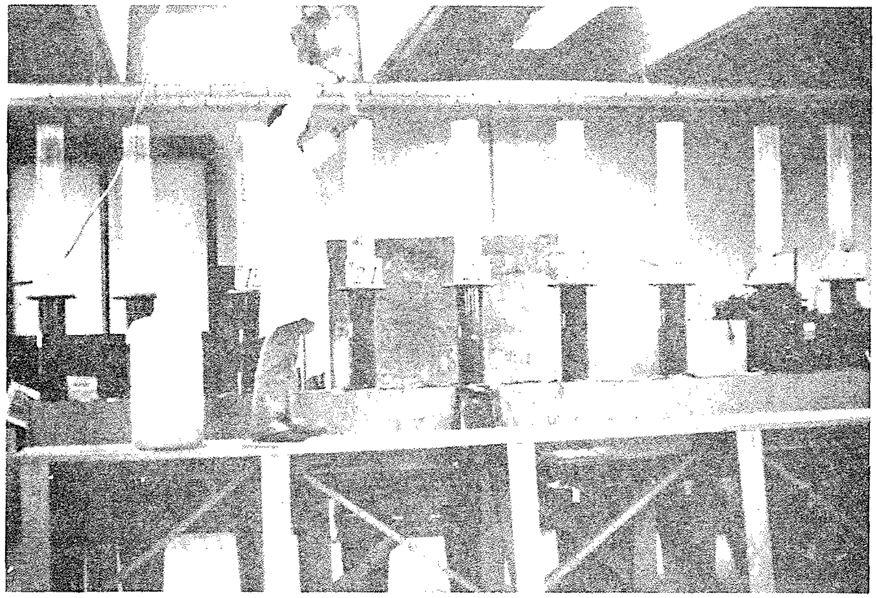

Figure 1. Nine laboratory scale intermittent sand filters shown during daily loading under laboratory conditions.

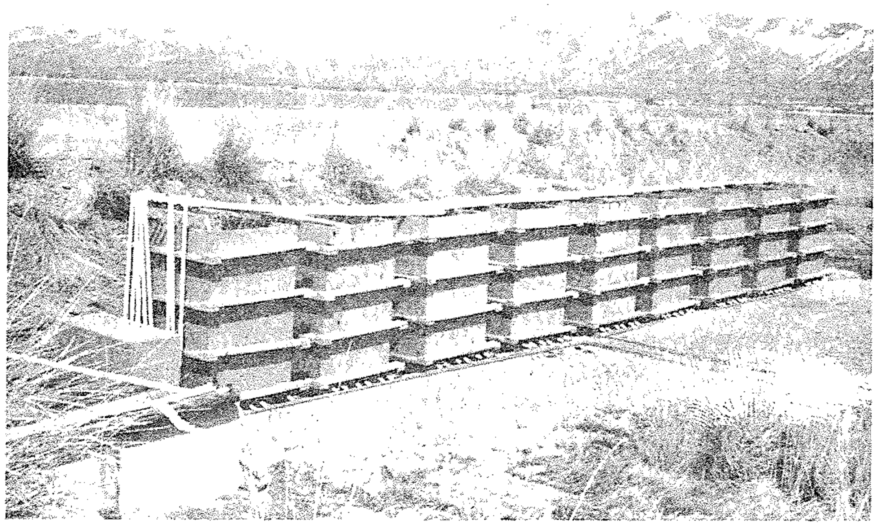

Figure 2. Nine prototype intermittent sand filters located at the point of discharge for the Logan City Wastewater Stabilization Ponds which were used for study under actual field conditions.

specified quantity of wastewater placed on a filter does not pass through the filter in a 24-hour period. Plugging did not occur during any of the three loading periods in the laboratory. At the end of the Loading Periods I and II, the filters were dismantled, the top 10 cm (4 inches) of sand removed from each and replaced with new sand of the same specifications, and the filters were returned to service the same day. At the end of Loading Period III, the top of the sand bed was not removed and daily operation was continued to determine an estimate of the time of operation possible before plugging occurs.

Suspended solids concentrations of 15 mg/l (Loading Period II), 30 mg/l (Loading Period I) 45 mg/l (Loading Period III) were maintained through each of the loading periods. During the first two loading periods, the wastewater used for filter loading was obtained directly from the Logan City Wastewater Stabilization Ponds. This water was obtained once weekly and stored under refrigeration for use throughout the remainder of the week. During the final loading period, the influent to the filters was obtained from model stabilization ponds operated in the laboratory. These ponds were enriched with inorganic nutrients and were illuminated on a fixed cycle of 16 hours of light and 8 hours of darkness. In addition, when water was removed each day for filter loading, the sample was replaced with tap water and once weekly the sample was replaced with water obtained from the Logan City wastewater stabilization ponds. This was done in an attempt to maintain an influent similar to secondary pond effluent. It was not necessary to dilute the effluent from the laboratory lagoons to obtain the desired suspended solids concentration.

Field study

Nine prototype field filters were erected at the discharge point of the Logan City Wastewater Stabilization Ponds and are shown in Figure 2. These units were 4 feet square (1.2 m x 1.2 m) and 6 feet (1.8 m) in height and were constructed of exterior plywood lined with fiberglass and resin. Underdrain construction was the same as the laboratory filters with the exception being that each of the three layers of gravel were 4 inches (10 cm) in depth.

Six filters each were filled with sands of effective sizes of 0.17 and 0.74 mm (.0067 and .0283 inch) to depths of 30 inches (76 cm). The remaining three units were initially filled with 1/4 inch (6 mm) maximum diameter rock to a depth of 60 inches (152 cm). Later in the steady and 1/4 inch rock was replaced with sand of 0.17 mm effective size providing six filters with the basic sand.

Lagoon effluent was applied to the filters with three calibrated pumps operated for a specified period of time. During the fourth week of operation, spreading units were installed to assure better distribution of the raw water on the filter bed. A typical spreading unit is shown in Figure 3.

During the 1972 portion of the study the field filters were also loaded once daily at rates of 100,000 gpad (153.18 m^3/hectare-day), 200,000 gpad (306.36 m^3/hectare-day), and 300,000 gpad (459.54 m^3/hectare-day). The hydraulic loading rates applied in 1973 are summarized in Table 8. The filter containing 0.17 mm effective size sand loaded at 900,000 gpad (1,378.62 m^3/hectare-day) was operated at this rate for only 28 days because of the lack of adequate freeboard to compensate for changes in percolation rate due to increased head loss. When the freeboard was exceeded, the unit (A9, Table 8) was taken out of service. A daily sample of filter influent was taken for suspended solids and pH analysis. All other influent parameters were measured on a weekly basis with the exception being the bacteriological samples which were taken immediately following the daily dosing with stabilization pond effluent. No attempt was made to maintain a specified suspended solids content in the field experiments. Filter influent characteristics, water temperature and the surrounding air temperature were recorded daily along with any unusual operational findings.

Sampling

Laboratory filter effluent samples were collected once weekly. Effluent samples were composited from 2 days of operation. Filter influent samples were collected for analysis on the days corresponding to the effluent composite sample. Prior to collection of the effluent from which the samples would be taken, the effluent collection buckets were thoroughly cleaned.

Raw or influent water samples for bacterial analysis were collected just prior to adding the pond effluent to the filters and analyzed for total bacteria and total coliform bacteria. The following day, effluent samples were then taken and analyzed for total bacteria and total coliform bacteria. The resulting data were used to show the filter's performance with respect to bacteria.

Effluent samples from the field filters were collected once a week and the samples were taken immediately following the application of the pond effluent. A filter influent sample was taken daily.

Analyses

Suspended solids, pH, and temperature measurements were performed on filter influent samples on a daily basis for both the laboratory and the prototype field filters. Filter influent and effluent samples were analyzed once weekly for biochemical oxygen demand (BOD), ammonia, nitrite, nitrate, orthophosphate, total unfiltered phosphorus, suspended solids, and pH. In addition, flask bioassays were performed on the laboratory filter efflu-

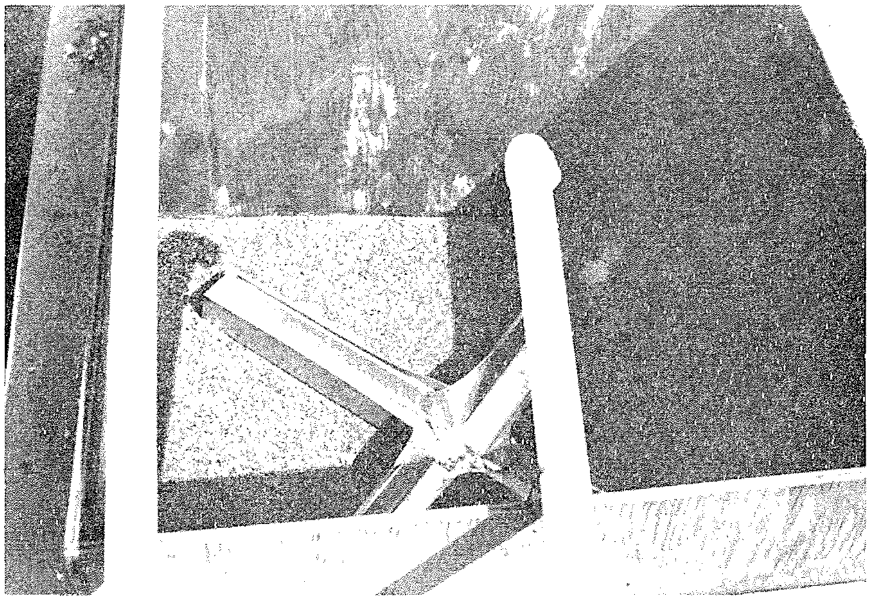

Figure 3. Typical troughs used on the field prototype filters to protect the sand bed and to evenly distribute the applied wastewater over the filter bed.

Table 8. Physical characteristics of the lagoon effluent and the hydraulic loading rates applied to the field filters.

Filter Unit Code	Effective Size of Sand		Filter Depth	Hydraulic Loading Rate	
	mm	inches	in	gpad	m^3/hectare-day
A4	0.17	0.0067	30	400,000	612.72
A5	0.17	0.0067	30	500,000	765.90
A6	0.17	0.0067	30	600,000	919.08
A7	0.17	0.0067	30	700,000	1,072.26
A8	0.17	0.0067	30	800,000	1,225.44
A9[a]	0.17	0.0067	30	900,000[a]	1,378.62
C4	0.72	0.0283	30	400,000	612.72
C5	0.72	0.0283	30	500,000	765.90
C6	0.72	0.0283	30	600,000	919.08

[a]Loaded at this rate for 28 days only.

ents to determine if viable algae cells were in the effluents. Approximately 200 ml of each filter effluent were placed in a 500 ml Erlenmeyer flask and exposed to the lighting pattern described for the laboratory ponds. Growth was measured three to four times weekly in each flask by determining the optical density of the suspension.

Suspended and volatile suspended solids, reactive orthophosphate, reactive nitrite, and reactive nitrate were measured by methods outlined in the Practical Handbook of Seawater Analysis (46). Total phosphorus and biochemical oxygen demand analyses were performed in accordance with Standard Methods (44). Ammonia concentration was determined by methods described in Limnology and Oceanography (43). Total plate counts were made in accordance with Standard Methods (44) with the exception being that all plates were incubated at 20°C for seven days (10). Total coliforms were determined by the procedures described in Standard Methods (44).

RESULTS AND DISCUSSION

The results of Phase I and Phase II of this study are shown in Tables A-1 through A-18 in Appendix A. Information on bacterial effects and ammonia removal and conversion are available for Loading Period III only. These parameters were added to the experimental design because it was felt that this additional data would aid in the evaluation of the field filters.

During Loading Periods I and II filter influent was stored under refrigeration to minimize the effects of storage on the parameters studied. Table 9 shows the average effects of storage on each of the major parameters studied. Changes in the measured characteristics (Table 9) were relatively small and apparently had an insignificant influence on the results of this study.

When the field study was started, the algae concentrations in the Logan City wastewater stabilization ponds were beginning to decrease because of the beginning of the fall season. In most cases, concentrations of algae applied during Loading Period II of the laboratory study are very close to the concentrations experienced in the field. Thus, the comparisons noted between the field study and laboratory study are referenced to laboratory Loading Period II and the field data.

Algae Genera

Laboratory filters

Water applied to the laboratory filters was effluent from domestic wastewater stabilization ponds and many different species of algae were present. *Chlamydomonas sp.* was predominant in both the Logan City and the laboratory stabilization ponds. This was expected because algae were introduced to the model ponds by the periodic addition of the wastewater pond effluent where *Chlamydomonas sp.* were abundant.

During the initial part of the study, filter influent and effluents were examined microscopically to determine the genus of algae present. When the study was expanded to include other chemical analyses, it was necessary to preserve the algae samples for evaluation at a later date. Samples were preserved in International Biological Program Algal Preservative (Appendix B). Analysis of the preserved samples supported the earlier results, i.e., *Chlamydomonas sp.* was the predominate algae in the water applied to the filters. In most cases, the most predominant groups of organisms in the filter effluent were "fusiform diatoms." They appeared at one time or another in all effluent samples studied, and were observed quite regularly in the applied water.

Study of randomly chosen effluents from the laboratory filters, both unpreserved and preserved, showed the presence of *Chlamydomonas sp.* and at times *Scenedesmus sp.* There were cases where *Chlamydomonas sp.*, *Scenedesmus sp.*, and diatoms were observed separately and in various combinations in the effluents. There were a few effluent samples in which algae were not observed. This usually occurred in the 0.17 mm (.0067 inch) effective size sand subjected to the lowest loading rate. When the lagoon effluents were applied to the filters, the algae, *Chlamydomas sp.*, were usually found in a "clumped" or palmeloid state and in the effluent were observed to be single, motile cells in nearly every case. This palmeloid state may have contributed significantly to the removal efficiencies obtained with the filters.

Table 9. The average effect of refrigerated storage on the wastewater applied to the laboratory filters.

Period	Change in Concentration During Refrigeration				
	NO_2 mg/l	NO_3 mg/l	O-PO_4 mg/l	TOT PO_4 mg/l	BOD_5 mg/l
I	+.00065 (5)	+.009 (5)	+.0821 (5)	-.2183 (5)	-.90 (5)
II	[a] +.0162 (7)	+.0229 (7)	-.0007 (7)	-.049 (7)	-.62 (5)

[a]+ denotes increase in concentration of constituent, - denotes decrease in concentration of constitution, and ( ) indicates number of measurements averaged.

Field filters

Microscopic examinations of the field filters influent and effluents were performed to establish the genera of algae being applied and removed and to insure that comparisons between the laboratory and field studies were based upon removing the same algae. Table 10 summarizes the results of the microscopic analyses.

As reported for the laboratory filters, *Chlamydomonas sp.* were again predominant in the filter influent during the second year of the field study. A variety of genera were present in the lagoon effluent, but *Chlamydomonas sp.* represented a minimum of 70 percent of the algal population throughout the study.

Chlamydomonas sp. were also predominant in the effluents but the majority apparently were dead. The majority of the suspended solids in the effluents was debris washed from the filter medium. The source of this debris is discussed in the section on Algae Removal.

Oxidation of Nitrogen

Ammonia concentrations in the influent and effluents were not measured until Loading Period III. This was because it was assumed that the nitrogen form which was available for oxidation by the filters was that of ammonia. Since this was only an assumption, it was decided to measure this parameter to ascertain whether or not ammonia was in fact the nitrogen compound being oxidized. As the applied, effluent and removal values show (Table 11 and Tables A-7 - A-9, Appendix A) ammonia was present in large quantities and was readily oxidized. This is in agreement with earlier results reported at the University of Florida where settled primary sewage was applied to intermittent sand filters (20,22).

Figure 4, Table 12, Tables A-1 - A-9, Appendix A, show the relationship between hydraulic loading rate, sand size, and the effluent nitrate concentration for the three algae concentrations applied (Loading Period I, II, III). The most significant effect was produced by the applied algae concentration. The greater the concentration of applied algae, the greater the effluent concentration of nitrate produced. This was very apparent at the highest concentration of applied algae studied in this case. But, at the lower concentrations of algae studied, the interaction between hydraulic loading rate and algae concentration was much less defined but yet still present. This appears to be opposed to the findings of Grantham, Emerson, and Henry (22), Furman, Calaway, and Grantham (20) and Pincince and McKee (37) in their studies on the relationship between organic loading rates and the nitrification ability of the intermittent sand filter.

Table 10. Algae genera population estimates for the influent and effluent samples from the field filters in 1973 (Phase II).

Sample Date	Genera: Influent Sample: *Anabaena*	Influent Sample: Chlamydomonas Vegetative	Influent Sample: Chlamydomonas Palmelloid	Influent Sample: *Daphnia*	Influent Sample: *Diatom*	Influent Sample: *Euglena*	A5 & C5 Effluent Samples: *Anabaena*	A5 & C5 Effluent Samples: *Chlamy.*	A5 & C5 Effluent Samples: Debris	A5 & C5 Effluent Samples: Diatom
26 July	0%	25%	70%	Occasional	5%	Occasional	0%	85%(dead)	Mainly	15%
2 Aug	0%	25%	70%	Occasional	5%	Occasional	0%	85%(dead)	Mainly	15%
9 Aug	0%	25%	70%	Occasional	5%	Occasional	0%	85%(dead)	Mainly	15%
15 Aug	Occasional	20%	70%	Occasional	5%	5%	0%	85%(dead)	Mainly	15%
22 Aug	Occasional	5%	85%	Occasional	5%	5%	0%	85%(dead)	Mainly	15%
28 Aug	5%	5%	80%	Occasional	5%	5%	0%	85%(dead)	Mainly	15%
7 Sept	10%	10%	75%	Occasional	5%	Occasional	Occasional	85% (dead)	Mainly	15%
13 Sept	15%	10%	75%	Occasional	Occasional	Occasional	5%	80%(dead)	Mainly	15%
19 Sept	20%	5%	70%	Occasional	5%	0%	10%	75%(dead)	Mainly	15%
27 Sept	10%	Occasional	80%	Occasional	10%	0%	0%	80%(dead)	Mainly	20%

Table 11. Mean applied and effluent ammonia nitrogen concentrations obtained during Loading Period III in the laboratory study.

Applied NH_4-N (mg/l)	Effluent NH_4-N Concentration, mg/l — Effective Size of Filter Media								
	0.17 mm			0.35 mm			0.72 mm		
	Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}		
	100	200	300	100	200	300	100	200	300
2.13	.006	.004	.006	.006	.014	.017	.043	.146	.217

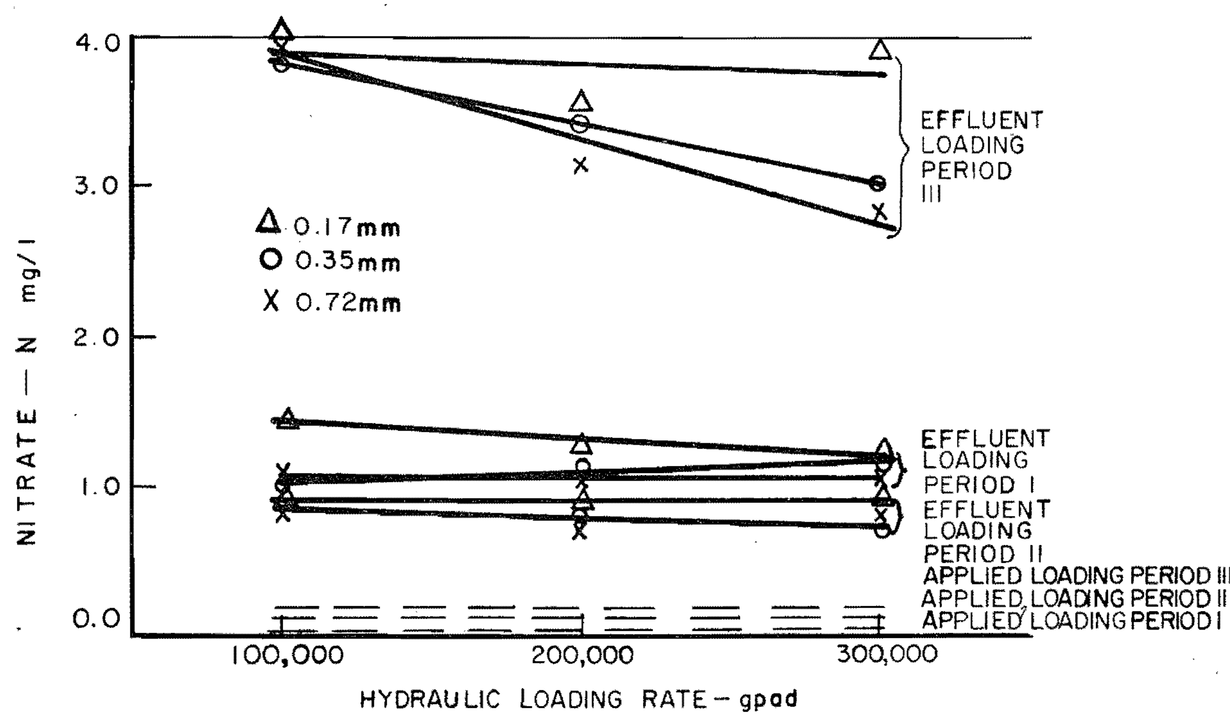

Figure 4. The relationship between hydraulic loading rate and effluent nitrate nitrogen concentration in laboratory filters operated with constant algae concentrations in the influent.

Table 12. Mean applied and effluent nitrate nitrogen concentrations obtained in the laboratory study.

Loading Period	Applied NO_3-N (mg/l)	Effluent NO_3-N Concentration, mg/l — Effective Size of Filter Media: 0.17 mm — Hydraulic Loading Rates, gpad x 10^{-3}: 100	200	300	0.35 mm — Hydraulic Loading Rates, gpad x 10^{-3}: 100	200	300	0.72 mm — Hydraulic Loading Rates, gpad x 10^{-3}: 100	200	300
I	0.034	1.45	1.25	1.202	0.9881	1.12	1.63	1.06	1.02	1.09
II	0.110	0.958	0.910	0.910	0.841	0.805	0.735	0.815	0.709	0.757
III	0.165	4.04	3.57	3.89	3.821	3.44	3.03	3.97	3.17	2.81

During Loading Period III, the ammonia concentration, Table 11, Tables A-7 - A-9, Appendix A, was found to be high in the artificially produced wastewater stabilization pond effluent when compared with concentrations that would be expected to exist in a tertiary treated wastewater stabilization pond effluent (Logan City's) such as that used for study during Loading Periods I and II. Thus, the large increase in nitrification observed during Period III, when compared with that of Periods I and II, was probably caused by the greater amounts of ammonia nitrogen present in the artificially enriched wastewater effluent produced in the laboratory ponds, and was probably not related to the increased applied algae concentrations during Loading Period III.

Filters constructed of sands with smaller effective sizes more readily oxidized ammonia to nitrate, (Table 12). This result agrees with the findings of Grantham, Emerson, and Henry (22), Furman, Calaway, and Grantham (20), and Pincince and McKee (37).

Hydraulic loading rate had little effect on the degree of nitrification produced by the intermittent sand filters. This is especially true for the algae concentrations of Loading Periods I and II. This lack of a relationship is attributed to the low concentrations of unoxidized nitrogen compounds and low suspended solids concentrations being applied to the filters at relatively low hydraulic loading rates for this process. Therefore, surface penetration by the liquid was essentially unrestricted which allowed the bed aeration period at each hydraulic loading rate to be essentially constant. During Period III, although the overall effect of increased algae concentration and ammonia concentration as discussed earlier was still apparent, the effects of increased hydraulic loading at higher organic loadings were also present (Figure 4). The decrease in the oxidation of nitrogen compounds as hydraulic loading rate increased as shown in Figure 4 was caused by increased time that the filter was submerged and the decreased aeration of the bed. These effects would most likely become more noticeable at higher applied suspended solids concentrations and higher hydraulic loadings.

Figure 5, Table 11, and Tables A-7 - A-9, Appendix A, show the changes in ammonia-nitrogen concentrations at the three hydraulic loading rates and filter sand sizes. The 0.72 mm (.0283 inch) effective size sand filter showed a slight decrease in ammonia-nitrogen reduction as the hydraulic loading rate increased. This decrease was probably caused by increased submergence, decreased aeration, and a reduction in the contact time within the filter bed.

Field experimental results for 1972, Tables 13 and 14, and Tables A-10 - A-12, Appendix A, were in agreement with the results observed in the laboratory filters. The 0.17 mm (.0067 inch) effective size sand was somewhat more efficient in the oxidation of ammonia-nitrogen than the 0.72 mm (.0283 inch) sand. Ammonia-nitrogen oxidation was not continued in the field study during 1973; however, as the hydraulic loading is increased, a corresponding decrease in oxidation would be expected.

The rock filtering media, Tables 13 and 14, oxidized little of the ammonia-nitrogen to nitrate. This is probably due to the short time required for the liquid to pass

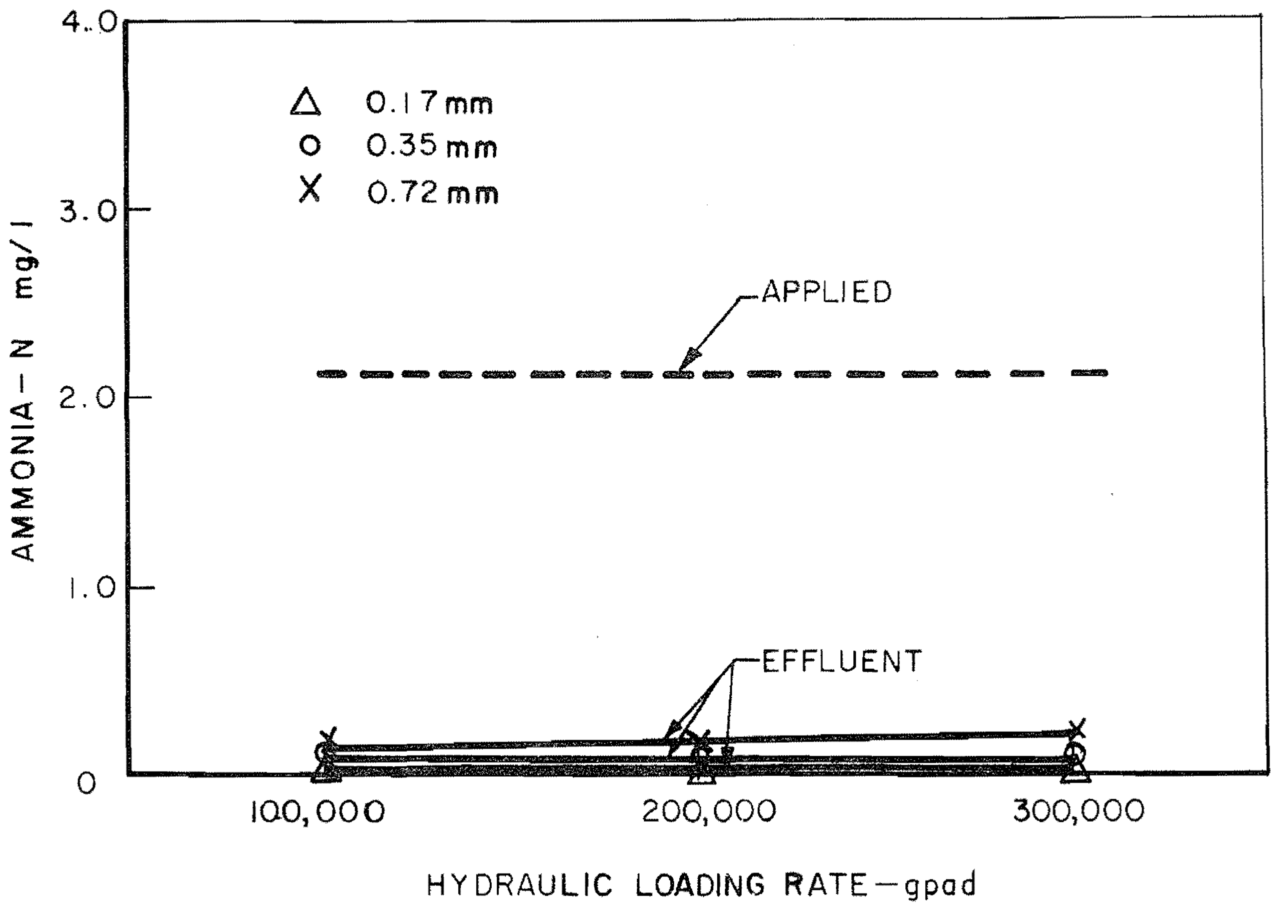

Figure 5. The relationship observed under laboratory conditions between hydraulic loading rate and ammonia nitrogen removal during Loading Period III.

Table 13. Mean applied and effluent ammonia nitrogen concentrations obtained in the field study in 1972.

Applied NH_4-N (mg/l)	Mean Effluent NH_4-N Concentrations, mg/l		
	.17 mm	.72 mm	6 mm max. dia. rock
1.09	0.013	0.426	1.10

Table 14. Mean applied and effluent nitrate nitrogen concentrations obtained in the field study in 1972.

Applied NO_3-N (mg/l)	Mean Effluent NO_3-N Concentrations, mg/l		
	.17 mm	.72 mm	6 mm max. dia. rock
0.078	0.996	1.11	0.479

through the media. Also, the media retains little moisture which makes it difficult to maintain biological life.

BOD Removal

Laboratory filters

As shown in Table 15 and Tables A-1 - A-9, Appendix A, the concentration of BOD_5 in the lagoon effluent applied to the laboratory filters was close to the existing Utah standard of 5 mg/l even before filtration during Loading Periods I and II. This was caused by two factors: the necessity to dilute the effluent to obtain the desired suspended solids concentration applied to the filters, and the high degree of BOD_5 removal produced by the 5-stage Logan lagoon system. During Period III, the measured BOD_5 values of the applied water were not directly related to the applied algae concentration, but in this case were much higher than expected.

Applied BOD_5 values for Periods I and II were almost equal; therefore, only data for Periods II and III were plotted (Figures 6 and 7) to show the relationships

Table 15. Mean applied and effluent BOD_5 concentrations obtained in the laboratory study.

Loading Period	Applied BOD_5 (mg/l)	Effluent BOD_5 Concentration, mg/l — Effective Size of Filter Media: 0.17 mm — Hydraulic Loading Rates, gpad x 10^{-3}			0.35 mm — Hydraulic Loading Rates, gpad x 10^{-3}			0.72 mm — Hydraulic Loading Rates, gpad x 10^{-3}		
		100	200	300	100	200	300	100	200	300
I	6.71	1.15	1.55	2.31	2.51	2.61	2.97	2.89	3.09	3.01
II	6.34	1.17	1.26	1.96	2.44	2.08	2.41	2.33	2.50	1.93
III	36.5	5.81	5.64	7.14	11.21	10.83	11.53	12.26	12.72	13.25

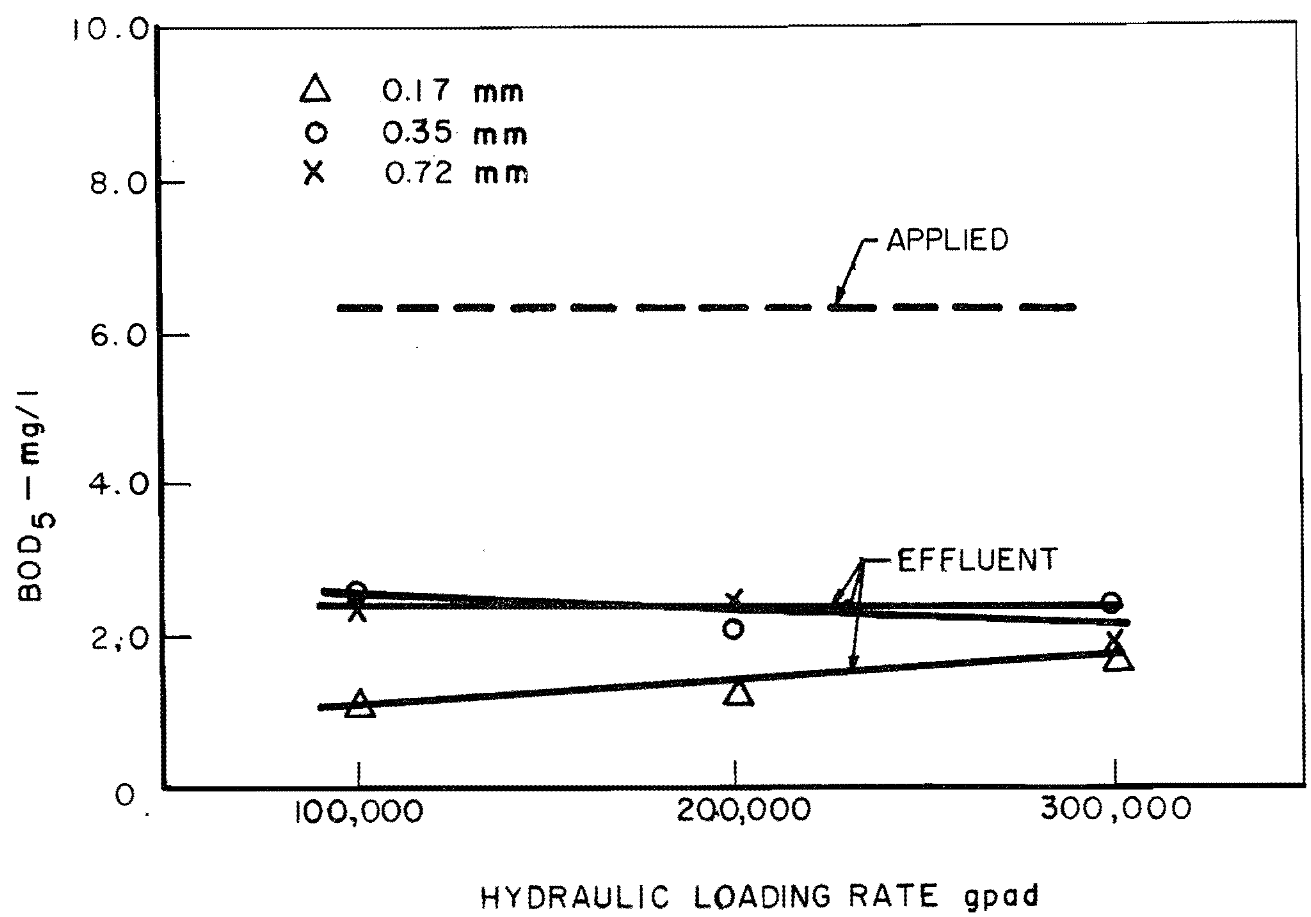

Figure 6. The relationship observed under laboratory conditions between hydraulic loading rate and BOD removal during Loading Period II.

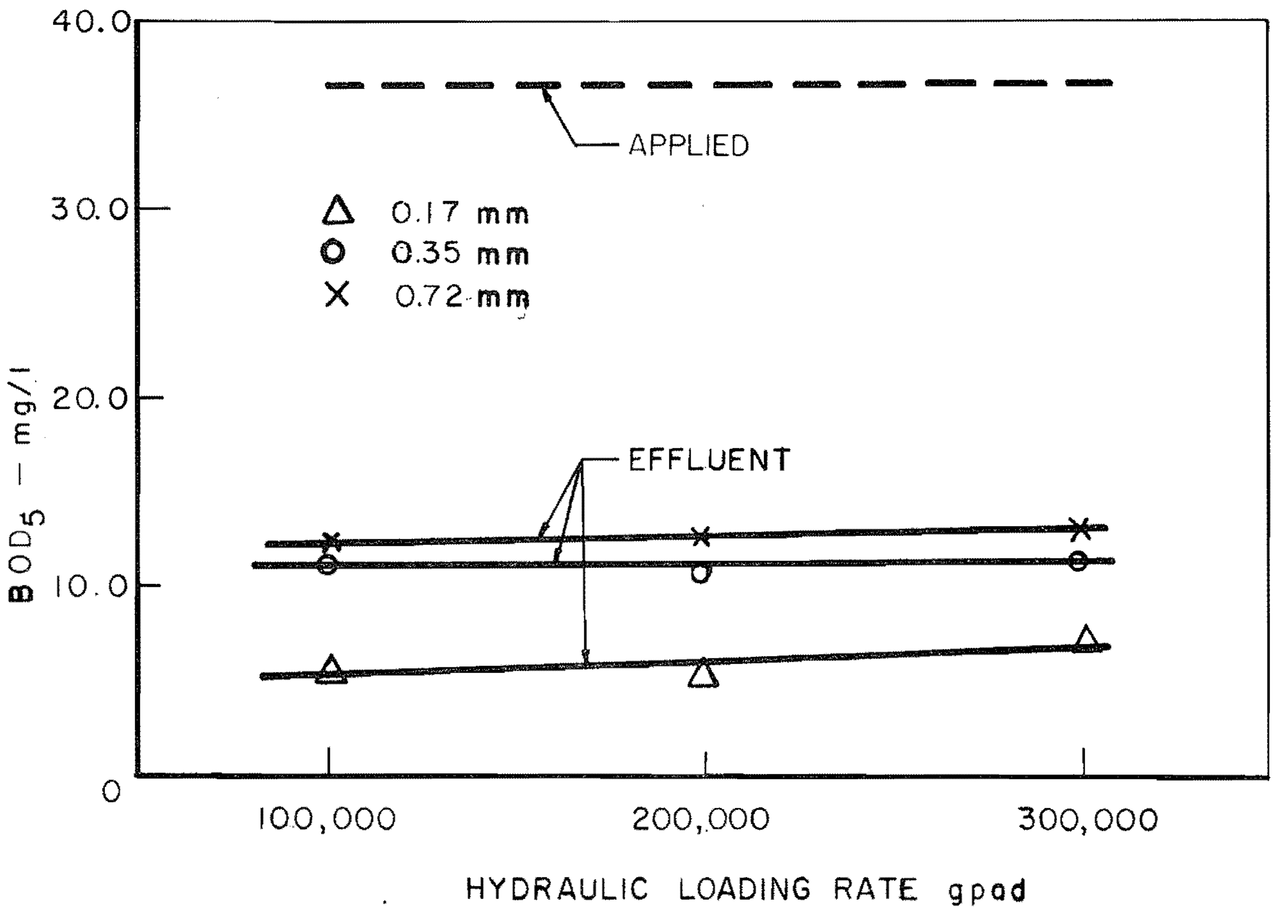

Figure 7. The relationship observed under laboratory conditions between hydraulic loading rate and BOD removal during Loading Period III.

between BOD_5 removal, sand size, and loading rate. Fortunately, it appears that the BOD_5 concentrations studied are representative of typical secondary effluents normally discharged in the State of Utah (33).

Effluents from these filters were well into the nitrogenous BOD stage as were those studied by Grantham et al. (22). The BOD_5 measurements made during this study did not appear to be erratic as shown in Table 15 during any of the three loading periods; therefore, it is felt that the comparisons and conclusions drawn are representative.

Results of the laboratory study were in good agreement with results obtained by Grantham et al. (22). Examination of Figures 6 and 7 and Table 16 shows that the loading rates used had little effect on BOD_5 removal. However, the data show a trend toward an increase in the concentration of BOD_5 in the effluent as the loading rate increased. Higher loadings would probably show an even greater increase in effluent BOD_5 concentrations for all sand sizes. With respect to sand size, the effect of loading rate does slightly decrease the filters's ability to remove the applied BOD_5 which agrees with the findings of Grantham et al. (22). As noted by Grantham, when the depth of bed is greater than 12 inches, hydraulic loading rate has little effect on the BOD removal (22).

Field filters

At the same hydraulic loading rates as those employed in the laboratory filters, BOD_5 removals obtained in the field units in general agreed with the laboratory findings with the exception being the lower removal efficiencies obtained with the 0.72 mm (.0283 inch) sand used in the field (Table 17 and Tables A-10 - A-12, Appendix A). The differences in performance summarized in Table 16 were probably caused by the 10 - 20°F greater operating temperature under laboratory conditions. In general, lower air temperatures produce filter effluents with higher BOD_5. This effect was even more pronounced in larger sized sands studied by Grantham (22). Coarser sands allow better aeration which would allow the air temperature to exert a much greater effect on the biological activity.

Table 16. The comparison of BOD_5 removal for the laboratory filters during Loading Period II and the field filters containing 0.72 mm (.0283 inch) size sand.

Hydraulic Loading (gpad)	Percent BOD_5 Removal Under Laboratory Conditions (70° F ave. air)	Percent BOD_5 Removal Under Field Conditions (60° F ave. air)
100,000 (153.4 m^3/hectare-day)	63.2	24.3
200,000 (306.4 m^3/hectare-day)	59.6	17.8
300,000 (459.5 m^3/hectare-day)	69.6	29.6

Table 17. Mean applied and effluent BOD_5 concentrations obtained in the field study.

Applied mg/l	Average Effluent BOD_5 Concentrations, mg/l		
	0.17 mm	0.72 mm	6 mm max. dia. rock
6.18	1.07	4.70	4.92

The mean monthly influent and effluent BOD_5 concentrations obtained during the second year of field operation at various hydraulic loading rates for the two effective size sands (0.17 and 0.72 mm) are presented in Table 18. Individual values of the BOD_5 concentrations are presented in Appendix A, Tables A-17 and A-18.

The BOD_5 of the influent remained essentially constant during the second year of operation ranging from 10.0 to 24.9 mg/l with an average value of 13.7 mg/l (Tables A-17 and A-18) and a median value of 12.5 mg/l. Figure 8 shows the relationship between hydraulic loading rate and mean BOD_5 concentrations in the effluents for the 0.17 and 0.72 mm effective size sands. There appears to be little variation in the effluent BOD_5 concentration with hydraulic loading rate for the 0.72 mm effective size sand; whereas, the 0.17 mm effective size sand shows a definite increase in effluent BOD_5 concentration as the hydraulic loading rate was increased. It is very likely that the effluent BOD_5 concentration would also increase for the 0.72 mm sand sizes if the loadings were increased to 0.7 and 0.8 mgad.

The mean effluent BOD_5 concentrations for the 0.17 mm effective size sand filters loaded at 700,000 and 800,000 gpad (1072.3 and 1225.4 m^3/hectare-day) was twice as high as the values obtained at a hydraulic loading rate of 600,000 gpad (Figure 8). A higher effluent concentration was expected, but whether such a large increase would have occurred if all of the filters had operated for an equal time period is unknown. However, based upon the data collected in this study it appears that BOD_5 removal efficiency reaches a limit in the vicinity of a hydraulic loading rate of 600,000 gpad.

BOD_5 reductions with the 0.17 mm effective size sand filters ranged between 38.7 and 97.4 percent with the lower reductions occurring principally at the higher hydraulic loading rates (700,000 and 800,000 gpad). BOD_5 reductions obtained with the 0.72 mm effective size sand appeared to be independent of the hydraulic loading rate and ranged between 27.0 and 80.7 percent.

Mean BOD_5 reductions for the 1973 season for the 0.17 mm sand ranged from 70.4 percent at a hydraulic loading rate of 800,000 gpad to 88.4 percent for the 400,000 gpad rate. Mean BOD_5 reductions for the 0.72 mm sand were essentially constant for all hydraulic loading rates and ranged from 59.9 to 63.2 percent.

Phosphorus Removal

Phosphorus has been suggested to be the limiting nutrient associated with algal blooms occurring in wastewater stabilization ponds. Ziebel and Hallock (50) found this to be the case in their study on the feasibility of using tertiary treated wastewaters to produce game fish. They found that if this nutrient was readily available in concentrations exceeding 0.5 mg/l, algal growth would easily occur, die, and deplete the oxygen supply resulting in fish kills.

The results of phosphorus removal in the laboratory filters are summarized in Table 19 and listed in Tables A-1 - A-9, Appendix A. Phosphorus was removed by the intermittent sand filters studied, but as shown in Figure 9, removal was greatly affected by the length of time that the units had operated and the hydraulic loading rate. Because little biological growth occurs on or in the filter as the water passes through, it is unlikely that any

Table 18. Mean influent and effluent BOD_5 concentrations obtained with each sand size and hydraulic loading rate during Phase II under field conditions.

Month	Mean Monthly Influent BOD Concentration (mg/l)	Mean Monthly Effluent BOD_5, mg/l															
		Effective Size, 0.17 mm										Effective Size, 0.72 mm					
		Hydraulic Loading Rates, gpad										Hydraulic Loading Rates, gpad					
		400,000		500,000		600,000		700,000		800,000		400,000		500,000		600,000	
		mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.
June	12.1	0.75	93.8	1.2	90.1	1.3	89.3	-	-	-	-	4.5	62.8	3.6	70.2	3.9	67.8
July	12.6	1.5	88.1	0.87	93.1	1.1	91.3	3.5	72.2	3.3	73.8	5.4	57.1	5.7	54.8	5.9	53.2
Aug.	12.9	2.2	82.9	3.1	76.0	3.1	76.0	4.2	67.4	4.3	66.7	6.2	51.9	5.8	55.0	6.8	47.3
Sept.	16.1	2.0	87.6	1.7	89.4	1.8	88.8	3.4	78.9	4.7	70.8	5.9	63.4	5.1	68.3	5.4	66.5

Table 19. Mean applied and effluent total unfiltered phosphorus concentrations obtained in the laboratory study.

Loading Period	Applied Total Phosphorus-P mg/l	Effluent Total Phosphorus-P Concentrations, mg/l								
		Effective Size of Filter Media								
		0.17 mm			0.35 mm			0.72 mm		
		Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}		
		100	200	300	100	200	300	100	200	300
I	0.832	0.029	0.083	0.166	0.100	0.174	0.258	0.257	0.339	0.341
II	1.18	0.056	0.340	0.489	0.285	0.448	0.553	0.365	0.639	0.669
III	3.00	0.769	1.56	1.90	0.986	1.71	2.00	1.49	2.10	2.20

significant phosphorus removal is obtained through growth needs. Therefore, the most obvious explanation of the relatively large phosphorus removals obtained at the beginning of the experiments was ion exchange. The sands contained some forms of carbonate which probably served as the exchange medium. Figure 10 and Table 18 show that phosphorus removal was affected by the sand size. The finer sands have a much greater surface area and would have more ion exchange sites than the coarser sands. The relationship between phosphorus removal and hydraulic loading rate is apparent since the more water that was applied to a filter, the faster the available exchange sites would be filled. Thus, as shown in Figure 10, phosphorus removal decreased as the loading rate increased. Also, the hydraulic loading rate would dictate the rate at which phosphorus removal would approach equilibrium (Figure 10).

Phosphorus removals in the field units followed the same pattern observed in the laboratory and the results are summarized in Table 20 and listed in Tables A-10 - A-12, Appendix A. The 0.72 mm (.0283 inch) effective size sand field filters became saturated with phosphorus in much less time than it took under controlled laboratory conditions. This is attributed to the practice of replacing the top 4 inches (10 cm) of sand in the laboratory units with new, clean sand between each loading period; whereas, the field sand was not removed during seven weeks of operation.

Therefore, phosphorus removal studies were not conducted on the field units during the second year. Because of the trends established at lower hydraulic loading rates, it was concluded that little phosphorus removal would occur.

Algae Removal

Algae concentrations in the influent were estimated by the suspended solids technique which measures a variety of organisms, inert suspended matter, and a number of various algae species. Effluent algae concentrations were also estimated as volatile suspended solids to overcome the disadvantages of the silts and clays washed from the filters during the early stages of the study.

Laboratory filters

Suspended and volatile suspended solids concentrations applied and in the effluents of the laboratory filters are shown in Tables 21 and 22 and Figures 11-13 for the various hydraulic loading rates and sand sizes employed. The suspended and volatile suspended solids removals were independent of the hydraulic loading rates employed. However, after the silt and clay were removed, it appeared that a general trend was developing which

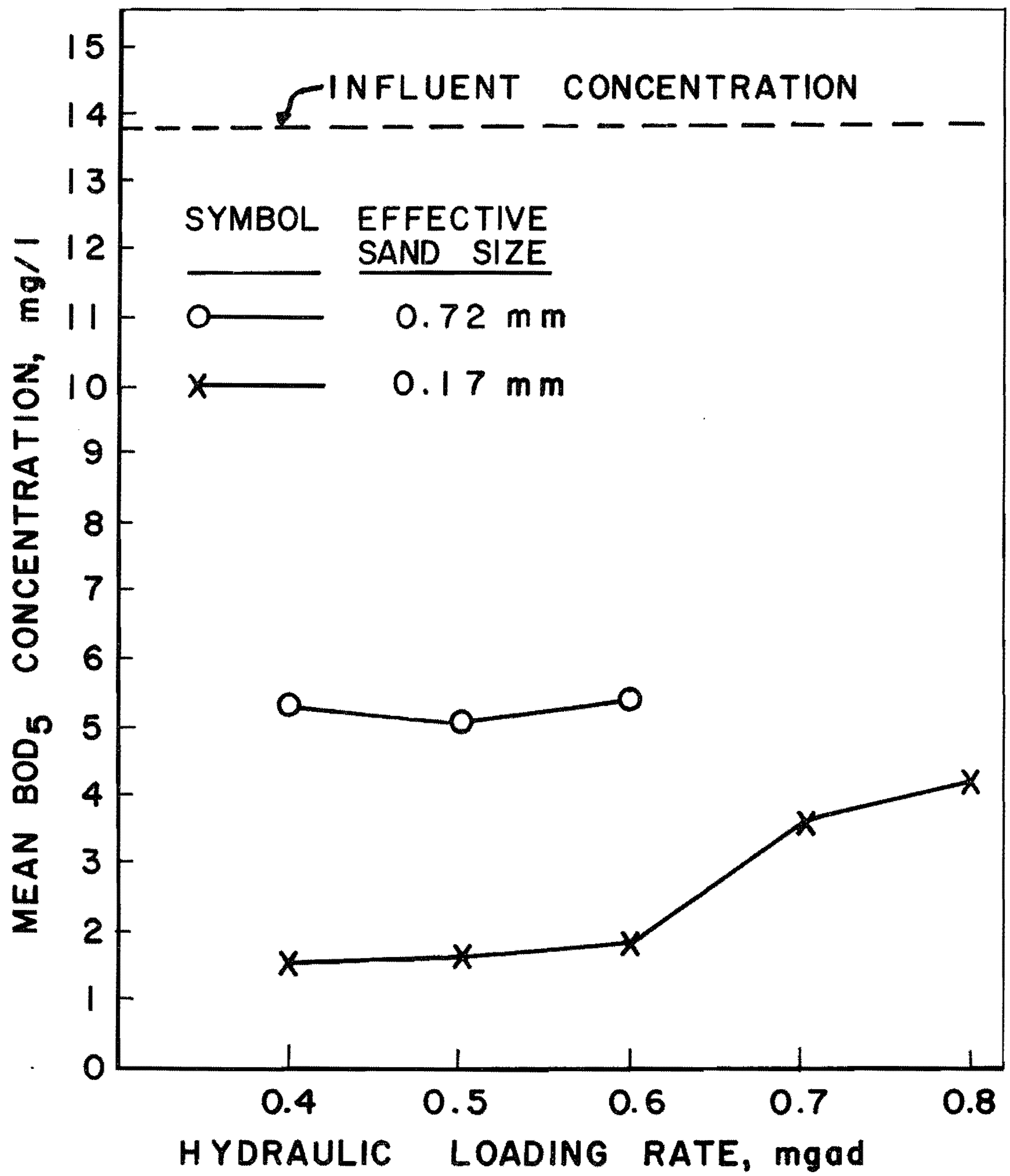

Figure 8. The relationship observed under field conditions between hydraulic loading rate and BOD_5 removal for the 0.17 and 0.72 mm effective size sands.

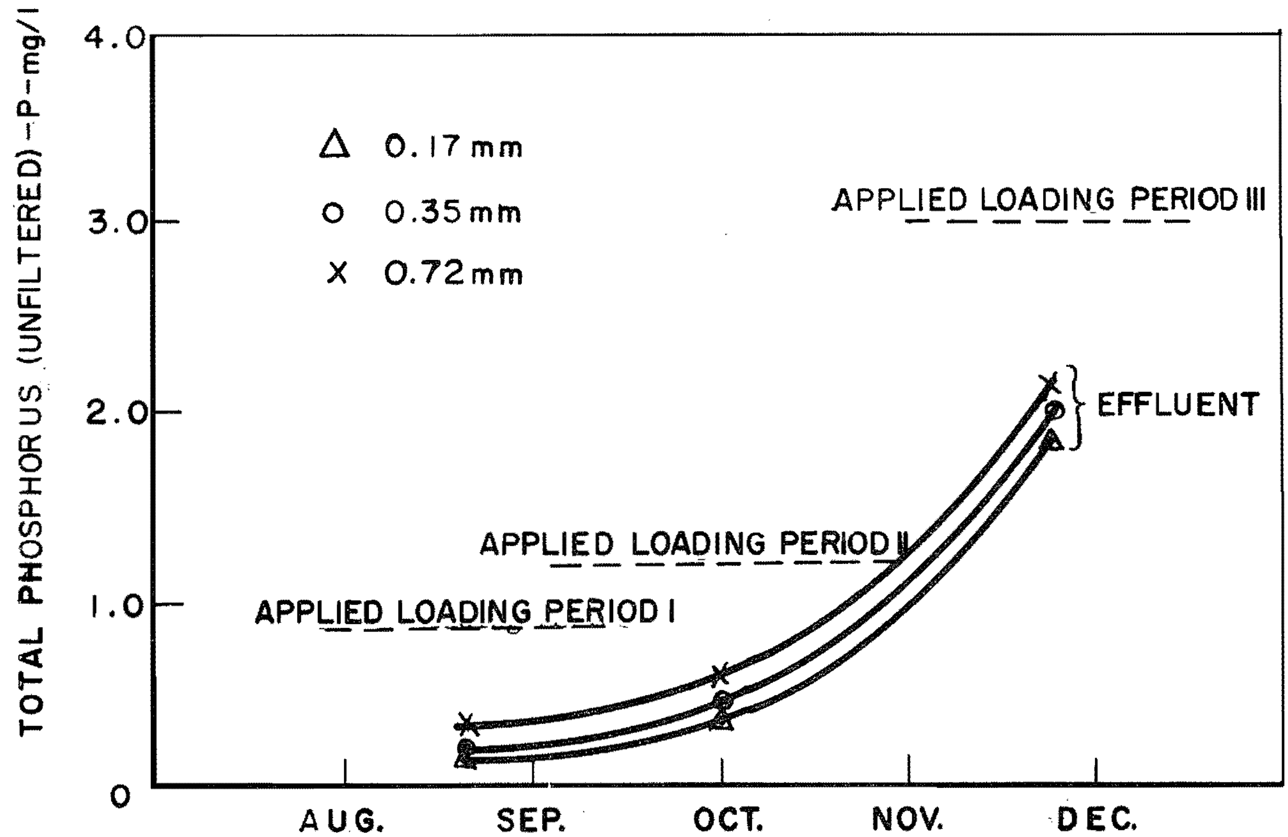

Figure 9. **The relationship observed under laboratory conditions between time and the concentration of total phosphorus-P (unfiltered) in the effluent of the filters subjected to a hydraulic loading rate of 300,000 gpad (459.5 m3/hectare-day).**

indicated an increase in effluent solids concentration as the hydraulic loading was increased, particularly when greater concentrations of suspended solids were applied.

Suspended solids removals were directly related to the effective size of the sands at the higher solids loading rates. At lower loadings the removals obtained on the 0.72 mm (.0283 inch) sand were approximately equal to the removals obtained with the 0.35 mm (.0137 inch) filters.

Table 20. Mean applied and effluent total phosphorus concentrations obtained in the field study during the first year of operation.

Applied Total Phosphorus-P mg/l	Effluent Total Phosphorus-P Concentrations, mg/l		
	0.17 mm	0.72 mm	6 mm max. dia. rock
1.41	0.379	0.889	1.27

A 3 x 3 factorial statistical design was employed to separate the interactions between the variables studied, and it was found that:

1. Applied algae concentration and sand size were significant factors, hydraulic loading rate was not.
2. Sand size was found to be insignificant when the applied algae concentrations were 15 mg/l and 30 mg/l, but sand size was found to be a significant factor at the 45 mg/l algae concentration.

These results were probably caused by the relatively low loading rates employed, and if the laboratory experiment had been expanded to include higher loadings, it is likely that entirely different conclusions would have been obtained.

Some algae passed through the entire depth of the filter bed as verified by microscopic examination of the effluents. Borchart (8), Ives (28), and Folkman and Wachs (16) have reported similar results. Figure 14 shows that percent removal efficiencies increased with the application of higher algae concentrations, but more algae passed the filter than at the lowest applied concentration. Flask bioassay results are presented later in an attempt to study the ability of those algae present in the effluent to grow.

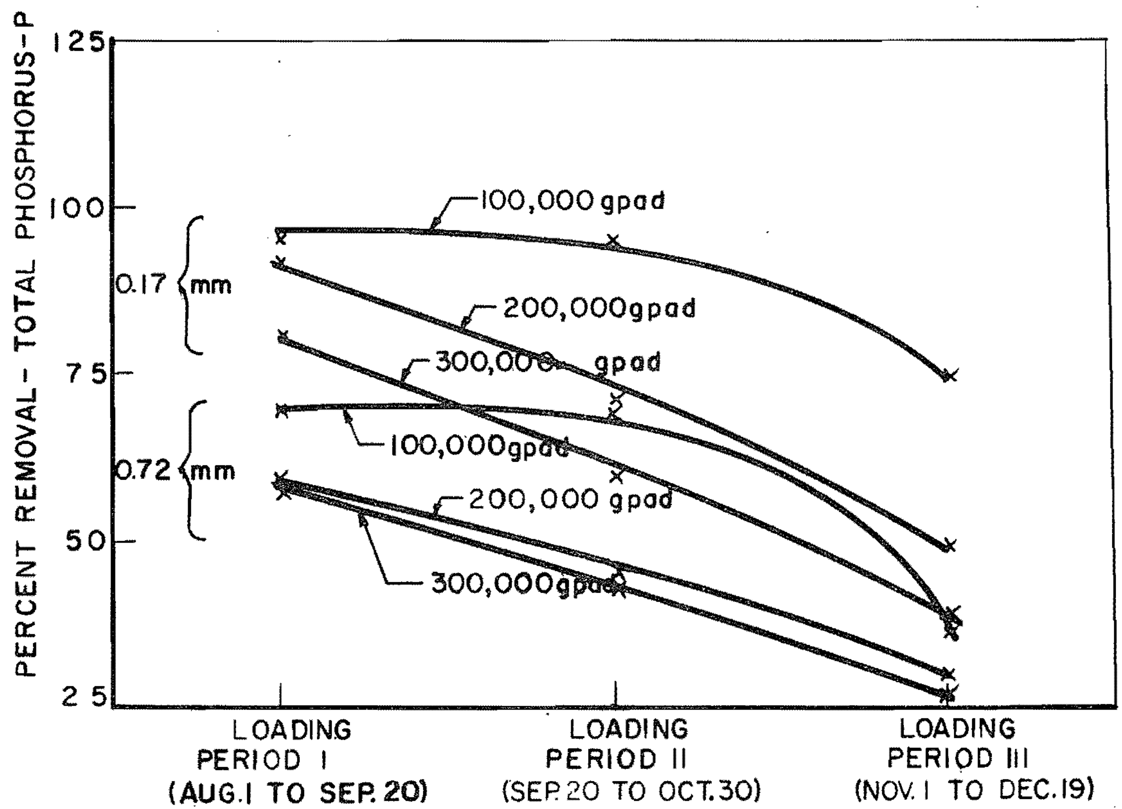

Figure 10. The relationships between total phosphorus removal and the time of operation, hydraulic loading rate, sand size, and an increase in applied phosphorus concentration are shown for laboratory conditions.

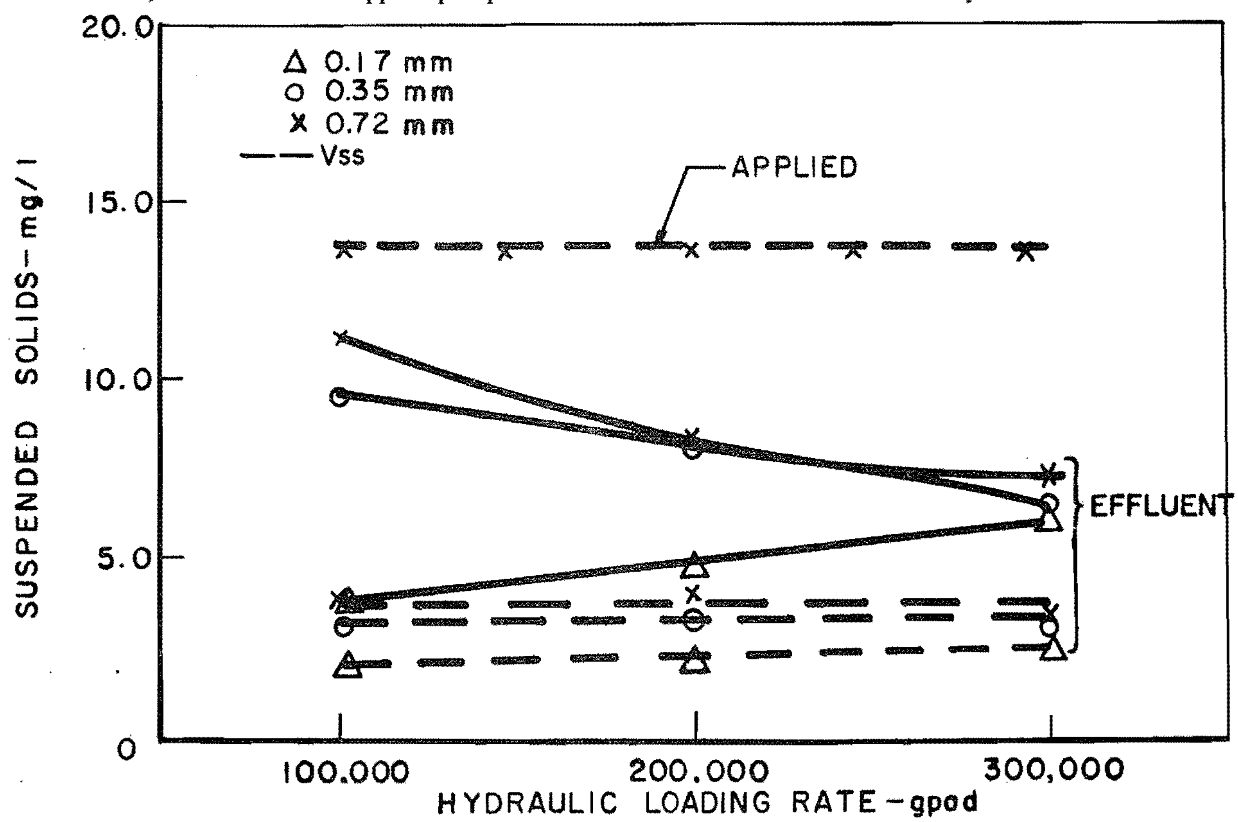

Figure 11. The relationship between the hydraulic loading rate and the effluent suspended and volatile suspended solids concentration is shown for Loading Period II in the laboratory study.

Table 21. Mean applied and effluent suspended solids concentration obtained in the laboratory study.

Loading Period	Applied Suspended Solids mg/l	Effluent Suspended Solids Concentrations, mg/l								
		Effective Size of Filter Media								
		0.17 mm			0.35 mm			0.72 mm		
		Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}		
		100	200	300	100	200	300	100	200	300
I	31.0	5.53	7.93	11.2	10.6	10.9	12.8	13.6	11.9	11.0
II	13.7	3.96	4.80	6.05	9.39	8.19	6.50	11.0	8.15	7.28
III	46.3	1.86	1.93	5.33	9.47	11.9	13.7	16.6	15.9	16.5

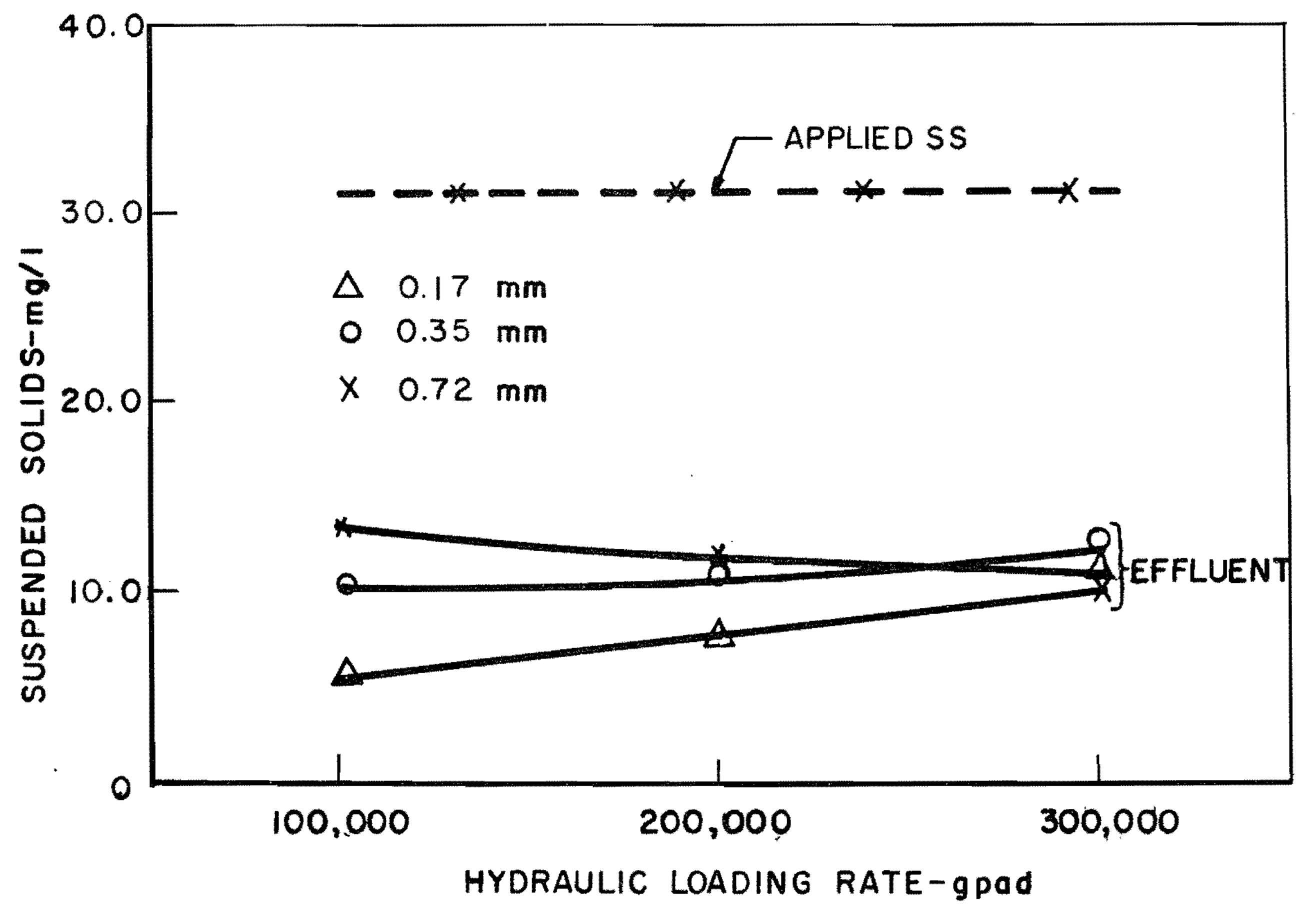

Figure 12. The relationship between the hydraulic loading rate and the effluent suspended solids concentrations is shown for Loading Period I of the laboratory study.

Table 22. Mean applied and effluent volatile suspended solids concentrations obtained in the laboratory study.

Loading Period	Applied Volatile Suspended Solids mg/l	Effluent Volatile Suspended Solids Concentration, mg/l								
		Effective Size of Filter Media								
		0.17 mm			0.35 mm			0.72 mm		
		Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}		
		100	200	300	100	200	300	100	200	300
II	9.16	1.99	2.14	2.30	3.38	3.33	3.40	3.85	4.00	3.17
III	41.3	1.46	1.70	3.48	7.28	7.14	8.31	10.1	13.1	13.2

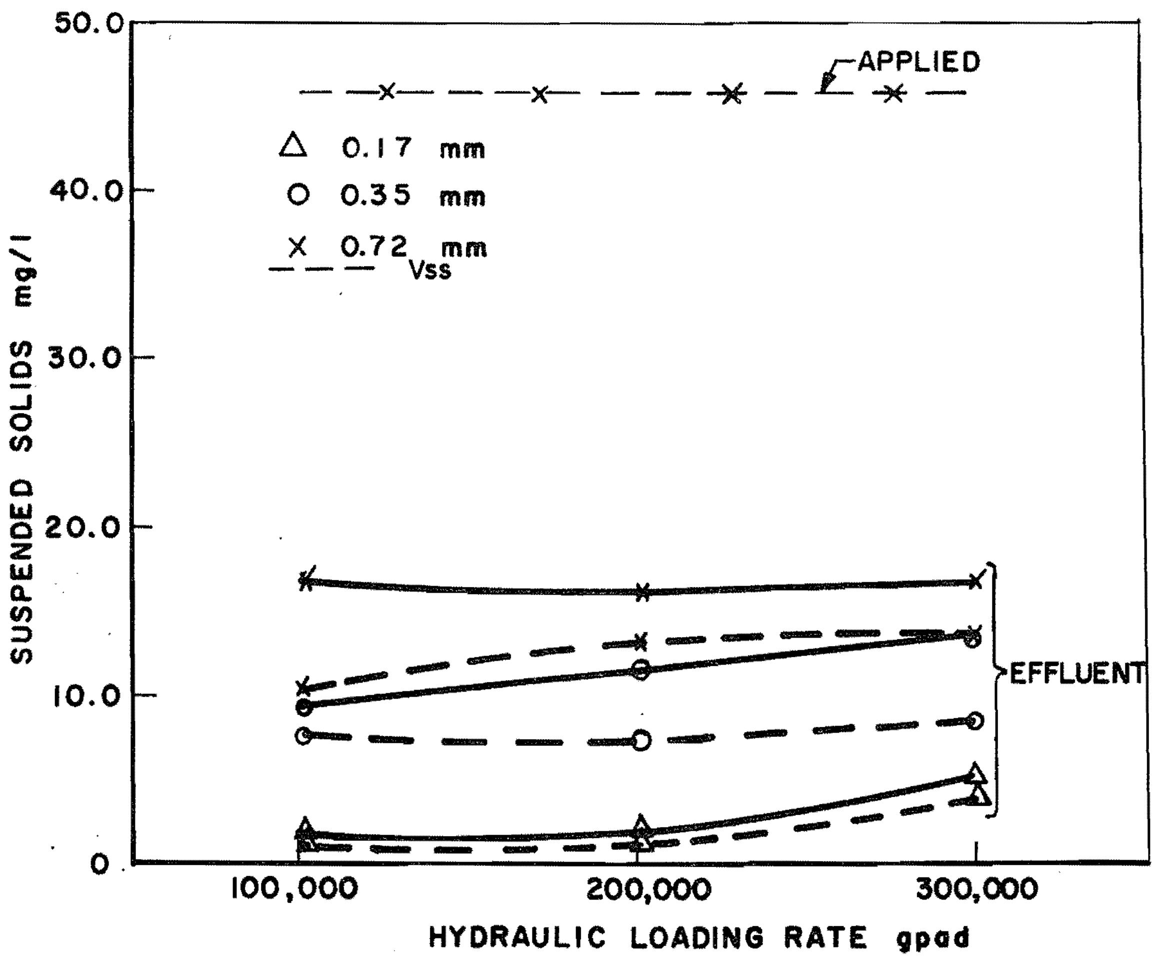

Figure 13. The relationship between the hydraulic loading rate and the effluent suspended and volatile suspended solids concentrations is shown for Loading Period III of the laboratory study.

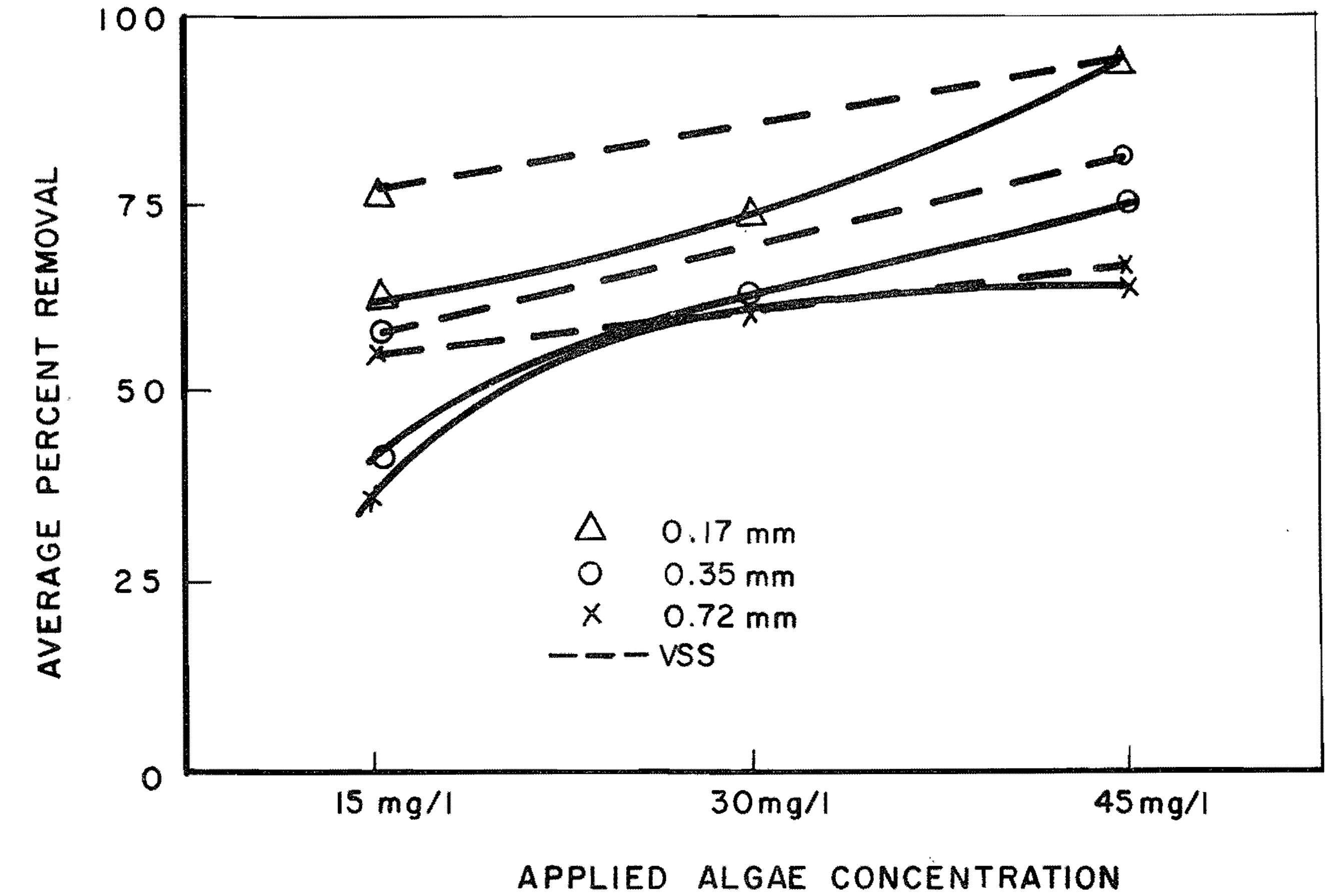

Figure 14. The relationship between applied algae concentration, expressed as suspended solids and volatile suspended solids, and the resulting mean removal efficiency for the three sands and three concentrations of algae applied is shown for the laboratory study. Plotted points are mean effluent concentrations resulting from the three hydraulic loading rates applied to each sand size during each Loading Period.

Field filters

Algal removals by the field filters during the first year of operation are listed only in Tables A-10 - A-12, Appendix A, because the silt and clay that was washed from the filters made interpretation of the results impossible. Attempts to correct mechanical and spreading problems increased the amount of silts and clay in the effluents. Clean water was not available in the field to prewash the filters as was employed to remove silts and clays from the laboratory filters. Due to these problems, first year results of the suspended solids analysis were concluded to be not representative of the process.

During the second year of operation, algal concentrations were also estimated by measuring fluorescence[1] and by determining suspended and volatile suspended solids. Linear regression analyses of the solids and fluorescence measurements produced a linear relationship significant at the 1 percent level (Appendix C).

Influent and effluent algae concentrations expressed as suspended solids produced by the field filters are summarized in Table 23. Volatile suspended solids concentrations are shown in Table 24. Detailed data are presented in Tables A-13 through A-16, Appendix A. Data for the 0.17 mm effective size sand filter loaded at 900,000 gpad are not presented because of the relatively short period of operation. However, algae removals were similar to those obtained with the 800,000 gpad loading rate.

Figure 15 shows the relationship between the hydraulic loading rates and the suspended and volatile suspended solids concentrations in the filter effluents for the 0.17 and 0.72 mm effective size sands. Algae removal apparently is independent of hydraulic loading rate up to a loading of approximately 600,000 gpad.

Effluent suspended solids concentrations for the months of May and June 1973 were much greater than

[1]G. K. Turner Associates, Palo Alto, California.

Table 23. **Mean influent and filter effluent algae concentrations measured as suspended solids for each sand size and hydraulic loading rate studied during Phase II under field conditions.**

Month	Mean Monthly Influent Algae Conc. as Suspended Solids (mg/l)	Mean Monthly Effluent Suspended Solids Concentrations and Percent Removals															
		Effective Size Sand, .17 mm										Effective Size Sand, .72 mm					
		Hydraulic Loading Rate - gpad										Hydraulic Loading Rate - gpad					
		400,000		500,000		600,000		700,000		800,000		400,000		500,000		600,000	
		mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.
May	5.0	25.1	-	56.0	-	20.9	-	-	-	-	-	31.7	-	7.5	-	15.9	-
June	6.5	15.7	-	38.9	-	14.5	-	-	-	-	-	11.6	-	9.4	-	12.5	-
July	29.8	14.2	52.3	23.9	19.8	20.2	32.2	21.4	28.2	15.4	48.3	17.9	39.9	14.4	51.7	16.9	43.3
Aug	44.2	23.2	47.5	18.8	57.5	30.0	32.1	34.5	21.9	39.1	18.6	33.0	25.3	22.4	49.3	26.9	39.1
Sept	25.2	8.7	65.5	13.6	46.0	8.8	65.1	20.5	18.7	16.5	34.5	12.4	50.8	12.4	50.8	11.4	54.8

the concentrations in the effluents. This was attributed to the washing of filter and clay from the filter sand. Filter media were produced from pit run sands containing large quantities of fines and clays that were easily washed from the filters. As mentioned above, clean water was not available to prewash the filters; therefore, it was necessary to wash with effluent. Then an attempt was made to compensate for the silt and clay. Much more material was washed from the 0.17 mm effective size sand because much of the fines were removed when preparing the 0.72 mm sand by screening.

At the 500,000 gpad hydraulic loading rate, monthly mean volatile suspended removals were essentially equal for the 0.17 and 0.72 mm effective size sands. Efficiencies fluctuated considerably from one sand to the other during the study period. But in general the 0.17 mm effective size sand produced a better quality effluent, particularly at the 600,000 gpad loading rate. Volatile suspended solids removal efficiencies appeared to be improving with the age of the filters, which is probably related to the washing of debris from the units (Table 24).

Examination of the effluent suspended solids concentrations at various hydraulic loading rates shown in Figure 15 indicates that the 0.72 mm filters were more efficient. However, the volatile suspended solids data show just the opposite. Again, this discrepancy is explained by the washing of silt and clay into the effluents.

Laboratory bioassay results indicated that as greater concentrations of algae were applied to the filters more viable cells passed through the 30 inches of sand. As shown in Table 24, during August when the algae concentration was at a maximum, more algae as volatile suspended solids passed the filters.

Although removal efficiencies appeared to improve with the age of the project, noticeable increases in removal efficiencies as the filters approached plugging did not occur. This is counter to the laboratory results and cannot be readily explained except by the variation normally occurring in solids analyses.

Bacterial Removal

Stream standards recently adopted by the State of Utah include acceptable levels for both total and fecal coliform organisms. In order to eliminate many variables that would be encountered by evaluating this process based on stream standards, for purposes of future discussion of this process, Class "C" standards will be assumed to be discharge standards. The standards require that a Class "C" water have an arithmetic monthly mean value of total and fecal coliform that does not exceed 5,000 and 2,000 per 100 ml, respectively (33).

Laboratory filters

Coliform removal data for the laboratory filters are presented in Tables A-10 - A-12 of Appendix A and Table 25. Total coliform removals of better than 86 percent were obtained with all three sand sizes but due to the high applied counts the process was not able to meet the earlier noted standards in this particular application. Even at removals above 95 percent, lesser numbers of applied coliforms would have to be applied in order to meet Class "C" discharge standards. Calaway, Carroll, and Long (10) presented similar results.

As the effective size of the sand was decreased, coliform removals increased, which agrees with the findings of Calaway, Carroll, and Long (10). But, at the hydraulic loading rates employed, total coliform removals with the 0.72 mm (.0283 inch) sand were equal to those obtained in the filters containing 0.35 mm (.0137 inch) sand.

Total coliform removals were independent of the hydraulic loading rates employed, but it is doubted that this would apply at higher loadings. Calaway, Carroll, and Long (10) found that at hydraulic loading rates approximately twice the rates used in this study that bacteria penetrated the bed to much greater depths. Therefore, more bacteria would be expected to pass the filter at higher loading rates. However, at the hydraulic loading rates of 100,000 (153.4 m^3/hectare-day), 200,000 (306.4 m^3/hectare-day), and 300,000 (459.5 m^3/hectare-day) gallons per acre per day, the coliforms removal was independent of loading.

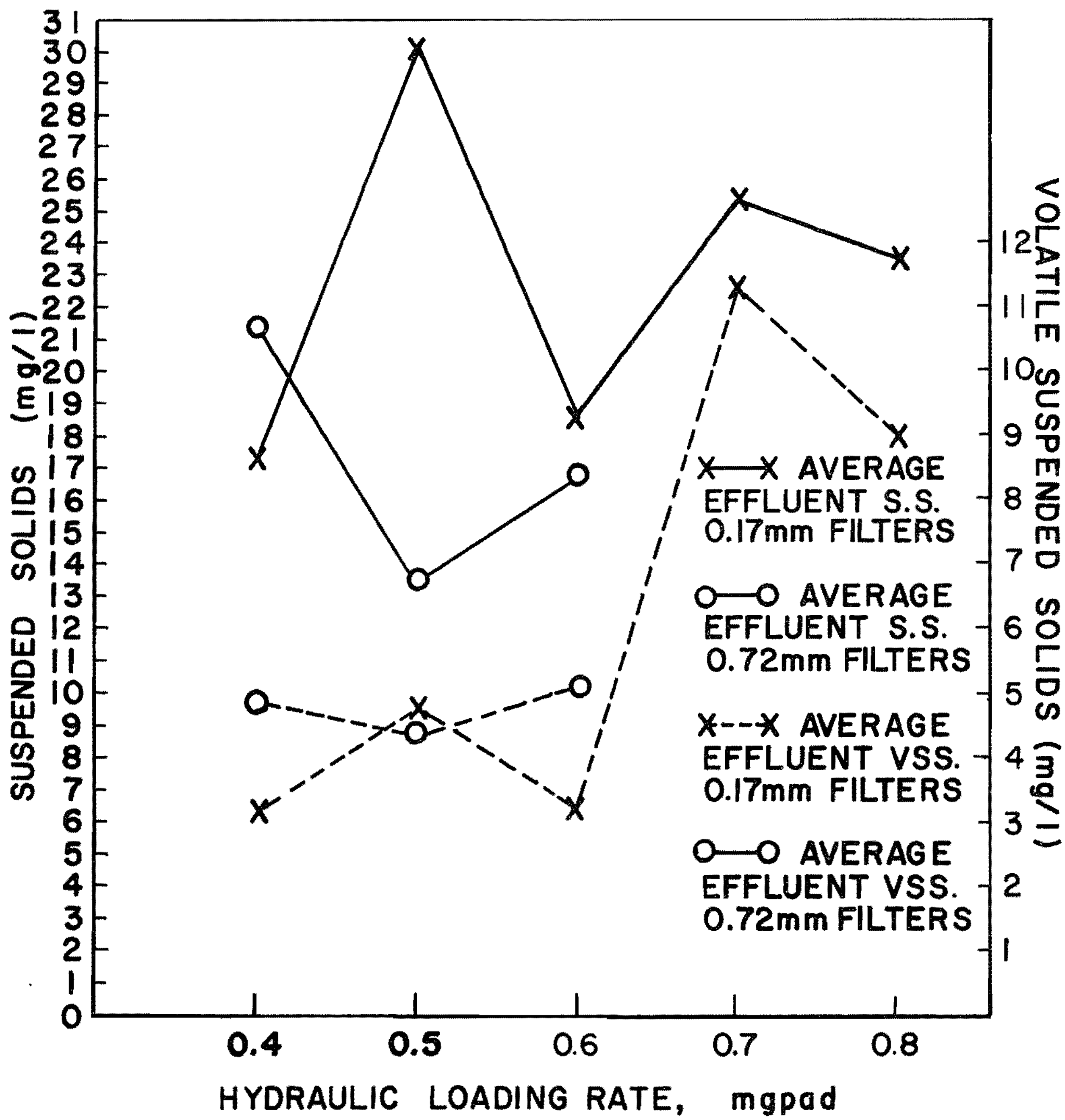

Figure 15. The relationship between the suspended and volatile suspended solids concentrations and the hydraulic loading rates for the field filters.

Table 24. **Mean influent and filter effluent algae concentrations measured as volatile suspended solids for each sand size and hydraulic loading rate studied during Phase II under field conditions.**

Month	Mean Monthly Influent Algae Conc. as Suspended Solids (mg/l)	Mean Monthly Effluent Volatile Suspended Solids Concentrations and Percent Removals															
		400,000		500,000		600,000		700,000		800,000		400,000		500,000		600,000	
		mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.	mg/l	% Red.
May	2.2	2.2	-	4.5	-	1.6	27.3	-	-	-	-	3.8	-	1.5	31.8	3.5	-
June	3.6	1.6	55.6	2.4	33.3	1.3	63.9	-	-	-	-	1.9	47.2	1.7	52.8	2.2	38.9
July	23.6	4.5	80.9	6.8	71.2	4.4	81.4	9.8	58.5	5.6	76.3	5.5	76.7	4.9	79.2	6.5	72.5
Aug	34.3	5.1	85.1	4.3	87.5	6.2	81.9	17.8	48.1	13.7	60.1	8.9	74.1	12.1	64.7	9.1	73.5
Sept	22.3	2.7	87.9	5.6	74.9	2.5	88.8	6.6	70.4	8.4	62.3	4.8	78.5	2.1	90.6	4.1	81.6

Table 25. **Mean applied and effluent total coliform counts obtained in the laboratory study.**

Applied Total Coliforms Colonies/100 ml	Effluent Total Coliforms (Colonies/100 ml) — Effective Size of Filter Media								
	0.17 mm			0.35 mm			0.72 mm		
	Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}		
	100	200	300	100	200	300	100	200	300
610,000	0	6,900	8,800	16,000	76,000	150,000	17,000	16,000	66,000

Total plate counts for bacteria are listed in Tables A-10 - A-12, Appendix A, and Table 26. These results show that the total number of bacteria in the filter influent and effluents were essentially unchanged by any of the three sand sizes studied. Also, hydraulic loading rate did not affect the numbers of bacteria present in the effluent. This result was surprising because bacterial removal efficiency was expected to approximate those obtained with slow sand filters (4). This would seem to be especially true in the case of the 0.17 mm (.0067 inch) sand as this is a sand frequently used in slow sand filters. But, even this sand shows essentially no reduction in bacteria present in the effluent samples. However, portions of the large populations of bacteria growing in the filters could easily be washed from the bed at each dosing.

Field filters

In an attempt to interpret the bacterial removal results obtained with the laboratory filters, total bacterial counts were performed on influent and effluent samples collected from the 0.17 mm and 0.72 mm effective size sands with both loaded at 500,000 gpad (765.90 m^3/hectare-day). Results summarized in Table 27 show that after the 0.17 mm filters plugged and were cleaned, three days after operation was resumed total bacterial counts were reduced by 99 percent. But after 18 days of consecutive loading, the same filter effluent contained higher concentrations of bacteria than found in the influent. This increase in effluent concentration with time of operation after cleaning is probably attributable to two factors: 1) the bacterial population in the filter dies off during the drying period before removing the top few inches of sand, and 2) when operation is resumed, the clean sand serves as an efficient filter but as more and more bacteria penetrate the bed and multiply, more are washed into the effluent.

The intermittent sand filter is as much biological as physical process and is capable of producing large populations of bacteria within the filter bed. Treatment provided by the intermittent filter when used as a polishing device

Table 26. Mean applied and effluent total bacteria counts obtained in the laboratory study.

Applied Total Bacteria Colonies/ml	Effluent Total Bacteria (Colonies/ml) Effective Size of Filter Media								
	0.17 mm			0.35 mm			0.72 mm		
	Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}			Hydraulic Loading Rates, gpad x 10^{-3}		
	100	200	300	100	200	300	100	200	300
1,100,000	99,000	1,100,000	1,200,000	1,100,000	910,000	1,200,000	1,200,000	1,000,000	1,200,000

is accomplished throughout the entire depth of the filter and not limited to the top 12 inches of the bed as implied in other studies.

Effluent Algal Bioassays

As mentioned earlier, microscopic examination indicated that algae were passing through the filters. In an attempt to quantify the degree of passage, flask bioassays were employed to assess the number of algae in the effluents.

Since it has been found that algae removal is unaffected by the hydraulic loading rates employed in this study, individual measurements at each loading rate were averaged to show the relationship between sand size and growth response (Figures 16 - 18). Algae growth, measured by an increase in light absorbancy, showed a much greater response in the effluents obtained from the filters when receiving the highest algae concentrations (Figure 18). Microscopic examination also showed higher concentrations of algae in the effluents when the highest concentration of algae was applied.

All of the flask assays exhibited a lag period of approximately three days before any significant growth occurred (Figures 16 - 18). This lag or acclimation period required for the algae to respond to a new environment could be advantageous in that it would allow the effluent to be transported considerable distances before an effect could develop. This would allow much of the algae that had passed the filter to settle out or be scavenged before growth could develop. If in the future it becomes neccessary to meet more stringent requirements than presently established in Utah, disinfection would eliminate practically any surviving algal cells.

Microscopic examinations of the field filter effluents yielded similar results but flask bioassays were not performed on the field filter effluents.

Filter Conditions at Plugging[2]

Laboratory filters

Plugging did not occur during the three original loading periods used in the laboratory study. To obtain an estimate of the time required for plugging to occur, dosing was continued after Loading Period III without removing any sand from the beds and using algae suspensions from the model stabilization ponds. Algae concentrations were the highest during Loading Period III. In order to estimate the plugging time under the most severe conditions evaluated, it was decided to continue loading at the Loading Period III concentrations.

A comparison of the effluent BOD_5 values at the time of plugging with those observed during normal operation showed no noticeable differences. Table 28 shows that at even the time of plugging, only sand size had a significant effect on the effluent BOD_5 value.

Table 28 also shows the effluent suspended solids concentrations at the time of plugging, and all the values are almost equal and near zero. This indicates that breakthrough does not occur in an intermittent sand filter. This finding is in agreement with the work of Ives (26) which showed that as the specific deposit increased,

[2]See page 15 for definition.

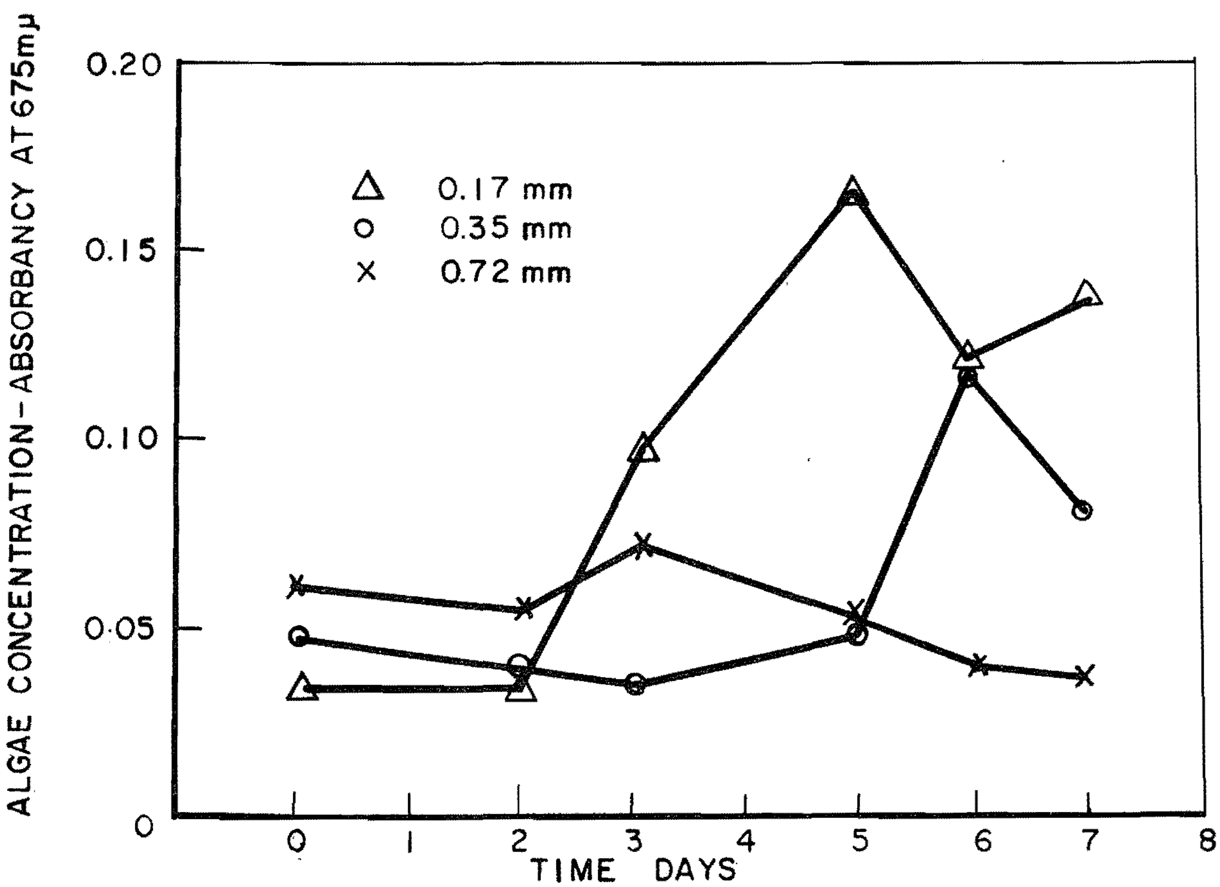

Figure 16. The mean algal growth response in filter effluents for the three sand sizes during Loading Period I.

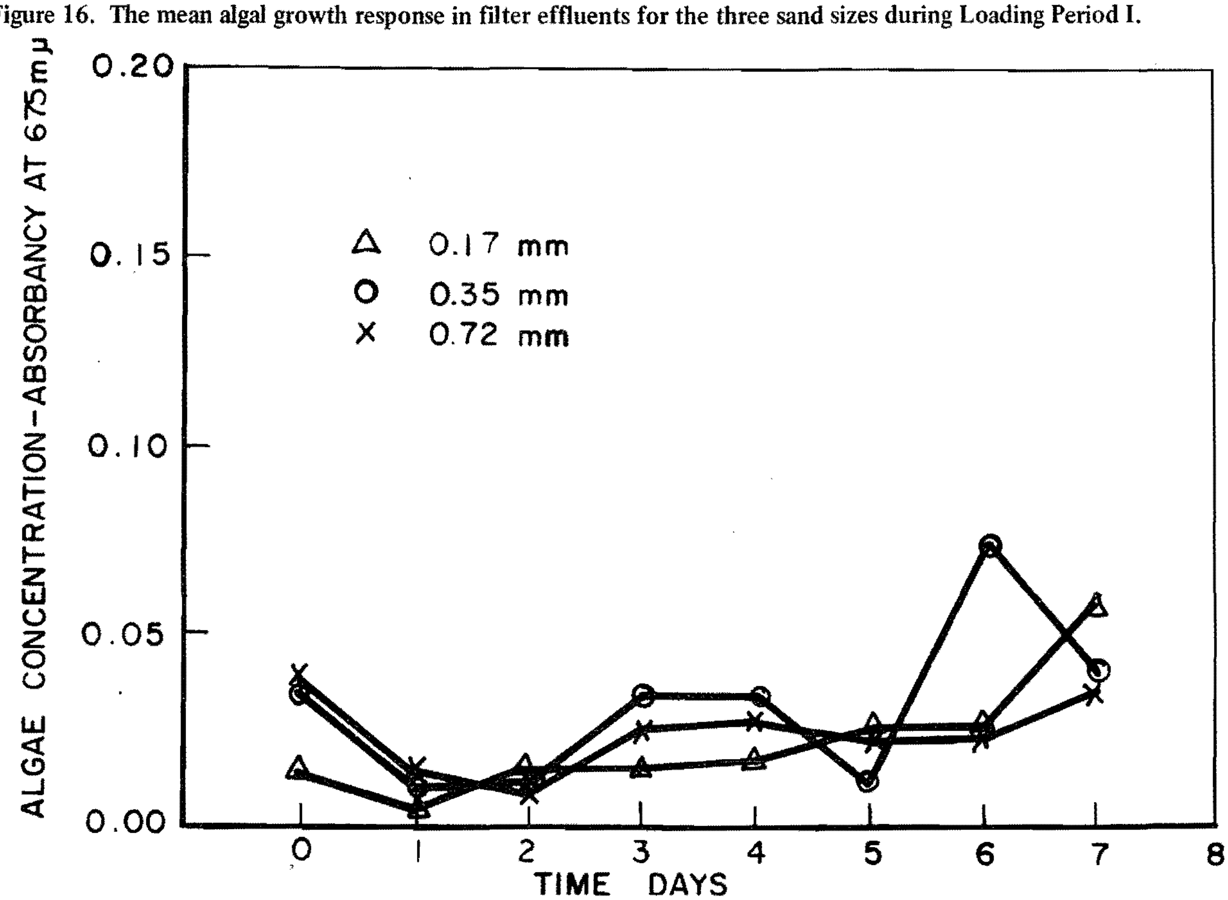

Figure 17. The mean algal growth response in the filter effluents for the three sand sizes during Loading Period II.

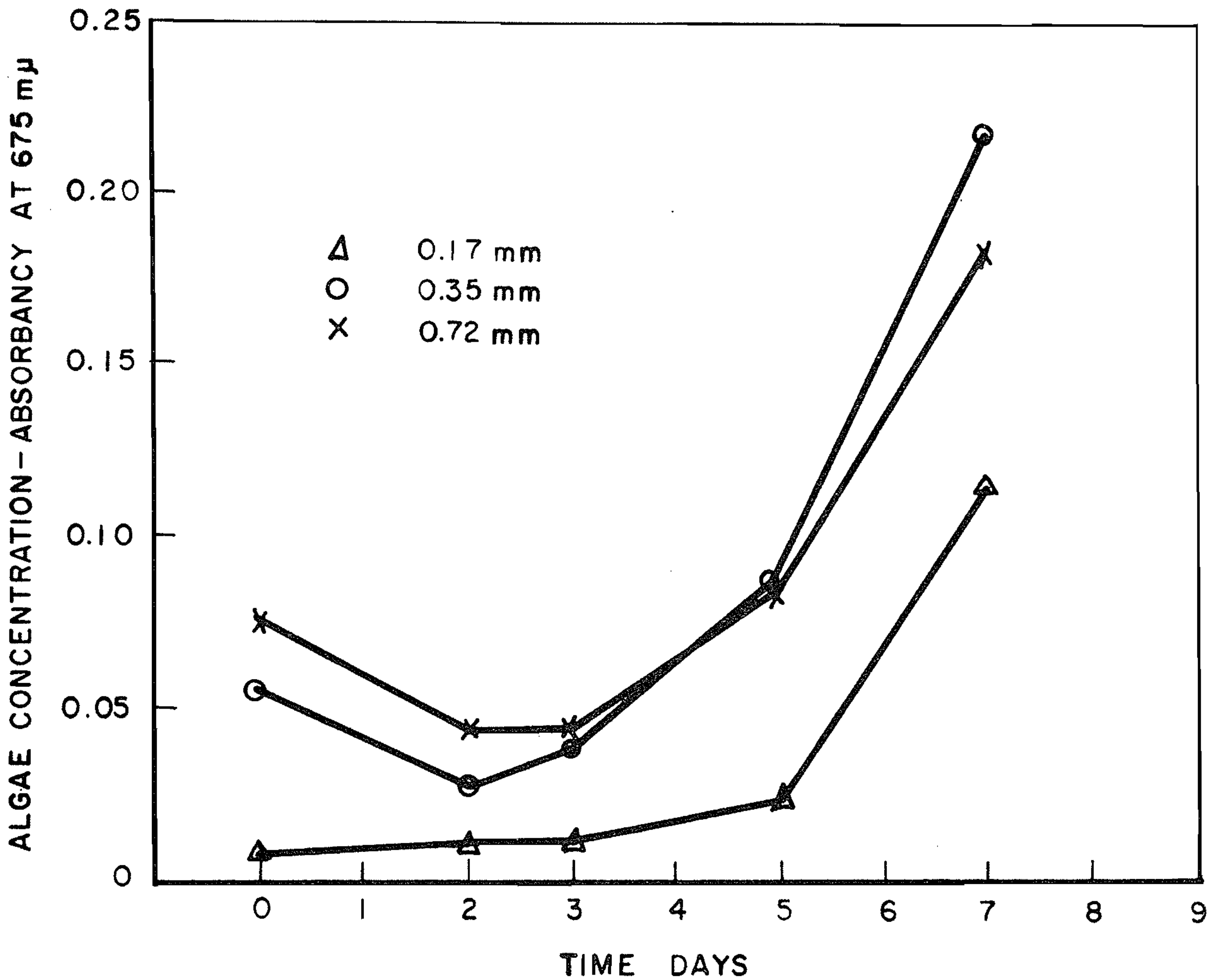

Figure 18. The mean algal growth response in the filter effluents for the three sand sizes during Loading Period III.

Possibly the most important polishing mechanisms in intermittent sand filtration is the surface mat or "schmutzdecke" which is composed of suspended matter trapped on the surface of the filter. In this study the mat was composed primarily of algae that had been deposited upon the sand surface.

Following Loading Periods I and II, the filters did not seem to have a predominant surface skin of deposited suspended matter. The top 2 inches (5 cm) of sand seemed to be cemented together by the trapped suspended matter. Below this, the sand particles, although moist, were loose and apparently unaffected by suspended matter. At no time was any of the applied suspended matter detected at depths below the top 2 - 3 inches (5 - 7.5 cm). Sand 2 - 3 inches (5 - 7.5 cm) below the surface mat examined at the end of Loading Period I and II did not appear to be affected by the applied algal suspensions. Once this sand had become dry, it was hard to tell it from new, clean sand. As the individual filters began to plug under the continued loadings following Loading Period III, a more predominant skin was noted on the sand surface. This skin was, in most cases, approximately one-sixteenth of an inch (1.6 mm) thick and covered the entire filter. During Loading Period III, the .17 mm (.0067 inch) and the .35 mm (.0137 inch) sands had surface mats that were moist, somewhat porous, and flat or well conformed to the sand surface. But, as shown in Figure 19, the surface mat for the .72 mm (.0283 inch) sand, although moist, was curled and irregular. Figure 20 shows a surface mat just before plugging occurred. It was moist, nonporous, and closely conformed to the sand surface. This change is probably caused by the longer time required for the liquid to pass through the filter. Thus, the standing water would tend to keep the mat moist and close to the sand surface which reduced the aeration and drying time between loadings. Figure 21 shows a plugged filter after it was allowed to drain and dry. It appears that if a plugged filter is allowed to dry, the surface mat will curl away from the top surface of the sand. This indicates why raking or scraping has been shown to extend the length of filter runs.

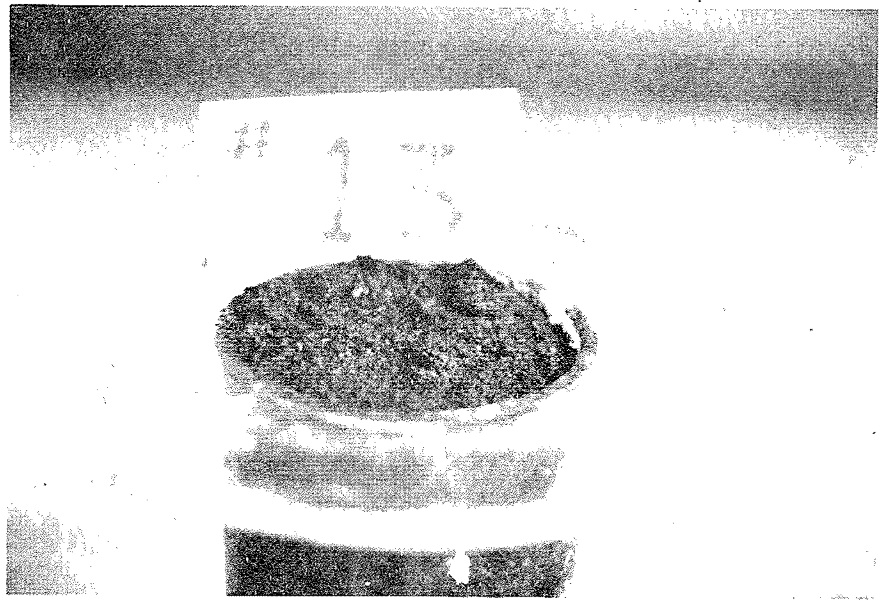

Figure 19. A .72 mm (.0233 in) size sand filter bed surface during normal operation before plugging occurred under laboratory conditions. (Continuation of Loading Period III.) Note porous and curled surface skin.

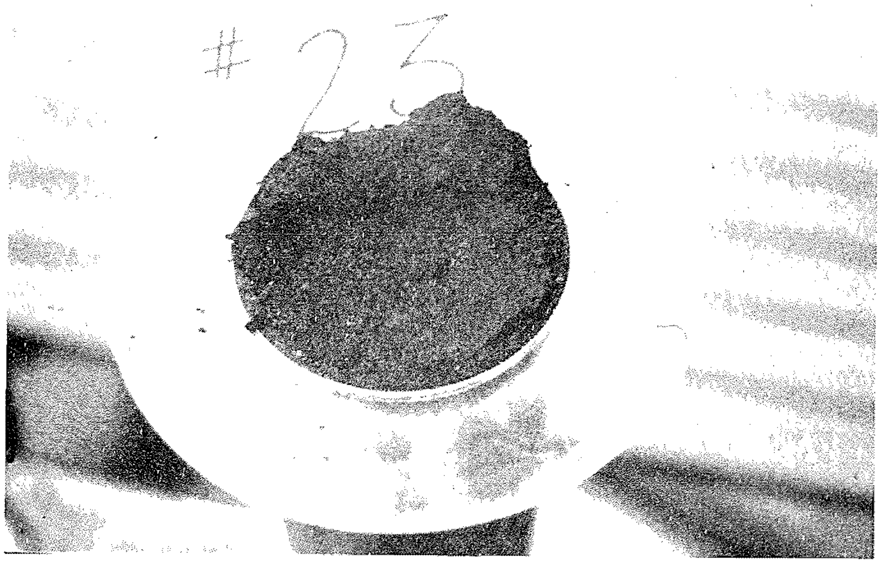

Figure 20. A .35 mm (.0137 in) size sand filter bed surface just prior to plugging under laboratory conditions. (Continuation of Loading Period III.) Note how surface skin is nonporous and well conformed to the sand bed surface.

Table 27. **Influent and effluent total plate count bacterial density for filters A-5 and C-5 loaded at 500,000 gpad.**

Date	Bacterial Density, Colonies/ml			Filter History	
	Applied	Effluent A5	Effluent C5	A5	C5
14 May 73	-	-	-	Begin Loading	
10 Aug 73	-	-	-	Plug	No Plugging All Season
21 Aug 73	-	-	-	Resume Loading	
23 Aug 73	43,000	370	280,000	After 3 Consecutive Loading Days	After 100 Consecutive Loading Days
7 Sept 73	5,600	49,000	104,000	After 18 Consecutive Loading Days	After 115 Consecutive Loading Days
12 Sept 73	6,000	61,000	99,000	After 23 Consecutive Loading Days	After 120 Consecutive Loading Days
27 Sept 73	37,000	76,000	4,100,000	After 38 Consecutive Loading Days	After 135 Consecutive Loading Days

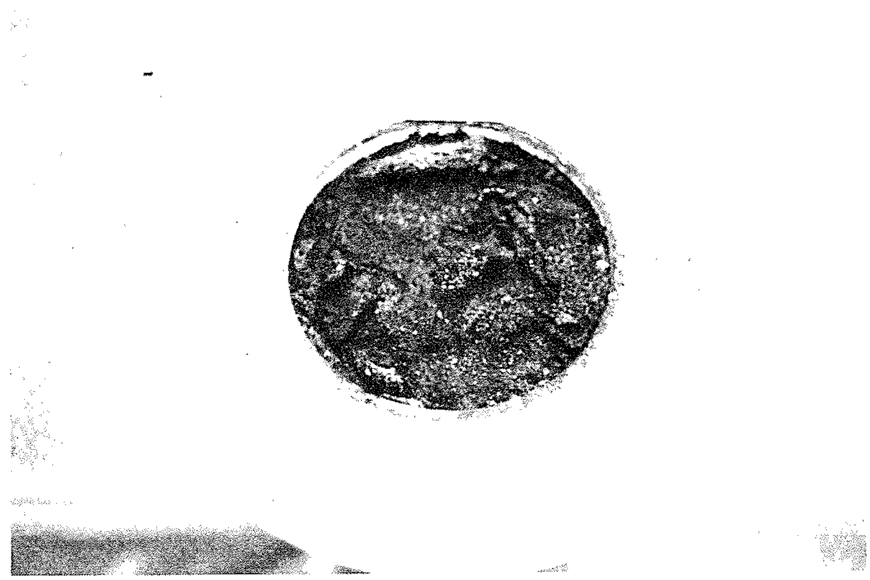

Figure 21. Typical plugged filter bed surface after it was allowed to drain and dry under laboratory conditions. (Continuation of Loading Period III.) Note how surface skin is curled and cracked.

the filter coefficient increased. Since the hydraulic head above the sand was not increased to the point that the filter coefficient was forced to decrease, the filters would plug when the filter coefficient was at a maximum. If it were practical to increase the head on intermittent sand filters, breakthrough might occur as in a high rate or pressurized filter.

Field filters

At the higher hydraulic loading rates employed in the field study the surface mats for both the 0.17 and 0.72 mm sands followed essentially the same pattern as that observed in the laboratory. The 0.17 mm field filters operated approximately the same period of time before plugging as reported for the laboratory filters (Table 29). At the loading rates employed (400,000 to 600,000 gpad) with the 0.72 mm filters, plugging did not occur during the entire study. Based upon the results of both the laboratory and field studies, it appears that much higher hydraulic loading rates can be employed with the 0.72 mm filters. Higher hydraulic loading rates may result in more efficient solids removals with the 0.72 mm filters because of an increase in thickness of the mat that would accumulate on the surface and serve to trap more of the algae and debris. More detailed economic studies of the operation of the filters needs to be completed, but it appears that the hydraulic loading rate for the 0.17 mm effective size sand filters is limited to approximately 1 mgpad.

The 0.17 mm filters were cleaned by raking only which accounts for the relatively short periods of operation between the first and second plugging. If a conventional cleaning by removing the top 2-3 inches of sand had been performed, the second period of operation would have matched the initial period. However, because raking is an inexpensive method of extending the period between sand removals, it should be considered part of the routine operating procedure.

Table 28. Effluent BOD_5 and suspended solids concentrations observed for the laboratory filters at the time of plugging.

Filter	Effluent BOD @ First Plugging	Second Plugging	Effluent Suspended Solids First Plugging	Second Plugging
11	-		3.8	
12	-		3.8	
13	-		1.9	
21	14.2	8.4	2.0	
22	13.2		0.0	
23	10.3		3.0	
31	7.5		1.9	
32	4.9		-	1.7
33	6.0		0.8	

Table 29. Operational history of the field filters during 1973.

Filter	Date Began Loading	Date 1st Plug	Ave. SS mg/l Applied	Type Cleaner	Date Loading Resumed	Date 2nd Plug	Ave. SS mg/l Applied	Date Loading Resumed	Type Cleaning	Date of 3rd Plug	Ave. SS mg/l Applied
A4	14 May	10 Aug	20.75	Rake	21 Aug	27 Sept[a]	27.57		-		
A5	14 May	10 Aug	20.75	Rake	21 Aug	27 Sept[a]	27.57		-		
A6	14 May	7 Aug	18.18	Rake	21 Aug	12 Sept	29.58	24 Sept	Rake	27 Sept[a]	24.57
A7	9 July	10 Aug	42.12	Rake	21 Aug	27 Sept[a]	27.57		-		
A8	9 July	10 Aug	42.12	Rake	15 Aug	7 Sept	34.99	7 Sept	Rake	27 Sept[a]	23.82
C4	14 May	27 Sept[a]	25.10								
C5	14 May	27 Sept[a]	25.10								
C6	14 May	27 Sept[a]	25.10								

[a]Project ends.

Time of Operation

Laboratory filters

Figure 22 shows the effect of sand size and run time before plugging occurs. It is again evident that the finer sands produce the lowest effluent suspended solids concentrations. But it is also quite apparent that this improved effectiveness was attained at the expense of a reduction in operation time before plugging.

As the operating time increased, an increase in the suspended solids removal efficiency was noted. This is the same as noted by Ives (28), i.e., as the specific deposit increases, the filter coefficient increases. Knowledge of this situation could prove to be valuable when operating a number of filters. Regular analysis of the effluent for suspended solids would allow one to predict when plugging was likely to occur.

Continued operation eventually caused plugging in all filters. The results show that the .72 mm (.0283 inch) filter operated 175 consecutive days before plugging when loaded at a mean algae concentration of 51 mg/l, the 0.17 mm (.0067 inch) sand operated 68 consecutive days at a mean algae concentration of 43 mg/l, and the 0.35 mm (.0137 inch) sand operated 99 consecutive days at applied algae concentration of 45 mg/l.

Table 30 shows in more detail the operational results for all the filters during the continuation of Loading Period III. Removing the top 4 inches (10 cm) of sand from the filters after plugging, replacing it with new sand, and putting the unit back in operation gives second performance periods generally less than the original period. Longer operating periods were expected with the lower hydraulic loading rates; however, the 0.17 mm (.0067 inch) and 0.35 mm (.0137 inch) effective size filters at the lowest loading rate were the first to plug. This anomaly was probably caused by the small hydraulic head available to force the liquid through these sands. Apparently this was the reason that it took less time for the highest loading rate volume to pass through the sands.

Figure 23 shows that the highest hydraulic loading rate studied also allowed greater volumes of applied water to pass the filter bed before plugging occurred. As the figure shows, the result was the same for all the sand sizes studied. Figure 24 shows the observed effect of hydraulic loading rate on the number of days each filter operated until plugging occurred. The effect of the additional hydraulic head afforded by the highest hydraulic loading rate studied becomes evident from this figure.

Field filters

As reported for the laboratory filters, the finer sand produced a superior effluent in all categories measured and again this higher efficiency was attained at the expense of a reduction in operation time before plugging (Tables 29 and A-13 through A-18).

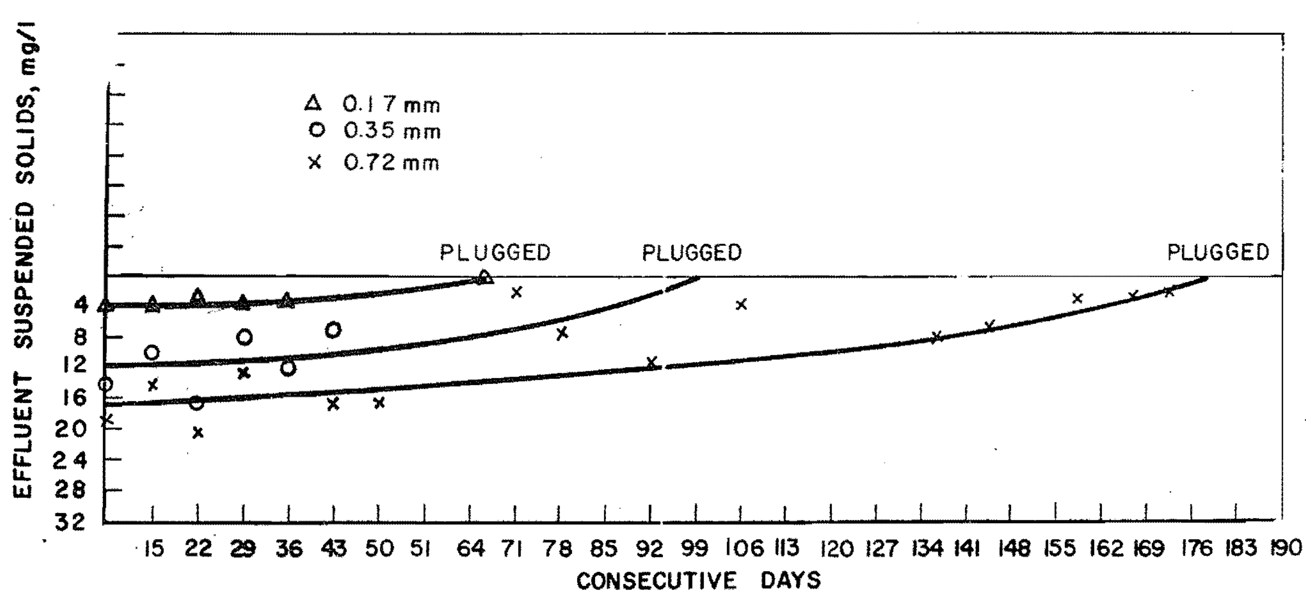

Figure 22. Observed times of operation under approximately 45 mg/l applied algae afforded by each sand size under laboratory conditions and the resulting effect on effluent suspended solids concentration. (Loading Period III plus continuation.)

Table 30. Results of the continuation of Loading Period III showing approximate period of operation by the laboratory filters using two cleaning methods. Loading Period III began 11/1/72 at which time all filters had the top 4 inches (10 cm) of sand removed and replaced with new sand.

Filter	Date First plugging	Mean applied SS to date	Type cleaning	Date put back in use	Date second plugging	Mean applied SS from first plugging	Type cleaning	Date put back in use	Date third plugging
SF 11	4/14	51.46	raking	4/20/73					
SF 12	4/27	51.08	-	-					
SF 13	4/25	51.08	-	-					
SF 21	12/26/72	44.57	scraping	1/10/73	2/6/73	51.76	scraping	2/8/73	
SF 22	12/27/72	44.57	scraping	1/10/73	4/2/73		raking		
SF 23	3/5/73	48.79	raking	3/8/73					
SF 31	12/18/72	46.35	scraping	1/10/73	1/20/73	53.59	scraping	1/30/73	
SF 32	1/28/73	45.19	scraping	1/30/73					
SF 33	1/17/73	44.47	scraping	1/30/73	3/14/73	63.56	raking	3/21/73	4/3/73

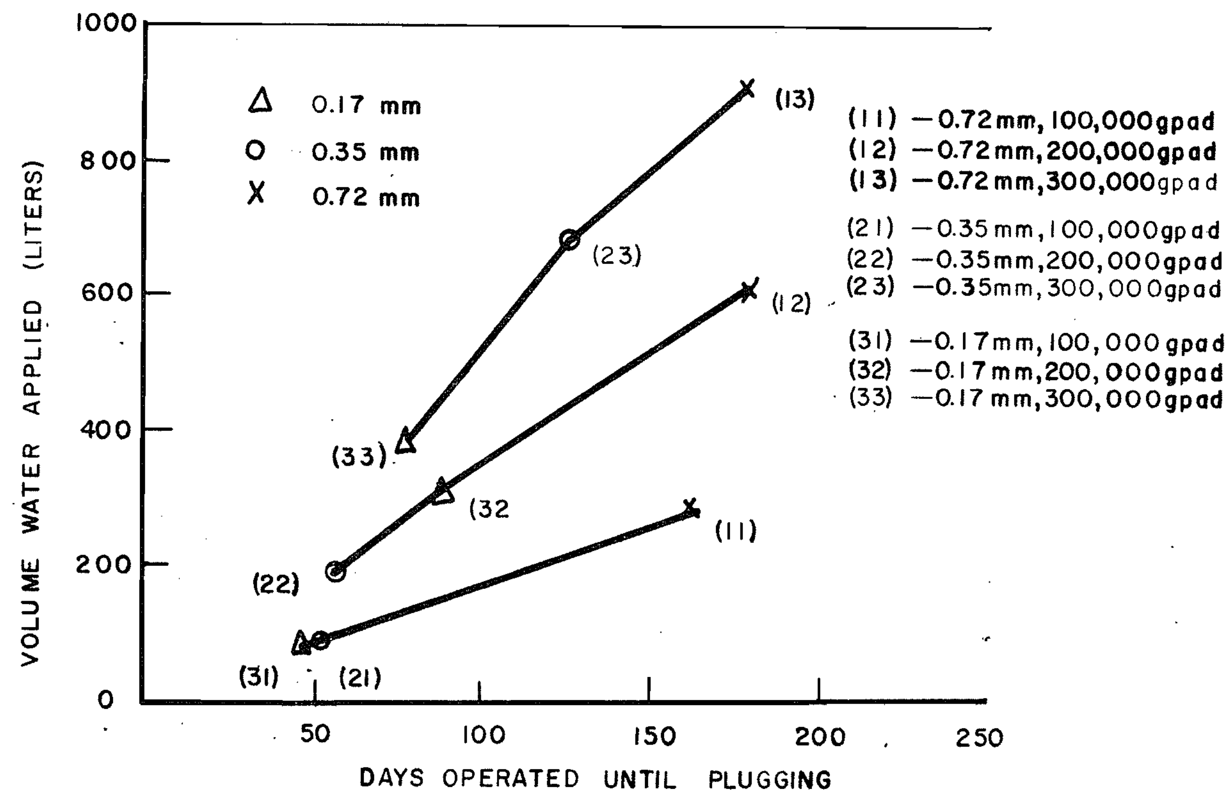

Figure 23. The relationship observed between the volume of water applied until plugging occurred under laboratory conditions.

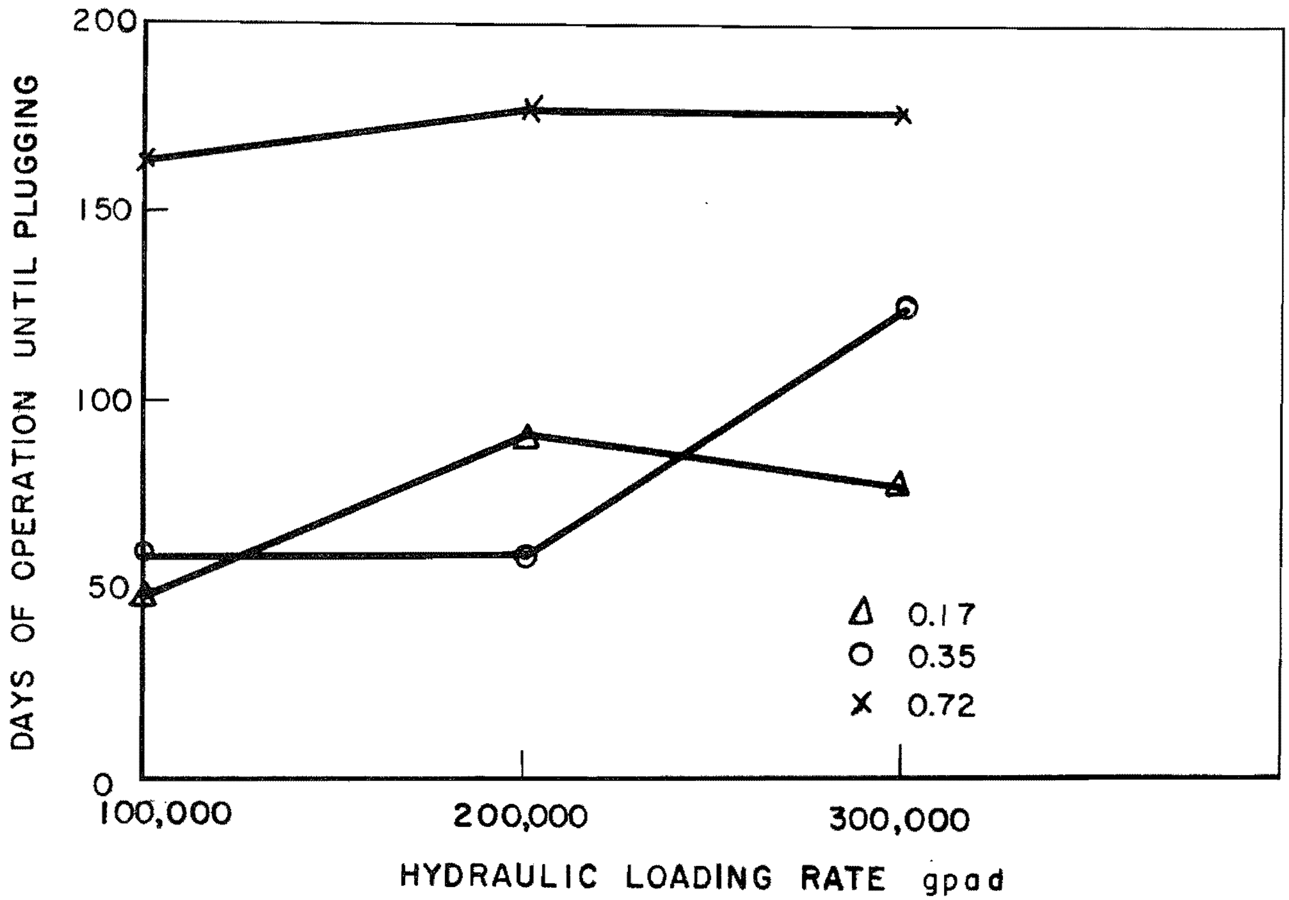

Figure 24. The relationship observed between hydraulic loading rate and the days of operation until plugging occurs under laboratory conditions.

The increase in suspended solids removal efficiency with increasing operating time was observed for the field filters, but the effluent concentrations appeared to reach a limit and did not continue to drop until plugging occurred. The removal efficiency increase with time of operation in the field filters appeared to be more closely associated with the washing of fines from the filters. However, there is no reason not to expect similar performances between the laboratory and field filters, and it may be that the lack of a decrease in effluent solids concentration is attributable to a continuous washing of fines from the filters up to plugging. After more than one summer of operation, it is likely that a pattern as observed in the laboratory would evolve in the field. Since the 0.72 mm field filters did not plug during the summer of operation, it is possible that the laboratory study results would have been duplicated had the project continued, or had the suspended solids concentrations in the influent been increased.

The 0.72 mm filters operated 137 consecutive days without plugging when loaded at 0.4, 0.5, and 0.6 mgpad with a lagoon effluent containing an average suspended solids concentration of 25 mg/l. It was not surprising that these units did not plug, because laboratory units with the same sand and a hydraulic loading rate of 0.3 mgpad operated 175 days before plugging and were dosed with a lagoon effluent containing an average suspended solids concentration of 51 mg/l.

The consecutive days of operation for the 0.17 mm filters appear to be directly related to the hydraulic loading rates. Figure 25 shows that up to a loading rate of 0.6 mgpad the filters operated approximately 100 days before plugging when receiving a lagoon effluent containing a mean algae concentration of 20 mg/l. During these 100 days, the filter influent algae concentration ranged from 4 to 51 mg/l. At loading rates of 0.7 and 0.8 mgpad, the 0.17 mm filters operated only 32 consecutive days when receiving a lagoon effluent containing a mean suspended solids concentration of 42 mg/l, and a range of concentrations varying between 30 and 50 mg/l. Because of the large difference in the mean applied suspended solids concentrations, it is impossible to compare the performances at the two hydraulic loading rates, or to develop relationships between consecutive days of operation and the hydraulic loading rate.

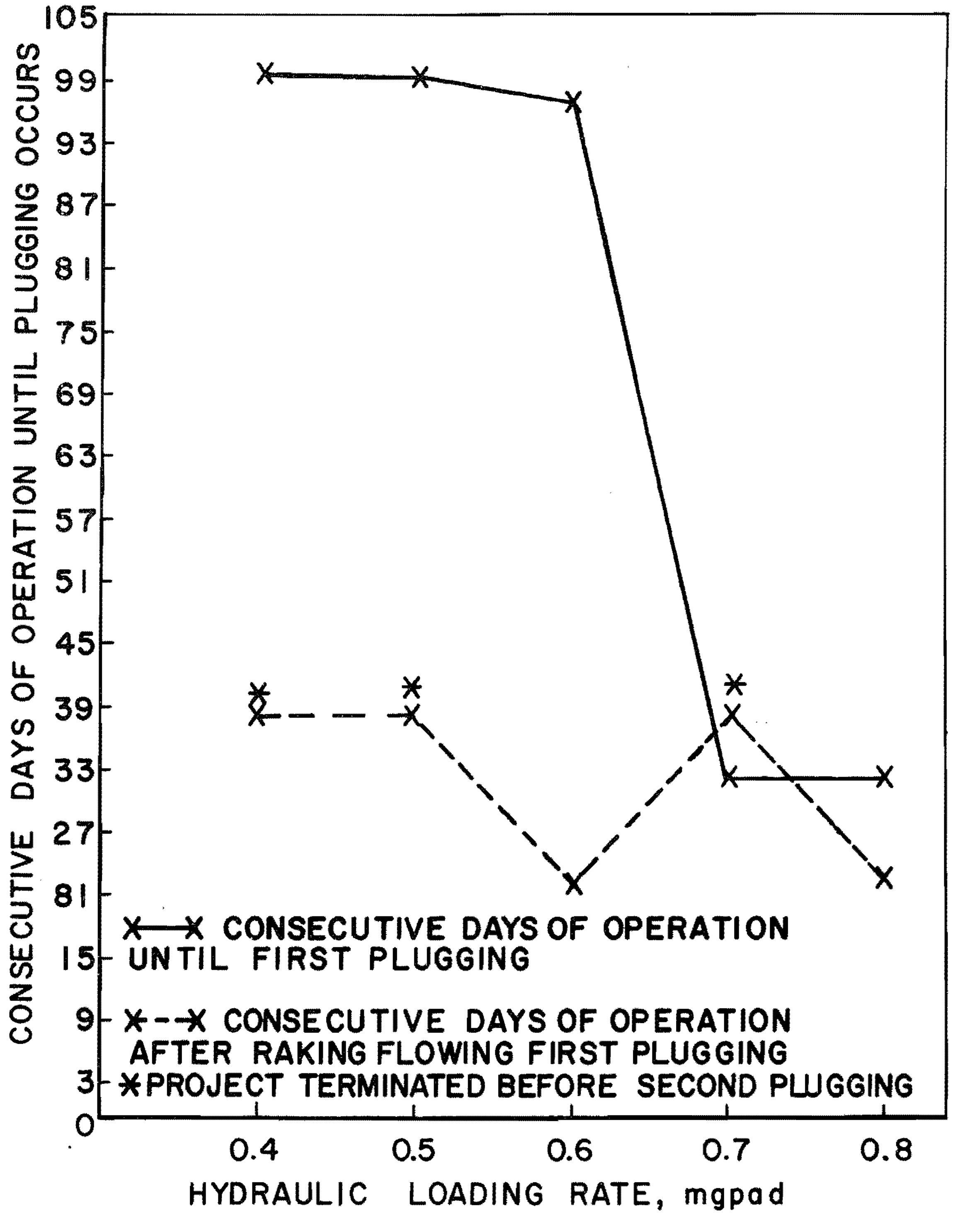

Figure 25. Consecutive days of operation until plugging occurred in the 0.17 mm effective size sand filters at various hydraulic loading rates.

However, the results are useful in estimating the number of times during an algae growing season that the filters must be raked and cleaned. During the early spring and summer it is likely that the units will perform effectively for the first 3 months, and removing the top 2 to 4 inches of sand the units should perform a minimum of one month even at very high algae concentrations in the filter influent. It is possible that the consecutive days of operation at the 0.7 and 0.8 mgpad hydraulic loading rates will match those at the 0.4 to 0.6 mgpad rates when receiving equal concentrations of influent algae. Length of operation and the economics of maintenance will be answered in the continuation of the project which will be conducted on a prototype scale.

When the filters plugged, the surface mat and approximately the top 2 inches of sand were raked and broken up and then placed in service again. Figure 25 shows that there was not a relationship between hydraulic loading rate and consecutive days of operation following the raking. The 0.17 mm filters receiving 0.4, 0.5, and 0.7 mgpad of lagoon effluent had loaded for 38 days and were still operating after the first raking when the project was terminated. The filters receiving 0.6 and 0.8 mgpad plugged within 22 days after the raking. Mean suspended solids concentrations in the lagoon effluent applied to all of the 0.17 mm filters following raking were approximately equal, but the two filters that plugged the second time did receive the highest concentrations of algae, 29.6 mg/l for the 0.6 mgpad loading rate and 35.0 mg/l for the 0.8 mgpad loading rate.

Although direct comparisons of the lengths of performance at the various hydraulic loading rates are difficult, it is obvious that the length of runs for all of the sands and hydraulic loading rates are of adequate length to make intermittent sand filtration competitive with all other processes available to upgrade lagoon effluents to meet new water quality standards.

Figure 26 shows the volume of lagoon effluent applied to the filters during the 137 days of operation. As reported for the laboratory filters, the greatest volume of water was treated in a given time span by the filters receiving the highest hydraulic loading rates even when plugging occurred and it was necessary to rest the filter and rake the surface before returning it to operation.

Overall Evaluation of the Process

Ability to meet present state standards

Intermittent sand filtration was evaluated to assess its capability to produce an effluent that would meet the Utah Class "C" stream standards shown in Table 31 when imposed as discharge standards. In a system such as the Logan City Wastewater Stabilization Ponds, the intermittent sand filter would produce an effluent meeting Class "C" discharge standards 99 percent of the time. The 0.17 mm (.0067 inch) effective size laboratory filters only produced an effluent with a mean BOD_5 greater than 5 mg/l (maximum effluent BOD_5 equal 8 mg/l) when loaded at the highest algae concentration and with an influent BOD_5 concentration averaging 36 mg/l. The BOD_5 concentration (36 mg/l) in the influent during Loading Period III was much higher than normally obtained from a well operated secondary wastewater treatment plant. The 0.17 mm field filters produced an effluent with a BOD_5 concentration of less than 5 mg/l on all days of operation when loaded at 0.6 mgpad or less. Effluent BOD_5 concentrations for the filter loaded at 0.7 mgpad exceeded 5 mg/l on only two days out of 69 days of operation and the maximum value in the effluent was 6.7 mg/l. The average BOD_5 in filter A7 effluent was 3.7 mg/l for the entire period of 69 days. A loading of 0.8 mgpad produced an effluent of slightly poorer quality but still reduced the BOD_5 to a mean value of 4.1 mg/l. Thus, most properly operated secondary treatment plants in the state would be able to meet Class "C" discharge standards with the addition of intermittent sand filtration. Even under such heavy BOD_5 loadings as studied during Loading Period III in the laboratory, reductions were greater than 80 percent for the 0.17 mm (.0067 inch) sand at all hydraulic loading rates.

Middlebrooks et al. (33) reported the effluent characteristics of 11 existing wastewater treatment plants in the State of Utah, some of which were heavily

Table 31. Class "C" stream standards for the State of Utah (33).

Parameter	Concentration or Unit
pH	6.5 - 8.5
Total Coliform, Monthly Arithmetic Mean	5,000/100 ml
Fecal Coliform, Monthly Arithmetic Mean	2,000/100 ml
BOD , Monthly Arithmetic Mean	5 mg/l
Dissolved Oxygen	> 5.5 mg/l
Chemical and Radiological	PHS Drinking Water Standards

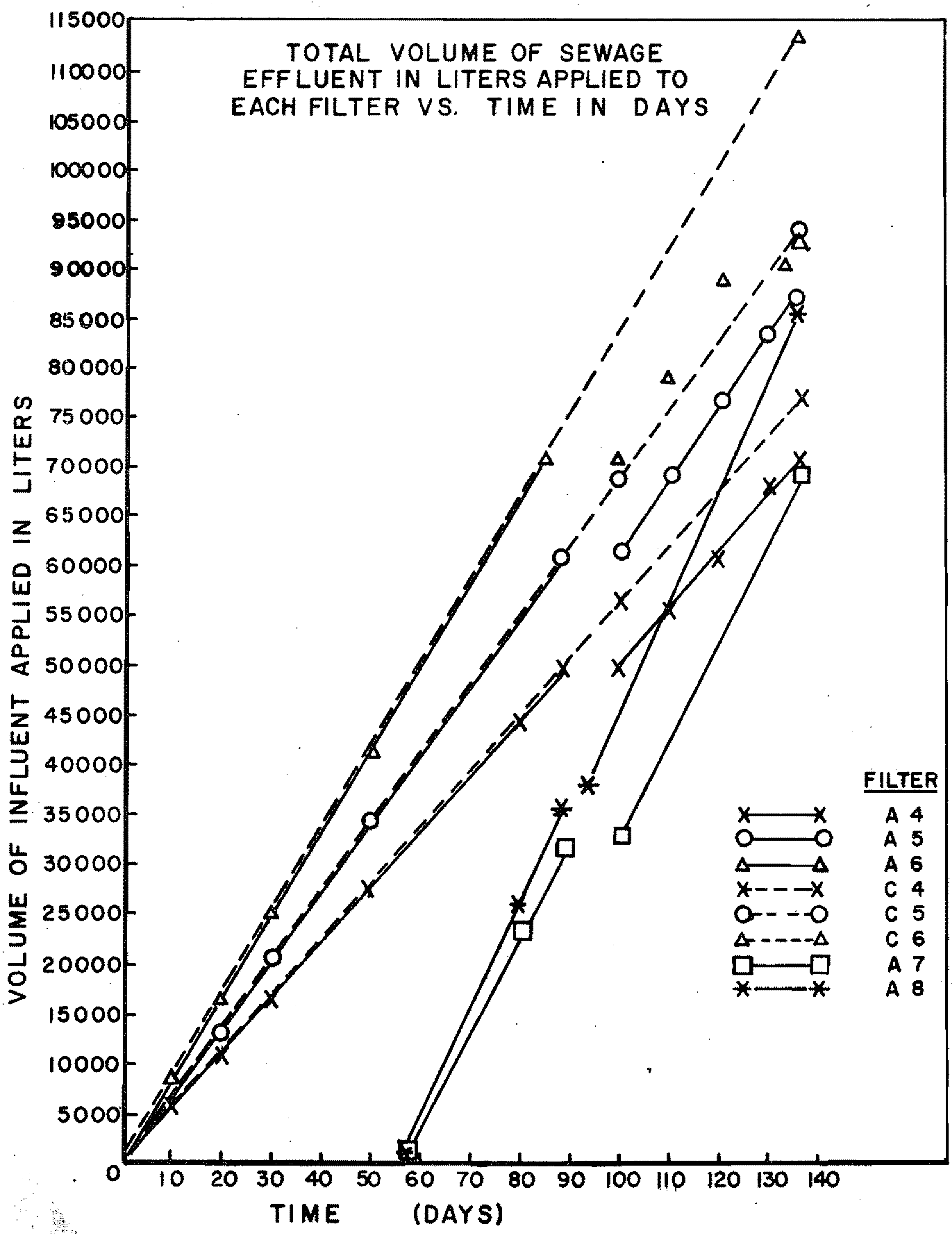

Figure 26. The relationship observed between the volume of water applied until plugging occurred under field conditions. Discontinuity in the lines represents the rest and cleaning period following plugging.

overloaded. Seven were trickling filters and five were wastewater stabilization ponds. Assuming that equivalent reductions in BOD_5, suspended solids, and coliform organisms would be obtained by intermittent sand filtration on all types of secondary treatment plant effluents, seven of the eleven plants would be able to meet the BOD_5 standards for Class "C" discharged waters by adding intermittent sand filters. If the overloading were corrected and the plants operated properly, all 11 plants could meet Class "C" discharge standards by installing intermittent filters. Several of these plants were serving metropolitan areas, and it may not be feasible to utilize intermittent filters because of land limitations and economic constraints usually associated with metropolitan areas.

On the basis of the mean total coliforms per 100 ml reported, five of the eleven Utah wastewater treatment facilities would be able to meet Class "C" discharge standards by the addition of intermittent sand filtration. If the plants were not overloaded, in all probability the coliform requirements could be met in all 11 plants. Again, the addition of a disinfection step would aid materially in meeting coliform removal requirements as well as eliminate the contributions to a downstream algae bloom problem.

Class "C" discharge requirements for pH value and dissolved oxygen are normally easily met by secondary treatment, and the intermittent sand filtration of these effluents further refines effluents. The pH values of the Logan lagoon effluents were approximately equal to a value of 9. When passed through the filters, the pH was reduced to values approximately within the limits imposed. Six to nine mg/l of dissolved oxygen were readily produced by intermittent sand filtration which also meets Class "C" water standards.

Cost Estimate

A general approach was taken in the preparation of the cost estimates shown in Appendix D for an effluent polishing intermittent sand filter process. The estimates shown for initial plant construction outlays are of a higher degree of reliability than the values estimated for operation. This is attributed to the nature of this study and the information gained during this initial phase. Much better estimates of operational expenses will be afforded by the future prototype study under actual field conditions.

The in place total construction cost estimates were prepared through the aid of a local consulting engineering firm. Thus, they are representative of the outlay necessary to construct a typical intermittent sand filter process in the intermountain area during 1973.

The construction and annual operation cost estimate shown for the 15 mgd Logan City facility is not as general in nature as the other estimates. This estimate was prepared on two assumptions. One, the process would be located such that pumping of the applied effluent was not necessary. Two, additional cost for land is not necessary as the final one and a half existing tertiary ponds would be drained and the polishing filters would be located within these boundaries. Also, the 15 mgd Logan stabilization pond system is presently the largest existing facility of this type in Utah. A cost estimate for this facility will then provide an expense evaluation for the entire range of stabilization pond systems in Utah.

A large difference was found between locally available filtering media and specially prepared media, so an economic evaluation of the two types of media was made. In this case, the .17 mm (.0067 inch) size media was locally available and the .35 mm (.0137 inch) and the .72 mm (.0283 inch) sizes were specially prepared. The specially prepared media in this area was found to be more than five times more costly than the locally available media. Based on the assumptions that the .17 mm (.0067 inch) locally available media was approximately two and one-half times more costly to operate than the .72 mm (.0283 inch) media, the .17 mm (.0067 inch) media was found to be the economic choice for a 1 mgd and the 15 mgd existing facility.

Construction cost estimates are shown in Appendix D for an effluent polishing intermittent sand filter process. The construction costs determined in Estimates 1 through 4, Appendix D, reflect a paired bed operation designed at 300,000 gpad (459.5 m^3/hectare-day) and 800,000 gpad (1225.4 m^3/hectare-day) and the application of the effluent to the filter in less than 90 minutes. It was assumed that in a municipal construction effort such as this, at least 75 percent of the construction cost would be funded by federal aid. Also, costs without federal assistance are reported in Appendix D. Based on these items, a filter process designed at 300,000 gpad (459.5 m^3/hectare-day) to treat 1 mgd of wastewater effluent when constructed with local media will cost the community a total of $96,200 to construct (Estimate 1). Although a hydraulic loading rate of 800,000 gpad (1225.4 m^3/hectare-day) was not studied under laboratory conditions, preliminary results of a study under field conditions show this rate to be feasible for design. Thus, designed at 800,000 gpad (1225.4 m^3/hectare-day), the same filter process will cost the community $38,700 to construct (Estimate 2). The same filter process designed at 800,000 gpad (1225.4 m^3/hectare-day) and constructed with a specially prepared media would cost the community $138,000 to construct (Estimate 3). The increased cost of the media is apparent in this cost estimate. Even when constructed of a locally available media and aided by federal funds, a filter process designed specifically for the 15 mgd Logan facility will cost the community $674,000 to construct (Estimate 4).

The construction costs for Estimate 5 reflect an optimum design situation for a 1 mgd facility. Conditions considered optimum are minimum bed area operated under scheduled rotation, no pumping required for dosing, locally available media, and plastic bed liners not required. Under these conditions with the aid of federal funds, a filter process designed at 800,000 gpad (1225.4 m^3/hectare-day) for a 1 mgd facility would cost the community $14,500 to construct.

From the itemized values listed for Estimates 1 - 4, Appendix D, sand or media expense is approximately 25 percent of the total construction cost. Also, the plastic liner for the bed is approximately 25 percent of the total construction cost. Whether or not the liners are a required expense in constructing effluent polishing intermittent sand filters will depend on the specific conditions and regulations governing each location and installation of this process. As shown in Estimate 5, considerable savings are made by not installing the plastic bed liners. In rural areas, land costs for this process are less than 5 percent of the total construction costs.

Based on rough estimates, operational costs, construction cost per acre of filter bed, and the final product costs are also shown in detail in Appendix D. As noted in the review of the literature, 1972 construction costs per acre and per million gallons of filtrate produced were reported for slow sand and intermittent sand filters. These estimates are summarized in Table 32.

Table 32. Undated costs for slow sand and intermittent sand filters for raw sewage (3,27,47).

	Construction cost per acre	Cost per million gallons
Slow sand filters	$252,000	$29-$99
Intermittent sand filters (raw & primary sewage)	$62,000-$87,000	$139

From Estimates 1, 2, and 4, Appendix D, construction costs for an intermittent sand filter of $58,000 per acre could be expected using a locally available media under general conditions. This value is less than the costs noted in Table 32 but is still in reasonable agreement with the updated values. The $220,000 per acre construction cost for the specially produced media (Estimate 3) again reflects the difference in cost between the sources of media. Also, the $25,800 per acre construction cost reflects the advantage of this process under optimum conditions.

In order that this process can be compared to other means of polishing wastewater effluents, the cost per million gallons of filtrate produced is shown in detail in Estimates 1-5, Appendix D. These costs are summarized in Table 33.

From Table 33, the cost per million gallons produced by this filter process is in agreement with the updated costs (Table 32) for slow sand filters and considerably less than those reported for intermittent sand filtration of sewage.

Costs per million gallons of effluent produced are shown in Table 33 with and without federal assistance. Without federal funds, the costs are greatly increased. The effect of an optimum condition application is noted by the cost of $16 per million gallons (Table 33). Combined effects of larger scale operation and specific application, which in this case held conditions near optimum, are noted by the $15 per million gallons cost for the Logan City facility. Finally, for the general applications estimates when a 1 mgd plant constructed with .17 mm (.0067 inch) effective size locally available media is compared to one constructed of a specially prepared media. The cost of operation and media using the .17 mm (.0067 inch) effective size sand designed for a hydraulic loading rate of .3 mgd is essentially equal to the operation

Table 33. Estimated cost per million gallons of filtrate produced by various designs of an effluent polishing intermittent sand filter process.

Application conditions	Existing facility flow rate	Design hydraulic loading rate	Effective sand size	Cost with federal assistance $/10 gallons	Cost without federal assistance $/10 gallons
General (Estimate 1)	1 mgd	0.3 mgad	.17 mm	$47	$115
General (Estimate 2)	1 mgd	0.8 mgad	.17 mm	$33	$ 61
General (Estimate 3)	1 mgd	0.8 mgad	.72 mm	$46	$145
Specific (Estimate 4)	15 mgd	0.6 mgad	.17 mm	$15	$ 48
Optimum (Estimate 5)	1 mgd	0.8 mgad	.17 mm	$16	$ 26

and media costs for the coarser .72 mm (.0283 inch) effective size specially produced sand. If the .72 mm (.0283 inch) effective size specially prepared sand filter were designed using much higher loading rates and optimum conditions, the cost per million gallons for this particular sand would decrease to the point where it would become economically competitive.

From the present understanding of the operation of effluent polishing intermittent sand filters, a cost ranging between $15 to $47 per million gallons can be assumed to be representative of this process. Table 34 lists alternative methods to meet Class "C" water standards and their estimated costs as reported by Middlebrooks et al. (33). Based on these values, the earlier stated cost for an effluent polishing intermittent sand filter process is quite competitive. There are many avenues of approach that may be taken to produce the same high quality effluent of this process at even lower expense. Coupling this possibility with the fact that a majority of the existing wastewater effluents in Utah can be upgraded to meet Class "C" water standards by the addition of this process, intermittent sand filtration of wastewater effluents has been found to be an economically feasible method of wastewater effluent polishing.

Table 34. Cost of alternative methods of polishing wastewater effluents (32).

Method	Cost per 10^6 gallons
Chemical treatment (solids contact)	$60-130
Granular or mixed media filtration w/chem	$50
Dissolved air flotation	$110
Electrodialysis	$200
Microstraining	$18

SUMMARY AND CONCLUSIONS

The major objective of this study was to evaluate the performance of the intermittent sand filter and to determine if it was capable of upgrading existing wastewater treatment plants in the State of Utah to meet Class "C" water quality standards. This study was conducted under laboratory and field conditions.

A literature review indicated that the use of intermittent sand filtration had not been evaluated as a means of upgrading secondary wastewater treatment plant effluents. Therefore, the literature review summarizes information about related areas, i.e., slow sand filtration, intermittent sand filtration of raw sewage, algae removal by granular media, and secondary effluent characterization. This information was then used to design the experiment and speculate as to how intermittent sand filtration should perform as a polishing unit. Actual wastewater stabilization pond effluent was applied to the laboratory filters when possible to better compare laboratory and field results. The major parameters investigated were measured on a weekly basis and included BOD_5, algae or suspended solids, phosphorus, and nitrogen. As a prelude to a more detailed field study, pilot field filters were operated for a period of seven weeks in 1972 and five months in 1974.

At the levels of application studied in the laboratory, hydraulic loading rate was found to have little effect on any of the parameters studied. In the field experiments at much higher hydraulic loading rates and varying algae concentrations, suspended and volatile suspended solids removal appeared to decrease with an increase in hydraulic loading. Although significant quantities of applied algae were removed by filtration, cells were found to pass the entire bed depth. Sand size was found to have a general effect on the quality of the effluent produced by filtration. Sand size was also found to be related to the time of operation before plugging occurred. It was concluded that intermittent sand filtration was capable of upgrading a majority of the existing wastewater effluents in Utah to meet Class "C" water standards.

In addition to the above findings, it was concluded that:

1. Smaller effective size sands better oxidize nitrogen compounds.
2. Hydraulic loading rate has little effect on ability of the sand filter to oxidize nitrogen at the loading rates studied in the laboratory.
3. The nitrogen form which is being oxidized is that of ammonia.
4. Intermittent sand filters do not remove a significant quantity of dissolved phosphorus compounds.
5. Hydraulic loading rate has little effect on BOD_5 removal when secondary wastewater effluent is applied to intermittent sand filters with bed depths of 30 inches.
6. BOD removal increased as the effective size of the sand decreased. The 0.17 mm effective size sand filters produced a project low mean effluent BOD_5 concentration of 1.6 mg/l at the 0.4 mgpad loading rate and a high value of 4.1 mg/l at 0.8 mgpad. The project mean effluent BOD_5 concentration for the 0.72 mm effective size sand filters ranged from 5.0 to 5.5 mg/l for the 0.4, 0.5, and 0.6 mgpad hydraulic loading rates.
7. BOD_5 removal was independent of the applied BOD value at the concentrations studied in the laboratory.
8. Viable algal cells passed the entire depth of all the filter sands studied.
9. Hydraulic loading rate did not affect the algae or suspended solids removal efficiency at the 100,000 (153.4 m^3/hectare-day), or 200,000 (153.4 m^3/ hectare-day), or 300,000 (454.9 m^3/hectare-day) gallons per acre-day loadings employed in the laboratory study. The effects of hydraulic loading rate on SS removals in the field studies were inconclusive because of the large quantities of fines washed from the filters, but volatile suspended solids removals did indicate a reduction in removal efficiency as the hydraulic loading rate was increased.
10. Smaller effective size sands produced better algal or suspended and volatile suspended solids removals.
11. Sand size was not a significant factor in algae removal at applied algae concentrations of 15 and 30 mg/l, but was significant when the concentration was increased to 45-50 mg/l in both the laboratory and field filters.
12. Intermittent sand filtration produced a 90 percent reduction in the total coliform count in the laboratory filters.
13. Coliform removal was independent of the hydraulic loading rates employed in the laboratory filters.
14. Total bacterial counts as measured by the standard plate count apparently was not reduced by any of the sands studied.
15. Filter plugging causes no decline or improvement in the effluent BOD at or near the time of plugging.
16. Immediately before a filter plugged in the laboratory filter, the filter effluent suspended solids

concentrations were approximately zero. As the filter operates with time, the suspended solids removal efficiency increases reaching a maximum point at the time of plugging. This did not occur in the field, but if fines were washed from the filter before placing it in operation, it is likely that a similar pattern would occur.

17. At hydraulic loading rates of 0.4 to 0.6 mgpad the 0.17 mm effective size sand filters will operate approximately 100 days before cleaning is required when receiving a lagoon effluent containing a mean suspended solids concentration of 20 mg/l.

18. At loading rates of 0.7 and 0.8 mgpad the 0.17 mm filters will operate 32 consecutive days before requiring cleaning when receiving lagoon effluent containing a mean suspended solids concentration of 42 mg/l.

19. Laboratory filters containing sands of 0.72 mm effective size operated 175 consecutive days before plugging when dosed with a lagoon effluent containing a mean suspended solids concentration of 51 mg/l at a rate of 0.3 mgpad.

20. Field filters containing 0.72 mm effective size sand operated 137 consecutive days before terminating the study without plugging when loaded at 0.4, 0.5, and 0.6 mgpad with a lagoon effluent containing a mean suspended solids concentration of 25 mg/l.

21. If operated and loaded properly, all existing wastewater treatment plants in the State of Utah could be upgraded by intermittent sand filtration to meet Class "C" state standards.

22. Based upon current cost figures it appears that an effluent polishing intermittent sand filter process can be constructed and operated for a cost ranging between $15 to $47 per million gallons of filtrate.

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APPENDIXES

Appendix A

Tabulated Results of Phase I Laboratory

and Phase II Field Analyses

Table A-1. The results observed for the .17 mm effective sand size receiving effluent containing approximately 15 mg/l of algae (Loading Period II).

Number of Samples	Analysis		Hydraulic Loading - gpad 100,000	200,000	300,000
6	BOD_5	Applied (mg/l)	6.34	6.34	6.34
6		Effluent (mg/l)	1.17	1.26	1.96
		% Removal	81.5	80.1	69.1
45	Suspended Solids	Applied (mg/l)	13.7	13.7	13.7
7		Effluent (mg/l)	3.96	4.80	6.05
		% Removal	71.1	64.9	55.8
7	Volatile Suspended Solids	Applied (mg/l)	9.16	9.16	9.16
7		Effluent (mg/l)	1.99	2.14	2.30
		% Removal	78.3	76.6	74.9
18	Nitrite-N	Applied (mg/l)	.021	.021	.021
7		Effluent (mg/l)	.012	.033	.030
13	Nitrate-N	Applied (mg/l)	.110	.110	.110
7		Effluent (mg/l)	.958	.909	.910
		% Increase	873	828	829
22	Orthophosphate-P	Applied (mg/l)	1.05	1.05	1.05
7		Effluent (mg/l)	.034	.303	.449
		% Removal	96.7	71.2	57.8
19	Total Phosphorus-P (unfiltered)	Applied (mg/l)	1.18	1.18	1.18
7		Effluent (mg/l)	.056	.340	.485
		% Removal	95.2	71.2	59.0
36	pH	Applied	8.92	8.92	8.92
7		Effluent	8.22	8.25	8.27
49	Temperature	Ave. Applied (°C)	12.9	12.9	12.9

Table A-2. The results observed for the .35 mm effective sand size receiving effluent containing approximately 15 mg/l of algae (Loading Period II).

Number of Samples	Analysis		Hydraulic Loading - gpad 100,000	200,000	300,000
6	BOD_5	Applied (mg/l)	6.34	6.34	6.34
6		Effluent (mg/l)	2.44	2.08	2.41
		% Removal	61.5	67.2	62.0
45	Suspended Solids	Applied (mg/l)	13.7	13.7	13.7
7		Effluent (mg/l)	9.39	8.19	6.50
		% Removal	31.4	40.1	52.5
7	Volatile Suspended Solids	Applied (mg/l)	9.16	9.16	9.16
7		Effluent (mg/l)	3.38	3.33	3.40
		% Removal	63.1	63.6	62.8
18	Nitrite-N	Applied (mg/l)	.021	.021	.021
7		Effluent (mg/l)	.013	.018	.021
13	Nitrate-N	Applied (mg/l)	.110	.110	.110
7		Effluent (mg/l)	.841	.805	.735
		% Increase	766	733	669
22	Orthophosphate-P	Applied (mg/l)	1.05	1.05	1.05
7		Effluent (mg/l)	.192	.386	.487
		% Removal	81.7	63.2	53.7
19	Total Phosphorus-P (unfiltered)	Applied (mg/l)	1.18	1.18	1.18
7		Effluent (mg/l)	.285	.448	.553
		% Removal	75.9	62.1	53.2
36	pH	Applied	8.92	8.92	8.92
7		Effluent	8.27	8.28	8.20
49	Temperature	Ave. Applied (°C)	12.9	12.9	12.9

Table A-3. The results observed for the .72 mm effective sand size receiving effluent containing approximately 15 mg/l of algae (Loading Period II).

Number of Samples	Analysis		Hydraulic Loading - gpad 100,000	200,000	300,000
6	BOD_5	Applied (mg/l)	6.34	6.34	6.34
6		Effluent (mg/l)	2.33	2.50	1.93
		% Removal	63.2	59.8	69.6
45	Suspended Solids	Applied (mg/l)	13.7	13.7	13.7
7		Effluent (mg/l)	11.0	8.15	7.28
		% Removal	19.6	40.4	46.8
7	Volatile Suspended Solids	Applied (mg/l)	9.16	9.16	9.16
7		Effluent (mg/l)	3.85	4.00	3.17
		% Removal	57.9	56.3	65.4
18	Nitrite-N	Applied (mg/l)	.021	.021	.021
7		Effluent (mg/l)	.012	.032	.038
13	Nitrate-N	Applied (mg/l)	.110	.110	.110
7		Effluent (mg/l)	.815	.709	.757
		% Increase	742	645	670
22	Orthophosphate-P	Applied (mg/l)	1.05	1.05	1.05
7		Effluent (mg/l)	.409	.794	.887
		% Removal	61.0	24.4	15.6
19	Total Phosphorus-P (unfiltered)	Applied (mg/l)	1.18	1.18	1.18
7		Effluent (mg/l)	.365	.640	.669
		% Removal	69.1	45.9	43.4
36	pH	Applied	8.92	8.92	8.92
7		Effluent	8.31	8.48	8.51
49	Temperature	Ave. Applied (°C)	12.9	12.9	12.9

Table A-4. The results observed for the .17 mm effective sand size receiving effluent containing approximately 30 mg/l of algae (Loading Period I).

Number of Samples	Analysis		Hydraulic Loading - gpad 100,000	200,000	300,000
6	BOD_5	Applied (mg/l)	6.7	6.71	6.71
6		Effluent (mg/l)	1.15	1.55	2.31
		% Removal	82.9	76.9	65.6
30	Suspended Solids	Applied (mg/l)	31.0	31.0	31.0
5		Effluent (mg/l)	5.53	7.93	11.2
		% Removal	82.2	74.4	63.8
14	Nitrite-N	Applied (mg/l)	.002	.002	.002
6		Effluent (mg/l)	.0522	.1487	.1833
10	Nitrate-N	Applied (mg/l)	.034	.034	.034
6		Effluent (mg/l)	1.45	1.25	1.20
		% Increase	4302	3731	3576
14	Orthophosphate-P	Applied (mg/l)	.516	.516	.516
6		Effluent (mg/l)	.014	.034	.099
		% Removal	97.3	93.5	80.6
13	Total Phosphorus-P (unfiltered)	Applied (mg/l)	.832	.832	.832
6		Effluent (mg/l)	.027	.083	.166
		% Removal	96.7	89.9	80.1
31	pH	Applied	9.37	9.37	9.37
7		Effluent	8.39	8.41	8.42
39	Temperature	Ave. Applied (°C)	14.1	14.1	14.1

Table A-5. The results observed for the .35 mm effective sand size receiving effluent containing approximately 30 mg/l of algae (Loading Period I).

Number of Samples	Analysis		Hydraulic Loading - gpad 100,000	200,000	300,000
6	BOD_5	Applied (mg/l)	6.71	6.71	6.71
6		Effluent (mg/l)	2.51	2.61	2.97
		% Removal	62.6	61.1	55.7
30	Suspended Solids	Applied (mg/l)	31.0	31.0	31.0
5		Effluent (mg/l)	10.6	10.9	12.8
		% Removal	65.9	64.7	58.6
14	Nitrite-N	Applied (mg/l)	.002	.002	.002
6		Effluent (mg/l)	.1324	.1635	.1898
10	Nitrate-N	Applied (mg/l)	.034	.034	.034
6		Effluent (mg/l)	.989	1.12	1.16
		% Increase	2942	3327	3461
14	Orthophosphate-P	Applied (mg/l)	.516	.516	.516
6		Effluent (mg/l)	.055	.121	189
		% Removal	89.3	76.4	63.3
13	Total Phosphorus-P (unfiltered)	Applied (mg/l)	.832	.832	.832
6		Effluent (mg/l)	.101	174	.259
		% Removal	87.9	79.1	68.9
31	pH	Applied	9.37	9.37	9.37
7		Effluent	8.50	8.42	8.43
39	Temperature	Ave. Applied (°C)	14.1	14.1	14.1

Table A-6. The results observed for the .72 mm effective sand size receiving effluent containing approximately 30 mg/l of algae (Loading Period I).

Number of Samples	Analysis		Hydraulic Loading - gpad 100,000	200,000	300,000
6	BOD_5	Applied (mg/l)	6.71	6.71	6.71
6		Effluent (mg/l)	2.89	3.09	3.01
		% Removal	56.9	53.9	55.1
30	Suspended Solids	Applied (mg/l)	31.0	31.0	31.0
5		Effluent (mg/l)	13.7	11.9	10.9
		% Removal	56.0	61.6	64.7
14	Nitrite-N	Applied (mg/l)	.002	.002	.002
6		Effluent (mg/l)	.1178	.1115	.1332
10	Nitrate-N	Applied (mg/l)	.034	.034	.034
6		Effluent (mg/l)	1.06	1.02	1.09
		% Increase	3165	3028	3229
14	Orthophosphate-P	Applied (mg/l)	.516	.516	.516
6		Effluent (mg/l)	.159	.252	.273
		% Removal	69.2	51.2	47.1
13	Total Phosphorus-P (unfiltered)	Applied (mg/l)	.832	.832	.832
6		Effluent (mg/l)	.257	.339	.341
		% Removal	69.1	59.2	59.0
31	pH	Applied	9.37	9.37	9.37
7		Effluent	8.45	8.79	8.77
39	Temperature	Ave. Applied (°C)	14.1	14.1	14.1

Table A-7. The results observed for the .17 mm effective sand size receiving effluent containing approximately 45 mg/l of algae (Loading Period III).

Number of Samples	Analysis		Hydraulic Loading - gpad 100,000	200,000	300,000
18	BOD_5	Applied (mg/l)	36.5	36.5	36.5
7		Effluent (mg/l)	5.81	5.64	7.14
		% Removal	84.4	84.9	80.5
45	Suspended Solids	Applied (mg/l)	46.3	46.3	46.3
6		Effluent (mg/l)	1.86	1.93	5.33
		% Removal	96.7	96.0	88.5
11	Volatile Suspended Solids	Applied (mg/l)	41.3	41.3	41.3
6		Effluent (mg/l)	1.46	1.70	3.48
		% Removal	96.4	95.9	91.6
26	Ammonia-N	Applied (mg/l)	2.13	2.13	2.13
9		Effluent (mg/l)	.006	.004	.006
		% Removal	99.7	99.8	99.7
25	Nitrite-N	Applied (mg/l)	.039	.039	.039
6		Effluent (mg/l)	.018	.044	.066
26	Nitrate-N	Applied (mg/l)	.165	.165	.165
6		Effluent (mg/l)	4.04	3.57	3.89
		% Increase	2453	2164	2360
28	Orthophosphate-P	Applied (mg/l)	1.97	1.97	1.97
6		Effluent (mg/l)	.696	1.44	1.71
		% Removal	64.7	26.9	13.3
29	Total Phosphorus-P (unfiltered)	Applied (mg/l)	3.00	3.00	3.00
6		Effluent (mg/l)	.768	1.56	1.90
		% Removal	74.4	48.1	36.8
36	pH	Applied	7.89	7.89	7.8[illegible]
6		Effluent	8.42	8.32	8.29
	Temperature	Ave. Applied (°C)	17.3	17.3	17.3
3	Total Count Bacteria per ml	Applied	1,100,000	1,100,000	1,100,000
3		Effluent	99,000	1,100,000	1,200,000
		% Removal	~0	~0	~0
5	Coliform Colonies per 100 ml	Applied	610,000	610,000	610,000
		Effluent	0 (2)[a]	6,900 (5)[a]	8,800 (4)[a]
		% Removal	99 +	98.8	98.5

[a]Number of samples.

Table A-8. The results observed for the .35 mm effective sand size receiving effluent containing approximately 45 mg/l of algae (Loading Period III).

Number of Samples	Analysis		Hydraulic Loading - gpad 100,000	200,000	300,000
18	BOD_5	Applied (mg/l)	36.5	36.5	36.5
7		Effluent (mg/l)	11.2	10.8	11.5
		% Removal	69.3	70.6	68.4
45	Suspended Solids	Applied (mg/l)	46.3	46.3	46.3
6		Effluent (mg/l)	9.47	11.9	13.7
		% Removal	79.2	74.4	70.5
11	Volatile Suspended Solids	Applied (mg/l)	41.3	41.3	41.3
6		Effluent (mg/l)	7.28	7.14	8.31
		% Removal	82.4	82.7	79.9
26	Ammonia-N	Applied (mg/l)	2.13	2.13	2.13
9		Effluent (mg/l)	.006	.014	.018
		% Removal	99.7	99.3	99.2
25	Nitrite-N	Applied (mg/l)	.039	.039	.039
6		Effluent (mg/l)	.036	.039	.047
26	Nitrate-N	Applied (mg/l)	.165	.165	.165
6		Effluent (mg/l)	3.82	3.44	3.03
		% Increase	2317	2084	1835
28	Orthophosphate-P	Applied (mg/l)	1.98	1.98	1.98
6		Effluent (mg/l)	.780	1.42	1.74
		% Removal	60.6	27.9	11.8
29	Total Phosphorus-P (unfiltered)	Applied (mg/l)	3.00	3.00	3.00
6		Effluent (mg/l)	.986	1.71	2.00
		% Removal	67.2	42.9	33.4
39	pH	Applied	7.89	7.89	7.89
6		Effluent	8.32	8.27	8.18
	Temperature	Ave. Applied (°C)	17.3	17.3	17.3
3	Total Count Bacteria per ml	Applied	1,100,000	1,100,000	1,100,000
3		Effluent	1,100,000	910,000	1,200,000
		% Removal	~0	~0	~0
5	Coliform Colonies per 100 ml	Applied	610,000	610,000	610,000
		Effluent	16,000 (3)[a]	76,000 (3)[a]	150,000 (5)[a]
		% Removal	97.4	87.5	75.4

[a]Number of samples.

Table A-9. The results observed for the .72 mm effective sand size receiving effluent containing approximately 45 mg/l of algae (Loading Period III).

Number of Samples	Analysis		Hydraulic Loading - gpad 100,000	200,000	300,000
18	BOD_5	Applied (mg/l)	36.5	36.5	36.5
7		Effluent (mg/l)	12.3	12.7	13.3
		% Removal	66.3	65.0	63.7
45	Suspended Solids	Applied (mg/l)	46.3	46.3	46.3
6		Effluent (mg/l)	16.6	15.9	16.5
		% Removal	64.2	65.5	64.5
11	Volatile Suspended Solids	Applied (mg/l)	41.3	41.3	41.3
6		Effluent (mg/l)	10.1	13.1	13.2
		% Removal	75.5	68.2	68.1
26	Ammonia-N	Applied (mg/l)	2.13	2.13	2.13
9		Effluent (mg/l)	.043	.146	.217
		% Removal	97.9	93.1	89.8
25	Nitrite-N	Applied (mg/l)	.039	.039	.039
6		Effluent (mg/l)	.042	.100	.149
26	Nitrate-N	Applied (mg/l)	.165	.165	.165
6		Effluent (mg/l)	3.97	3.17	2.81
		% Increase	2408	1924	1707
28	Orthophosphate-P	Applied (mg/l)	1.97	1.97	1.97
6		Effluent (mg/l)	1.26	1.75	1.99
		% Removal	36.3	11.4	0.0
29	Total Phosphorus-P (unfiltered)	Applied (mg/l)	3.00	3.00	3.00
6		Effluent (mg/l)	1.49	2.10	2.20
		% Removal	38.5	29.9	26.8
39	pH	Applied	7.89	7.89	7.89
		Effluent	8.23	8.14	8.08
	Temperature	Ave. Applied (°C)	17.3	17.3	17.3
3	Total Count Bacteria per ml	Applied	1,100,000	1,100,000	1,100,000
3		Effluent	1,200,000	1,000,000	1,200,000
		% Removal	~0	~0	~0
5	Coliform Colonies per 100 ml	Applied	610,000	610,000	610,000
5		Effluent	17,000	16,000	66,000
		% Removal	97.2	97.3	89.1

Table A-10. The results observed for the .17 mm effective sand size placed in the field filters located at the Logan City wastewater stabilization ponds.

Number of Samples	Analysis		Hydraulic Loading - gpad 100,000	200,000	300,000
14	BOD_5	Applied (mg/l)	6.18	6.18	6.18
7		Effluent (mg/l)	1.12	1.19	.907
		% Removal	81.9	80.7	85.3
7	Suspended Solids	Applied (mg/l)	14.6	14.6	14.6
7		Effluent (mg/l)	21.8	34.9	18.1
		% Removal			
7	Volatile Suspended Solids	Applied (mg/l)	9.51	9.51	9.51
7		Effluent (mg/l)	7.01	4.64	4.02
		% Removal			
6	Ammonia-N	Applied (mg/l)	1.10	1.10	1.10
2		Effluent (mg/l)	.008	.018	.012
		% Removal	99.3	98.3	98.9
14	Nitrite-N	Applied (mg/l)	.032	.032	.032
7		Effluent (mg/l)	.002	.005	.001
14	Nitrate-N	Applied (mg/l)	.078	.078	.078
7		Effluent (mg/l)	1.20	0.928	0.859
		% Increase	1535	1185	1097
14	Orthophosphate-P	Applied (mg/l)	1.19	1.19	1.19
7		Effluent (mg/l)	.143	.528	.841
		% Removal	88.1	55.8	29.6
14	Total Phosphorus-P (unfiltered)	Applied (mg/l)	1.41	1.41	1.41
7		Effluent (mg/l)	.153	.369	.616
		% Removal	89.2	73.8	56.3
36	pH	Applied	8.92	8.92	8.92
7		Effluent	8.12	7.99	7.98
41	Temperature	Ave. Applied (°C)	13.7	13.7	13.7
	Air Temperature Range	(°F)	45 - 75	45 - 75	45 - 75

Table A-11. The results observed for the .72 mm effective sand size placed in the field filters located at the Logan City wastewater stabilization ponds.

Number of Samples	Analysis		Hydraulic Loading - gpad 100,000	200,000	300,000
14	BOD_5	Applied (mg/l)	6.18	6.18	6.18
7		Effluent (mg/l)	4.68	5.08	4.35
		% Removal	24.3	17.8	29.6
7	Suspended Solids	Applied (mg/l)	14.6	14.6	14.6
7		Effluent (mg/l)	80.0	33.5	17.0
		% Removal			
7	Volatile Suspended Solids	Applied (mg/l)	9.51	9.51	9.51
7		Effluent (mg/l)	13.1	9.66	6.97
		% Removal			
6	Ammonia-N	Applied (mg/l)	1.10	1.10	1.10
2		Effluent (mg/l)	.177	.285	.812
		% Removal	83.9	74.0	25.6
14	Nitrite-N	Applied (mg/l)	.032	.032	.032
7		Effluent (mg/l)	.073	.126	.066
14	Nitrate-N	Applied (mg/l)	.078	.078	.078
7		Effluent (mg/l)	1.35	1.40	.583
		% Increase	1725	1792	749
14	Orthophosphate-P	Applied (mg/l)	1.19	1.19	1.19
7		Effluent (mg/l)	.912	1.24	1.52
		% Removal	23.6	~0	~0
14	Total Phosphorus-P (unfiltered)	Applied (mg/l)	1.41	1.41	1.41
7		Effluent (mg/l)	.787	.884	1.03
		% Removal	44.2	37.3	27.2
[illegible]	pH	Applied	8.92	8.92	8.92
[illegible]		Effluent	8.49	8.58	8.68
41	Temperature	Ave. Applied (°C)	13.7	13.7	13.7
	Air Temperature Range	(°F)	45 - 75	45 - 75	45 - 75

Table A-12. The results observed for the 6 mm diameter rock placed in the field filters located at the Logan City wastewater stabilization ponds.

Number of Samples	Analysis		Hydraulic Loading - gpad 100,000	200,000	300,000
14	BOD_5	Applied (mg/l)	6.18	6.18	6.18
7		Effluent (mg/l)	5.34	5.5	3.93
		% Removal	13.6	11.0	36.4
7	Suspended Solids	Applied (mg/l)	14.6	14.6	14.6
7		Effluent (mg/l)	75.5	36.4	22.2
		% Removal			
7	Volatile Suspended Solids	Applied (mg/l)	9.51	9.51	9.51
7		Effluent (mg/l)	14.6	13.1	10.3
		% Removal			
6	Ammonia-N	Applied (mg/l)	1.10	1.10	1.10
2		Effluent (mg/l)	1.12	.917	1.20
		% Removal	~0	~0	~0
14	Nitrite-N	Applied (mg/l)	.032	.032	.032
7		Effluent (mg/l)	.053	.069	.057
14	Nitrate-N	Applied (mg/l)	.078	.078	.078
7		Effluent (mg/l)	.733	.461	.244
		% Increase	935	589	311
14	Orthophosphate-P	Applied (mg/l)	1.19	1.19	1.19
7		Effluent (mg/l)	1.57	1.16	1.48
		% Removal	~0	~0	~0
14	Total Phosphorus-P (unfiltered)	Applied (mg/l)	1.41	1.41	1.41
7		Effluent (mg/l)	1.25	1.25	1.30
		% Removal	~0	~0	~0
36	pH	Applied	8.92	8.92	8.92
7		Effluent	8.87	8.91	8.92
41	Temperature	Ave. Applied (°C)	13.7	13.7	13.7
	Air Temperature Range	(°F)	45 - 75	45 - 75	45 - 75

Table A-13. The suspended solids results observed for the .17 mm (.0067 in) sand size receiving Logan City wastewater stabilization pond effluent under field conditions (Phase II).

Sample Date	Mean Weekly Applied Suspended Solids mg/l	Effluent Suspended Solids mg/l Observed at Each Hydraulic Loading Rate									
		400,000	% Rem	500,000	% Rem	600,000	% Rem	700,000	% Rem	800,000	% Rem
5/26	4.43	42.0	-	55.2	-	26.6	-	-	-	-	-
5/31	5.49	8.28	-	56.8	-	15.2	-	-	-	-	-
May Ave.	4.96	25.1	-	56.0	-	20.9	-	-	-	-	-
6/6	5.21	9.62	-	79.2	-	10.6	-	-	-	-	-
6/13	7.48	28.8	-	36.2	-	17.8	-	-	-	-	-
6/19	8.81	14.2	-	27.4	-	17.2	-	-	-	-	-
6/26	4.51	10.3	-	12.7	-	12.5	-	-	-	-	-
June Ave.	6.50	15.7	-	38.9	-	14.5	-	-	-	-	-
7/3	12.3	14.4	-	25.3	-	25.9	-	-	-	-	-
7/12	32.3	14.6	54.8	21.2	34.4	21.0	35.0	13.0	59.8	12.3	61.9
7/19	30.4	12.4	59.2	30.0	1.3	12.3	59.5	20.0	34.2	14.3	53.0
7/26	44.3	15.2	65.7	19.3	56.4	21.5	51.5	31.3	29.3	19.7	55.5
July Ave.	29.8	14.2	52.3	23.9	19.8	20.2	32.2	21.4	28.2	15.4	48.3
8/2	44.9	17.0	62.1	31.7	29.4	21.3	52.6	13.1	70.8	61.3	-
8/9	49.0	6.3	87.1	8.5	82.7	Plugged	-	17.3	64.7	27.0	44.9
8/14	51.4	Plugged	-	Plugged	-	Plugged	-	Plugged	-	Plugged	-
8/22	38.6	46.0	-	16.5	57.3	32.5	15.8	70.5	-	25.0	35.2
8/28	37.0	23.3	37.0	18.7	49.5	36.3	1.9	37.0	-	43.0	-
Aug. Ave.	44.2	23.2	47.5	18.8	57.5	30.0	32.1	34.5	21.9	36.0	18.6
9/7	29.4	5.3	82.0	18.5	37.1	8.5	71.1	38.5	-	36.5	-
9/13	22.3	10.5	52.9	11.0	50.7	Plugged	-	16.5	26.0	7.00	68.6
9/19	23.3	9.0	61.4	11.0	52.8	Plugged	-	14.0	39.9	10.5	54.9
9/27	25.8	10.0	61.2	14.0	45.7	9.00	65.1	13.0	49.6	12.0	53.5
Sept. Ave.	25.2	8.7	65.5	13.6	46.0	8.8	65.1	20.5	18.7	16.5	34.5

Table A-14. The volatile suspended solids results observed for the .17 mm (.0067 in) sand size receiving Logan City wastewater stabilization pond effluent under field conditions (Phase II).

Sample Date	Mean Weekly Applied Volatile Suspended Solids mg/l	Effluent Volatile Suspended Solids (mg/l) Observed at Each Hydraulic Loading Rate - gpad									
		400,000	% Rem	500,000	% Rem	600,000	% Rem	700,000	% Rem	800,000	% Rem
5/26	2.48	3.60	-	5.20	-	2.20	11.2	-	-	-	-
5/31	-	0.71	-	3.80	-	1.00	-	-	-	-	-
May Ave.	2.48	2.17	12.5	4.50	-	1.60	-	-	-	-	-
6/6	2.98	2.25	24.4	3.80	-	1.25	58.0	-	-	-	-
6/13	3.98	1.13	71.6	1.44	63.8	1.33	66.5	-	-	-	-
6/19	4.49	1.00	77.7	2.00	55.4	1.56	65.3	-	-	-	-
6/26	2.32	1.89	18.5	2.33	-	1.20	48.3	-	-	-	-
June Ave.	3.44	1.57	54.4	2.39	30.5	1.34	61.0				
7/3	10.3	0.80	92.2	1.75	83.0	0.88	91.5	-	-	-	-
7/12	28.9	5.20	82.0	6.80	76.5	2.80	90.3	7.00	75.8	4.66	83.9
7/19	22.3	5.00	77.6	10.0	55.2	4.50	79.8	7.25	69.5	4.33	80.6
7/26	26.5	6.80	74.3	8.75	70.0	9.50	64.2	15.0	43.4	7.67	71.1
July Ave.	22.0	4.45	80.0	6.83	70.0	4.42	79.9	9.75	55.7	5.55	74.8
8/2	30.1	4.00	86.7	6.67	77.8	4.00	86.7	-	-	13.8	54.2
8/9	36.3	3.43	90.6	1.25	95.8	Plugged	-	9.50	73.8	17.4	52.1
8/14	44.9	Plugged	-	Plugged	-	Plugged	-	Plugged	-	Plugged	-
8/22	23.1	3.50	84.8	3.50	84.8	8.00	65.4	16.5	28.6	16.0	30.7
8/28	32.8	9.33	71.6	5.67	82.7	6.66	79.7	6.33	80.7	7.50	77.1
Aug. Ave.	33.4	5.07	84.8	4.27	87.2	6.22	81.4	10.8	67.7	13.6	59.3
9/7	23.1	1.66	92.8	12.0	48.1	3.00	87.0	13.5	41.6	15.5	32.9
9/13	20.3	6.00	70.4	4.00	80.3	Plugged	-	8.00	60.6	6.00	70.4
9/19	21.0	2.00	90.5	4.00	81.0	Plugged	-	2.50	88.1	6.00	71.4
9/27	20.7	1.00	95.2	2.50	88.0	2.00	90.3	2.50	88.0	6.00	71.0
Sept. Ave.	21.3	2.67	87.5	5.63	73.6	2.50	88.3	6.63	68.9	8.38	60.7

Table A-15. The suspended solids results observed for the .72 mm (.0283 in) sand size receiving Logan City wastewater stabilization pond effluent under field conditions (Phase II).

Sample Date	Applied Suspended Solids (mg/l)	Effluent Suspended Solids (mg/l) Observed at Each Hydraulic Loading Rate (gpad)					
		400,000	% Rem	500,000	% Rem	600,000	% Rem
5/26	4.43	33.0	-	8.20	-	15.2	-
5/31	5.49	30.4	-	6.80	-	16.6	-
May Ave.	5.0	31.7	-	7.50	-	15.9	-
6/6	5.21	3.33	36.1	6.37	-	6.57	-
6/13	7.49	22.00	-	16.5	-	26.6	-
6/19	8.81	-	-	7.25	17.7	12.0	-
6/26	4.51	9.50	-	7.60	-	4.90	-
June Ave.	6.50	11.6	-	9.43	-	12.5	-
7/3	12.3	28.0	-	18.2	-	19.2	-
7/12	32.3	16.3	49.5	16.4	49.2	15.6	51.7
7/19	30.4	16.0	47.4	10.8	64.5	16.4	46.1
7/26	44.3	11.3	74.5	12.3	72.2	16.3	63.2
July Ave.	29.8	17.9	39.9	14.3	52.0	16.9	43.3
8/2	44.9	15.3	43.7	20.6	54.1	40.0	10.9
8/9	48.0	17.0	64.6	14.3	70.2	18.0	62.5
8/14	51.4	21.7	57.8	30.0	41.6	30.7	40.3
8/22	38.6	46.0	-	21.0	45.6	25.5	33.9
8/28	37.0	65.0	-	26.0	29.7	20.5	44.6
Aug. Ave.	44.0	33.0	25.0	22.4	43.2	26.9	38.9
9/7	29.4	13.5	54.1	16.5	43.9	16.0	45.6
9/13	22.3	12.5	43.9	7.00	68.6	7.00	68.6
9/19	23.3	12.0	48.5	11.0	52.8	9.00	61.4
9/27	25.8	11.5	55.4	15.0	41.9	13.50	47.7
Sept. Ave.	25.2	12.8	49.2	12.4	50.8	11.4	54.8

Table A-16. The volatile suspended solids results observed for the .72 mm (.0283 in) sand size receiving Logan City wastewater stabilization pond effluent under field conditions (Phase II).

Sample Date	Weekly Mean Applied Volatile Suspended Solids (mg/l)	Effluent Volatile Suspended Solids (mg/l) Observed at Each Hydraulic Loading Rate (gpad)					
		400,000	% Rem	500,000	% Rem	600,000	% Rem
5/26	2.48	4.00	-	0.80	64.5	2.80	-
5/31	-	3.60	-	2.20	-	4.20	-
May Ave.	2.48	3.80	-	1.50	49.2	3.50	-
6/6	2.98	1.44	51.7	2.63	11.7	3.14	-
6/13	3.98	3.11	21.9	1.88	52.8	3.25	18.3
6/19	4.49	-	-	0.88	80.4	2.00	55.5
6/26	2.32	1.00	56.9	1.30	44.0	0.40	82.8
June Ave.	3.44	1.85	46.2	1.67	51.5	2.20	36.0
7/3	10.3	3.60	6.70	4.40	57.3	5.80	43.7
7/12	28.9	5.00	82.7	4.00	86.2	6.00	79.2
7/19	22.3	8.00	64.1	4.80	78.5	8.00	61.1
7/26	26.5	5.33	80.0	6.33	76.1	6.33	76.1
July Ave.	22.0	5.48	73.6	4.88	77.8	6.53	70.3
8/2	30.1	14.7	51.2	14.0	53.5	20.0	33.6
8/9	36.3	8.67	76.1	9.33	74.3	11.6	68.0
8/14	44.9	0.34	99.2	23.3	48.1	0.28	99.4
8/22	23.1	12.5	45.9	10.5	54.5	10.0	56.7
8/28	32.8	8.50	74.1	3.50	89.3	3.50	89.3
Aug. Ave.	33.4	8.93	73.3	12.1	63.8	9.09	72.8
9/7	23.1	1.50	93.5	1.50	93.5	4.50	80.5
9/13	20.3	6.00	70.4	2.50	87.7	1.50	92.6
9/19	21.0	6.50	69.0	4.00	81.0	2.00	90.5
9/27	20.7	5.00	75.8	0.50	97.6	8.50	58.9
Sept. Ave.	21.3	4.75	77.7	2.13	90.0	4.13	80.6

Table A-17. The BOD_5 results observed for the .17 mm (.0067 in) sand size receiving Logan City wastewater stabilization pond effluent under field conditions (Phase II).

Sample Date	Applied BOD_5 mg/l	Effluent BOD_5 (mg/l) Observed at Each Hydraulic Loading Rate - gpad									
		400,000	% Rem	500,000	% Rem	600,000	% Rem	700,000	% Rem	800,000	% Rem
6/20	-	0.5	-	1.1	-	1.4	-	-	-	-	-
6/27	12.1	1.0	91.7	1.3	89.3	1.2	90.0	-	-	-	-
June Ave.	12.1	0.75	93.8	1.2	90.1	1.3	89.3	-	-	-	-
7/11	11.3	1.0	88.5	0.8	92.9	0.8	92.9	4.4	61.1	-	-
7/19	13.1	2.4	81.7	1.2	90.8	1.4	89.3	4.3	67.2	4.2	67.9
7/25	13.3	1.1	91.7	0.6	95.5	1.1	91.7	1.8	86.5	2.4	82.0
July Ave.	12.6	1.5	88.1	0.87	93.1	1.1	91.3	3.5	72.2	3.3	73.8
8/1	15.4	2.9	81.2	4.1	73.4	3.2	79.2	2.9	81.2	3.2	79.2
8/9	15.6	0.4	97.4	3.2	79.5	Plugged	-	1.9	87.8	3.0	80.8
8/15	12.5	Plugged	-	Plugged	-	Plugged	-	Plugged	-	Plugged	-
8/23	10.0	2.4	76.0	1.9	81.0	2.7	73.0	5.1	49.0	5.1	49.0
8/29	11.1	3.1	72.1	3.0	73.0	3.4	69.4	6.7	38.7	6.0	45.9
Aug. Ave.	12.9	2.2	82.9	3.1	76.0	3.1	76.0	4.2	67.4	4.3	66.7
9/6	24.9	2.4	90.4	1.4	94.4	1.2	95.2	3.6	85.5	4.6	81.5
9/12	13.3	1.4	89.5	1.3	90.2	Plugged	-	3.1	76.7	5.8	56.4
9/19	14.5	1.5	89.7	1.5	89.7	Plugged	-	3.9	73.1	5.1	64.8
9/27	11.7	2.6	77.8	2.4	79.5	2.3	80.3	3.0	74.4	3.3	71.8
Sept. Ave.	16.1	2.0	87.6	1.7	89.4	1.8	88.8	3.4	78.9	4.7	70.8
Overall Ave.	13.9	1.61	88.4	1.69	87.8	1.81	87.0	3.68	73.5	4.11	70.4

Table A-18. The BOD_5 results observed for the .72 mm (.0283 in) sand size receiving Logan City wastewater stabilization pond effluent under field conditions (Phase II).

Sample Date	Applied BOD_5 (mg/l)	Effluent BOD_5 (mg/l) Observed at Each Hydraulic Loading Rate (gpad)					
		400,000	% Rem	500,000	% Rem	600,000	% Rem
6/20	-	5.1	-	3.5	-	4.0	-
6/27	12.1	3.9	67.8	3.6	70.2	3.7	69.4
June Ave.	12.1	4.5	62.8	3.6	70.2	3.9	85.1
7/11	11.3	5.2	54.0	5.8	84.0	6.3	44.2
7/19	13.1	6.8	48.1	6.1	53.4	6.0	54.2
7/25	13.3	4.2	68.4	5.2	60.9	5.4	59.4
July Ave.	13.6	5.4	60.3	5.7	58.1	5.9	56.6
8/1	15.4	4.3	72.1	6.6	57.1	7.7	50.0
8/4	15.6	5.2	66.7	4.8	69.2	4.8	69.2
8/15	12.5	6.8	45.6	5.3	57.6	7.0	44.0
8/23	10.0	7.3	27.0	5.8	42.0	6.6	34.0
8/29	11.1	7.1	36.0	6.7	39.6	8.1	27.0
Aug. Ave.	12.9	6.2	51.9	5.8	55.0	6.8	47.3
9/6	24.9	5.1	79.5	4.8	80.7	5.9	76.3
9/12	13.3	5.7	57.1	4.5	66.2	5.2	60.9
9/19	14.5	6.0	58.6	6.1	57.9	5.5	62.1
9/27	11.7	6.6	43.6	4.9	58.1	4.8	59.0
Sept. Ave.	16.1	5.9	63.4	5.1	68.3	5.4	66.5
Overall Ave.	13.7	5.47	60.1	5.04	63.2	5.5	59.9

Appendix B

International Biological Program

Algal Preservative

Prepare and use as follows:

Iodine - 10 gr.

KI - 20 gr.

Glacial Acetic Acid - 20 gr.

H_2O - 200 ml

Add 1 ml of preservative/100 ml of sample

Store in amber bottle.

Appendix C

Results of Statistical Analyses of Suspended and Volatile Suspended Solids and Fluorescence Data

Data Evaluated	Comparison Mode	Correlation Coefficient	Number of Samples	Degrees of Freedom	Significance Level	Equation of Best Fit
All Data	OD vs VSS	.727	175	173	1%	OD = 0.23 (VSS) + 4.52
	OD vs SS	.618	175	173	1%	SS = 2.20 (OD) + 9.21
Lagoon Effluent Data	OD vs VSS	.396	83	81	1%	OD = 0.12 (VSS) + 8.72
	OD vs SS	.577	83	81	1%	SS = 2.17 (OD) + 9.92
Filter Effluent Data	OD vs VSS	.392	92	90	1%	OD = 0.14 (VSS) + 4.29
	OD vs SS	.260	92	90	5%	SS = 1.72 (OD) + 11.43

Appendix D

Cost Estimating

Estimate 1

Existing facility flow: 1 mgd
Design hydraulic loading rate: 0.3 mgad
Locally available sand: .17 mm effective size @ 30" bed depth
Interest rate: 6%
Economic life: 20 yr.

Initial construction cost (in place):

	Quantity	Unit Cost	Total Cost
Granular media (.17 mm sand)	26,300 cy	$ 4.00	$105,200
Gravel	9,430 cy	$ 3.00	$ 28,300
4" lateral tile (8 ft. spacing)	36,600 LF	$ 2.00	$ 73,200
Main drain tile (10" dia.)	1,520 LF	$ 2.75	$ 4,180
Plastic bed lining	32,200 sy	$ 3.60	$116,000
Pumps - 3330 gpm	4 ea.	$3200.00	$ 12,800
Ductile iron pipe	2,440 LF	$ 9.50	$ 23,200
Excavation and embankment	41,200 cy	$ .50	$ 20,600
Land	7 acres	$ 500.00	$ 3,500
		Total	$384,590

75% of construction cost funded by federal monies.

Total cost to community $96,200 x .08719 = $8370.

Annual maintenance cost:	$ 750/yr
Annual operating costs:	
1 man-year @ $8,000	$ 8,000/yr
Electrical power	
$5/month x 12 month/year	$ 60/yr
Total annual cost:	$17,180

Cost per 10^6 gallons:

With federal assistance:

$$\frac{\text{Total annual cost}}{\text{Total annual flow}} = \frac{\$17{,}180}{365} = \$47 \text{ or } \$.047/1{,}000 \text{ gal.}$$

Without federal assistance:

$$\frac{\text{Total annual cost}}{\text{Total annual flow}} = \frac{\$42{,}310}{365} = \$115 \text{ or } \$.115/1{,}000 \text{ gal.}$$

Construction cost per acre:

$$\frac{\$384{,}590}{6.66 \text{ acre}} = \$57{,}800/\text{acre}$$

Estimate 2

Existing facility flow: 1 mgd
Design hydraulic loading rate: 0.8 mgad
Locally available sand: .17 mm effective size @ 30" bed depth
Interest rate: 6%
Economic life: 20 yr.

Initial construction cost (in place):

	Quantity	Unit Cost	Total Cost
Granular media (.17 mm sand)	9,850 cy	$ 4.00	$ 39,400
Gravel	3,800 cy	$ 3.00	$ 11,400
4" lateral tile (8 ft. spacing)	13,500 LF	$ 2.00	$ 27,000
Main drain tile (10" dia.)	640 LF	$ 2.75	$ 1,760
Plastic bed lining	12,900 SY	$ 3.60	$ 46,400
Pumps - 6700 gpm	2 ea.	$4500.00	$ 99,000
Ductile iron pipe	940 LF	$ 11.00	$ 10,400
Excavation and embankment	15,700 cy	$ .50	$ 7,850
Land	3 acres	$ 500.00	$ 1,500
		Total	$154,710

75% of construction cost funded by federal monies.

Total cost to community $38,700 x .08719 = $3,370.

Annual maintenance cost:	$ 750/yr
Annual operating costs:	
1 man-year @ $8,000	$ 8,000/yr
Electrical power	
$5/month (12 months/year)	$ 60/yr
Total annual cost:	$12,180

Cost per 10^6 gallons:

With federal assistance:

$$\frac{\text{Total annual cost}}{\text{Total annual flow}} = \frac{\$12,180}{365} = \$33 \text{ or } \$.033/1,000 \text{ gal.}$$

Without federal assistance:

$$\frac{\text{Total annual cost}}{\text{Total annual flow}} = \frac{\$22,310}{365} = \$61 \text{ or } \$.061/1,000 \text{ gal.}$$

Construction cost per acre:

$$\frac{\$154,710}{2.5 \text{ acre}} = \$61,800/\text{acre}$$

Estimate 3

Existing facility flow: 1 mgd
Design hydraulic loading rate: 0.8 mgad
Specially prepared sand: .35 mm, .72 mm effective size @ 30" bed depth
Interest rate: 6%
Economic life: 20 yr.

Initial construction cost (in place):

	Quantity	Unit Cost	Total Cost
Granular media (.35 mm, .72 mm)	9,850 cy	$ 40.00	$394,000
Gravel	3,800 cy	$ 3.00	$ 11,400
8" tile laterals (4 ft. spacing)	27,000 LF	$ 2.50	$ 67,500
Main drain tile (14" dia.)	640 LF	$ 3.25	$ 2,080
Plastic bed lining	12,900 sy	$ 3.60	$ 46,400
Pumps - 6700 gpm	2 ea.	$4500.00	$ 9,000
Ductile iron pipe	940 LF	$ 11.00	$ 10,400
Excavation and embankment	15,700 cy	$ .50	$ 7,850
Land	3 acre	$ 500.00	$ 1,500
		Total	$550,130

75% of construction cost funded by federal monies.

Total cost of community $138,000 x .08719 = $12,050.

Annual maintenance cost:	$ 750/yr
Annual operating cost:	
1/2 man-year @ $8,000	$ 4,000/yr
Electrical power	
$5 month (12 months/year)	$ 60/yr
Total annual cost	$16,860

Cost per 10^6 gallons:

With federal assistance:

$$\frac{\text{Total annual cost}}{\text{Total annual flow}} = \frac{\$16,860}{365} = \$46 \text{ or } \$.046/1,000 \text{ gal.}$$

Without federal assistance:

$$\frac{\text{Total annual cost}}{\text{Total annual flow}} = \frac{\$52,910}{365} = \$145 \text{ or } \$.145/1,000 \text{ gal.}$$

Construction cost per acre:

$$\frac{\$550,130}{2.5 \text{ acre}} = \$220,000/\text{acre}$$

Estimate 4

Existing facility flow: 15 mgd (Logan City)
Design hydraulic loading rate: 0.6 mgad
Locally available sand: .17 mm effective size @ 30" bed depth
Interest rate: 6%
Economic life: 20 yr.

Initial construction cost (in place):

	Quantity	Unit Cost	Total Cost
Granular media (.17 mm sand)	198,000 cy	\$ 4.00	\$ 785,000
Gravel	76,500 cy	\$ 3.00	\$ 230,000
4" lateral tile (8 ft. spacing)	260,000 LF	\$ 2.00	\$ 520,000
Main drain tile (10" dia.)	15,100 LF	\$ 2.75	\$ 41,600
Plastic bed lining	265,000 sy	\$ 3.60	\$ 955,000
Excavation and embankment	56,000 cy	\$ 1.00	\$ 56,000
Ductile iron pipe	10,070 LF	\$11.00	\$ 110,000
		Total	\$2,697,600

75% of construction cost funded by federal monies.

Total cost of construction \$674,000 x .08719 = \$58,800.

Annual maintenance cost: \$ 1,000/yr

Annual operating costs:
3 man-years @ \$8,000 \$24,000/yr

\$83,800

Cost per 10^6 gallons:

With federal assistance:

$$\frac{\text{Total annual cost}}{\text{Total annual flow}} = \frac{\$83,800}{15(365)} = \$15 \text{ or } \$.015/1,000 \text{ gal.}$$

Without federal assistance:

$$\frac{\text{Total annual cost}}{\text{Total annual flow}} = \frac{\$260,000}{15(365)} = \$48 \text{ or } \$.048/1,000 \text{ gal.}$$

Construction cost per acre:

$$\frac{\$2,697,600}{50 \text{ acre}} = \$53,700/\text{acre}$$