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Series Intermittent Sand Filtration of Wastewater Lagoon Effluents

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SERIES INTERMITTENT SAND FILTRATION OF WASTEWATER LAGOON EFFLUENTS

by

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ABSTRACT

Previous researchers have found single stage intermittent sand filtration to be a feasible and economic means of upgrading wastewater lagoon effluent to meet future standards. However the major constraint on their use has been the length of the filter runs.

Laboratory scale and pilot-scale series intermittent sand filtration of wastewater lagoon effluents has been found to substantially increase the length of the filter runs as well as produce a high quality effluent able to meet future standards. Higher loading rates were found to be possible with series intermittent sand filtration. The operation consistently produced an effluent meeting present Utah "Class C" water quality standards for BOD_5 ($\leqslant$ 5 mg/l), and the operation also consistently met the 1980 Utah wastewater treatment plant effluent standard for suspended solids ($\leqslant$ 10 mg/l).

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INTRODUCTION

Nature of the Problem

Because lagoons provide simple, efficient, and economical wastewater treatment, over 4,000 communities in the United States use this type of wastewater treatment. Approximately 90 percent of these communities have populations of less than 5,000 people. These small communities are usually lacking in resources and competent personnel. Often only periodic inspection or maintenance is carried out by the general municipal employees.

Historically wastewater lagoons have provided economical wastewater treatment for small communities, but the effluent from lagoons may not meet future wastewater treatment plant effluent standards under PL 92-500. In addition, many states are imposing stringent discharge standards. Table 1 summarizes the effluent and in-stream standards for the State of Utah. In order to meet these standards, an inexpensive treatment which does not require sophisticated or constant operation and maintenance is needed for "upgrading" lagoon effluent.

Many sections of the country are still fortunate to be surrounded by large areas of open and relatively inexpensive land, and many smaller communities adopted waste stabilization lagoons as a means of wastewater treatment. However, now a better quality effluent is necessary.

If these smaller communities 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.

The use of single stage intermittent sand filtration has been shown to be a feasible method of upgrading lagoon effluents (Marshall and Middlebrooks, 1974; Reynolds et al., 1974). However, the major constraint on the use of single stage intermittent sand filters has been the length of the filter runs. Optimization of the intermittent sand filtration process could provide a simple, economic, and low maintenance method of publishing wastewater lagoon effluents.

Objectives

The general objective of this study is to determine the feasibility of using series intermittent sand filters to upgrade wastewater lagoon effluents.

Table 1. Stream standards and effluent standards for the State of Utah.

Parameter	Average Effluent Concentration			
	Stream Standards[a]		Effluent Standards[b]	
	Class D	Class C	June 30, 1977	June 30, 1980
pH	6.5 - 9.0	6.5 - 8.5	6.5 - 9.0	6.5 - 9.0
Total Coliform	5000/100 ml	5000/100 ml	2000/100 ml	200/100 ml
Fecal Coliform	--	2000/100 ml	200/100 ml	20/100 ml
BOD_5	25 mg/l	5 mg/l	25 mg/l	10 mg/l
Suspended Solids	--	--	25 mg/l	10 mg/l
Dissolved Oxygen	--	> 5.5 mg/l	--	--
Chemical and Radiological	PHS Standards[c]	PHS Standards[c]	--	--

[a]From Utah Water Pollution Control Act, amended 1967, Utah State Division of Health.

[b]From an abstract of the order dated June 19, 1974, Utah State Board of Health.

[c]USPHS, Public Drinking Water Standards, 1962.

To satisfy the general objective, the following specific objectives were undertaken:

1. Evaluate the performance of pilot scale and laboratory scale series intermittent sand filters for polishing lagoon effluents.
2. Compare the performance of a series intermittent sand filter operation with that of a single stage intermittent sand filtration operation previously studied by other researchers.
3. Develop design criteria for a full scale series intermittent sand filtration operation.
4. Determine the overall cost for construction and operation of a series intermittent sand filter operation.

REVIEW OF LITERATURE

General

Beds of sand have been in use for upgrading culinary water and wastewater for many years. Currently rapid sand filtration is a popular method of water treatment, and the use of intermittent sand filtration has been brought back into focus for the purpose of upgrading wastewater lagoon effluents to meet future standards.

A rather detailed literature review will be attempted in order to cover all facets of intermittent sand filtration. Marshall and Middlebrooks (1974) presented a lengthy and detailed literature review covering slow sand filters, intermittent sand filters, and related media such as rapid sand filters. In this paper, these areas will be covered and in addition sand filtering and clogging mechanisms, mathematical modeling of intermittent sand filters, and the use of intermittent sand filtration to upgrade wastewater lagoon effluents will be discussed.

A review of the filter cleaning methods used in slow sand filtration, intermittent sand filtration, and rapid sand filtration will also be presented. The scraping and washing of the clogged sand may constitute the major maintenance and operation consideration of intermittent sand filters used to upgrade wastewater lagoon effluents. The filter cleaning methods reviewed may be pertinent for use with intermittent sand filters used to upgrade lagoon effluents.

Review of Slow Sand Filtration of Potable Water

General history

The use of slow sand filtration for the filtering of culinary water began at the Chelesea Water Works in 1828 (Oakes, 1943). The natural process of water percolating through sand appealed to many municipalities who attempted to upgrade the quality and safety of their culinary water. The simplicity of operation and economics were the biggest advantages of the slow sand filtration of water.

Slow sand filtration spread rapidly throughout Europe. About the turn of the century slow sand filters appeared on the Eastern coast of the United States. Many of the larger populated areas could no longer supply clean, safe water for their citizens, and therefore turned to the use of slow sand filtration to upgrade their potable water supply. Slow sand filtration of water was in general use in the 1920s, but by 1930 had declined due to increasing land cost and new methods of water treatment. However Karalekas (1952) and Smith (1945) noted that several were still in operation during the middle of the twentieth century.

Design

The literature indicates that the basic design of slow sand filters has remained relatively unchanged over the years. Most designs called for an adequate distribution and collection system, and an adequate filter media enclosed within some permanent concrete structure. Many of the slow sand filters were covered, and the operation was usually built in open areas where future expansion could take place.

The designs called for a bed of sand 18 to 60 inches (0.457 to 1.52 m) in depth supported by a layer of gravel 12 to 18 inches (30.5 to 45.7 cm) in depth (Anonymous, 1912b, 1918; de Varona, 1909; Gregory, 1914; Karalekas, 1952; Saville, 1924; Story, 1909). The supporting gravel was laid in three or four equal layers of different size aggregate. Coarse gravel was laid at the structure bottom around the underdrains, while the finest grade of gravel was laid directly beneath the bed of sand (Gregory, 1914; Karalekas, 1952). One installation at El Centro, California, however, reports that only a 6 inch (15.2 cm) gravel layer composed of less than 1/4 inch (0.63 cm) aggregate was used to support the bed of sand (Anonymous, 1918).

The sand used in slow sand filters is specified by its effective size and uniformity coefficient. Effective size is defined as the size of grid through which 10 percent of the sand will pass, while uniformity coefficient is defined as the size of grid through which 60 percent of the sand will pass divided by the size of grid through which 10 percent of the sand will pass. The design specifications called for a clean, well graded sand having an effective size between 0.21 mm (0.0083 inch) and 0.35 mm

(0.0138 inch), and having a uniformity coefficient less than 2.75 (Anonymous, 1912b, Bailey, 1937; de Varona, 1909; Karalekas, 1952; Mitchell, 1921; Saville, 1924; Story, 1909).

The ultimate filtering capacity of the slow sand filters often was determined by the size of the underdrains. These underdrains had to be large enough to carry away the flow without having filtered water back up into the filter. Collector channels covered with open-jointed tile was a popular design (Anonymous, 1918; Story, 1909), as were open-jointed vitrified clay pipe (de Varona, 1909; Karalekas, 1952) and terra-cota pipe split longitudinally (de Varona, 1909).

The hydraulic loading rates used in slow sand filtration depended upon the effective size of the sand, the influent suspended solids concentration, and the influent turbidity.

The hydraulic loading rates varied from 1,500,000 gallons per acre per day (gpad) (14,030.9 cubic meters/hectare-day (m^3/h-d)) to 3,000,000 gpad (28,061.8 m^3/h-d) (de Varona, 1909; Gregory, 1914; Jordon, 1909; Karalekas, 1952; Mitchell, 1921). However, due to increased water demands, these loading rates often were increased. Constant hydraulic loading rates varying from 5,000,000 gpad (46,769.6 m^3/h-d) to 8,000,000 gpad (74,831.4 m^3/h-d) were commonly used, and peak loadings of 15,000,000 gpad (140,308.9 m^3/h-d) and 29,000,000 gpad (271,263.9 m^3/h-d) were noted at Springfield, Massachusetts, and El Centro, California, respectively (Anonymous, 1918; Karalekas, 1952). Higher hydraulic loading rates were accomplished by better distribution and collection systems and improved filter cleaning methods.

Operation

Initially, slow sand filters were operated by applying a continuous volume of raw or chemically treated water to the filter until a predetermined headloss (usually 3 to 4 feet; 0.914 to 1.22 m) was reached. At this point the filter was taken out of service and usually allowed to dry. Men and machines would then go onto the filter to recondition the surface by one of several methods.

One method of reconditioning involved scraping the top 2 inches (5.1 cm) of sand, transporting the scraped sand by hydraulic ejectors to a sand washer, washing the sand, storing the sand or transporting it back to the filter, and restarting the flow to the slow sand filter at a slow rate until the filter became "ripened" (a "schmutzdecke" or filtering skin buildup) at which time normal hydraulic loading rates were used (Fuller, 1908a; Gaub, 1915).

Another filter reconditioning method involved intensely raking the surface of the slow sand filter to breakup the surface mat. Story (1909) reported that raking followed by a drying period, provided an economical method of restoring the filter to its original filtering ability. Saville (1924), at the Hartford, Connecticut, plant, found that four rakings between scrapings provided an economical method of maintenance. Saville (1924) reported that five men could rake a bed in 2 hours, while it took eleven men 16 hours to scrape and wash the same bed.

A simple method of filter cleaning called the Brooklyn method was reported by de Varona (1909), Fuller (1908b), and Gaub (1915). The Brooklyn method consisted of lowering the water depth over the filter to just a few inches. Boards were driven in the sand surface to separate the sand filter into sections. After this, unfiltered water was run in streams over each section of the sand filter, while men with rakes and shovels agitated the sand to suspend the dirt and organic matter. Gaub (1915) considered this method costly because it required considerable manpower. A foreman and 14 men were required to clean a 0.75 acre (0.384 hectare) bed in one 8 hour working day (Gaub, 1915).

Hydraulic ejectors, reported by Gaub (1915) and Karalekas (1952), utilized water under pressure entering from the bottom while sand was shoveled into the top. This formed a suspension between the sand and water which could be transported by lengths of hoses to sand washers, storage bins, or slow sand filters. Gaub (1915) reported that the ejectors had a tendency to stratify the sand and recommended specifying a low uniformity coefficient sand for use in slow sand filters.

The mechanical sand washers utilized a method of agitating the dirty sand by clean wash water so that the organic material, fines, and debris were suspended and withdrawn to waste. The sand being heavier dropped to the bottom of the machine and was transported by ejectors to filters or storage. Gaub (1915) reported that the Nichlas washer was the predominant sand washer in use during the early 1900s, while Karalekas (1952) reported the use of Allan Hazen sand washers at Springfield, Massachusetts, in 1952.

Fuller (1908a, 1908b) reported that the use of cleaning methods (scraping and raking by manpower) were seriously retarding the use of slow sand filters, and that the use of a mechanical sand washer which washed the sand as it lay in place on the filter while the slow sand filter was in full operation. Other authors in the literature have also reported the use of the Blaisdell machine and how higher loading rates were possible from its use (Anonymous, 1918; Bailey, 1937; de Varona, 1909; Gaub, 1915).

Smith (1945) referred to a machine used by the McMillan Slow Sand Plant which operated on a dry bed. A screw conveyer attached to the front pushed sand into a receiving box. This receiving box was attached to an ejector which transported the sand to a sand separator located at the top of the machine. The sand was washed and deposited on the filter behind the machine as it traveled across the filter. This machine could scrape and clean 7 cubic yards (5.35 m^3) of sand an hour. Smith (1945) also noted that a mechanical raking machine was used to break up the "schmutzdecke" (filtering skin) between filter cleanings at this plant.

Operational problems have been cited by several authors (Flu, 1922; Madiley, 1921, 1927; Story, 1909). Flu (1922) reported that insects, crabs, and fish created a nuisance at the Weltercreden, Dutch East Indies, slow sand filter plant because they bored through the filtering layer. Madiley (1921) agreed with Flu's assessment that fish, crabs, and insects caused a deterioration in the effluent quality because of the breaking and floating of the filtering skin ("schmutzdecke"). Madiley (1921) noted that the sunlight caused excessive algal growth in the filtering skin, and suggested putting screens over the influent pipes to solve the fish and crab problem and increase the depth of water over the filters to solve the algal growth problem.

On the same filters at Madras City, Madiley (1927) cited a failure of slow sand filters. Ferrous sulfide presence in the quartz filter sand produced hydrogen sulfide gas in the hot, humid climate. The gas collected in pockets within the sand bed and eventually burst through the filtering layer. No amount of cleaning of the filters or pretreatment of the water seemed to help. It is emphasized that the problems cited by Flu (1922) and Madiley (1921, 1927) took place in tropical climates. Madiley (1921) stated that slow sand filters located in the tropics worked quite differently than slow sand filters located in more moderate climates such as England.

Story (1909) reported on the operation of filters at Ludlow Reservoir at Springfield, Massachusetts. This was a temporary solution for Springfield until a new source of potable water could be found. In June 1907, when the slow sand filters were placed into operation for the year, the raw water had large numbers of *Uroglena* sp. and *Asterionella* sp. (diatom). These organisms formed a cement like layer on the filter causing rapid clogging. Story (1909) found that intense raking followed by a period of sunlight and drying worked almost as well as scraping in renewing the filtering ability of the sand. *Anabaena* sp. (a blue-green alga) appeared in the water supply in late June, and when these organisms died they created numerous problems for the filtering plant. Lengths of filter runs were short and taste and odor problems plagued the plant. However, the filters continued to give a good quality effluent through this difficult period. Story (1909) found that subsurface clogging had taken place during the summer of 1907, because new sand had been laid over old unscraped sand during the spring. The clogged sand was removed and length of filter runs improved slightly. Story (1909) reported that intermittent sand filtration was also tried at the Ludlow plant, but this produced about the same results as slow sand filtration.

Effluent quality

Slow sand filtration was originally used for the purpose of filtering out the bacteria in culinary water. Chlorination was unknown or unavailable during the early years of slow sand filtration. Jordan (1909) reported a 99 percent removal of bacteria at the Indianapolis Water Treatment Plant. A slow sand filter at Toronto, Canada, reported an efficiency of 99.7 percent removal of E-coli during 1918 (Howard, 1919). In the period of 1905-1916, the District of Columbia reported a reduction in bacteria from 5,540 per cubic centimeter average influent concentration to a value of 15 per cubic centimeter average effluent concentration (Anonymous, 1917). Two slow sand filters at Camp Perry, Ohio, reported removal rates of 90 percent (Anonymous, 1912a).

Madiley (1921) and Flu (1922) reported reductions of over 90 percent in total bacteria for filters operated in the tropics. Flu (1922) reported that sunlight was beneficial in slow sand filtration because it helped to kill bacteria. Madiley (1921) determined that bacteria removal was very dependent upon the condition of the filtering skin ("schmutzdecke"). Periods of "schmutzdecke" deterioration resulted in a greater number of bacteria in the effluent.

Willcomb (1913) and an anonymous author (1912a) reported that the limiting depth of bacterial penetration into slow sand filters appears to be 10 inches (25.4 cm). Powell (1911) stated that deeper beds afford larger bacterial removal than shallower beds and that the "schmutzdecke" (filtering skin) did the majority of the work in bacterial removal.

Turbidity at the District of Columbia Plant was reduced from an influent value of 238 to an effluent value of slightly greater than 0 (Anonymous, 1917). Race (1915) reported no significant removal of color by slow sand filters, but Story (1909) reported reductions of 55 percent at the Ludlow filter plant. Clark (1925) reported color reductions of 25 to 30 percent; however, the color reductions rose to 60 to 90 percent when chemicals were used with slow sand filters.

Upgrading slow sand filter plants

Most efforts at upgrading slow sand filter performance consisted of improved distribution channels, improved collection networks, covering the filters, and improved filter cleaning methods (Anonymous, 1918; de Varona, 1909; Fuller, 1908a, Karalekas, 1952; Smith, 1945). All of these efforts helped to increase the volume of raw water filtered per day.

Sattler (1941) reported that the length of slow sand filter runs were increased six times by the use of flocculation and a pre-treatment rapid sand filter and that slow sand filter runs were doubled by the use of a coarse pre-filter. Mitchell (1921) experimented with a series slow sand filter operation at Aberdeen, England. The first filter in the series was a rapid sand filter having an effective size sand of 0.40 mm (0.0158 inch) and having a loading rate fifty times the standard rate. The standard rate was 4 inches (10.2 cm) of infiltration per hour. The intermediate and final filters in the series were slow sand filters having an effective size sand of 0.32 mm (0.013 inch) and having loading rates of one and one-half times the standard loading rate. After 1,530,000,000 gallons of raw water per acre (14,311,509 m^3/hectare) had passed through the operation, the intermediate slow sand filter had not plugged, while the final check slow sand filter had plugged 16 times. This is equivalent to an average final check slow sand filter run of approximately 28 days at a 3.4 mgad (31,801.2 m^3/h-d) hydraulic loading rate.

Mitchell (1921) also reported a method of filter cleaning at Worcester, England. The basic Brooklyn method was used, but the bed was left slightly dirty as this would "ripen" (build-up of "schmutzdecke") the filter faster, but yet not affect the normal loading rates.

Bailey (1937) reported the possibility of using some media other than sand to produce a high quality effluent. At the Eastman Kodak Company, he removed the top 4 inches (10.2 cm) of sand and replaced it with 4 inches (10.2 cm) of anthracite coal. He compared the anthracite filter's performance with a regular slow sand filter composed of 0.18 mm (0.0071 inch) effective size sand. The filters were treated similarly, and no significant difference was found in the quality of the two filter effluents. However the anthracite coal filter required cleaning only one-third as often as the slow sand filter. Anthracite coal also was shown to be hard and durable enough to take the abuse of filter cleaning.

Economics

A very crude attempt has been made to update the reported construction and operating costs to December 1975 values by means of the Engineering News Record (1975) Cost Indexes. The construction costs were updated using the building cost index while the operating costs were updated using the construction cost index. The building cost index is probably more applicable in measuring the degree of change in construction because its labor component is more representative of labor's share of the total cost of labor and materials in most types of construction. However, the construction cost index has a large common labor component and may better represent the degree of change in operating costs. The first figure shown in the following paragraphs is the reported cost from the literature while the updated cost is shown in parentheses.

Only one source, Story (1909) reports construction costs for a slow sand filter plant; $50,724 ($682,262.91) for 4 acres (1.62 hectares) in 1907. However this plant was only a temporary plant as it contained no permanent structure to hold the filter media. The cost did not include land costs, but did include the cost of an aerator used before the filters. From the information available, a present day construction cost of approximately $170,566 per acre may be assumed.

The District of Columbia reported operating costs ranging from $0.84 ($21.25) in 1912 to $1.39 ($32.99) in 1908 per million gallons of filtered water (Anonymous, 1917). Story (1909) reported operating costs of $5.73 ($133.23) per million gallons of filtrate in 1907-08, while Saville (1924) reported costs ranging from $4.61 ($60.99) in 1922 to $5.15 ($55.40) in 1923 per million gallons of filtrate at Hartford, Connecticut. Jordon (1909) reported that operating costs and construction costs should not exceed $5.00 ($126.47) in 1909 per million gallons of filtered water. Updated to 1975 values, these operating costs would range approximately from $20 to $135 per million gallons of filtrate.

Intermittent Sand Filtration of Wastewater

General history

The use of intermittent sand filtration to treat sewage originated in England over 130 years ago (Daniels, 1945). The first large application of intermittent sand filtration was an operation totaling 20 acres (8.09 hectares) at Merthyr Tydfil, Wales, in 1871 (Pincince and McKee, 1968). In 1889, S. C. Heuld built the first intermittent sand filter in this country at Framingham, Massachusetts. Their use spread rapidly throughout New England. By 1945, 448 intermittent sand filter plants were in operation in the United States; however, this figure declined to 398 by 1957 (American Society of Civil Engineers and Water Pollution Control Federation Joint Committee, 1959; hereafter referred to as ASCE-WPCF, 1959). In 1957, 94 percent of the intermittent sand filters were located in communities of less than 10,000 people.

The Lawrence Experiment Station at Lawrence, Massachusetts, began to study the intermittent sand filter in 1890. Many small communities in the area were in need of an economical method of treating wastewaters at a central location. Land was economically available and well-graded bank sand was found often already in place. The research at the Lawrence Experiment Station encouraged other communities to adopt intermittent sand filters to treat sewage (Massachusetts Board of Health, 1912).

The State of Florida experienced rapid growth during the early World War II years. This growth continued after the war with retiring people moving to the state. Many isolated installations such as motels, trailer courts, schools, and small housing developments were constructed. Because Florida has such a high water table, the use of septic tanks and leaching fields could not be used to treat these wastewaters (Calaway, 1957). A simple and economic method of treating wastewater had to be developed. The Sanitary Research Laboratory at the University of Florida attempted to develop and improve the use of intermittent sand filtration with eight 7.4 sq. ft. (0.687 m^2) pilot scale intermittent sand filters to treat settled sewage. Much of the literature and knowledge of intermittent sand filtration performance we have today was developed at the University of Florida (Calaway et al., 1952; Furman et al., 1955; Grantham et al., 1949).

Design

In intermittent sand filtration, wastewater was applied intermittently to a natural or man-made bed of porous media and allowed to percolate through the media to underdrains or to the groundwater (Pincince and McKee, 1968; Salvato, 1972). As might be expected the design of intermittent sand filters was similar to the design of slow sand filters.

The intermittent sand filter site was governed by topography, ultimate disposal of effluent, length of outfall, pumping requirements, length of filter media haul, and isolation from nearby housing. In some cases, a site was chosen where a clean well-graded sand occurred naturally, in others the site was chosen at an economical haul distance from a sand source. The land had to be stripped and the top soil and subsoil removed. The wasted soil could be used for building embankments around the filters. The use of soil embankments was the most economical construction method, but because of weed growth and erosion, soil embankments required the most maintenance. The embankments had to be mowed continually to keep the vegetation from encroaching on the sand filters. These embankments were laid on a 1:1 to 1:1.5 vertical to horizontal slope and were rarely higher than 18 inches (45.7 cm) (ASCE-WPCF, 1959; Metcalf and Eddy, 1935). Many sources recommended the use of concrete sidewalls between and around the filters (ASCE-WPCF, 1959; Hansen, 1910; Metcalf and Eddy, 1935).

A minimum of three intermittent sand filters was recommended in order to compensate for a filter which was out of service due to scraping and cleaning (ASCE-WPCF, 1959). Metcalf and Eddy (1935) and Steel (1960) recommended that filter sizes should be no larger than 3/4 to 1 acre (0.303 to 0.405 hectare) in size.

The bottom of intermittent sand filters was composed of natural soils in contrast with the concrete bottoms of slow sand filters. The filter bottoms were graded toward the lines of underdrains at a slope of at least 6 inches (15.2 cm) to 100 ft. (30.5 m) (Metcalf and Eddy, 1935; Steel, 1960). These underdrains were located in trenches below the bottom level of the filter bed. The underdrains usually specified were 2 ft (61.0 cm) vitrified clay pipe sections, 4 inches (10.2 cm) or more in diameter. The spigot end of the pipe was separated from the bell end of the next section by 1/4 - 3/8 inch (0.635 - 0.953 cm) to allow infiltration of the filtered water (ASCE-WPCF, 1959; Metcalf and Eddy, 1935; Steel, 1960). The lateral underdrains were 10 - 40 ft. (3.1 - 12.2 m) apart and drained into central underdrains 6 - 8 inches (15.2 - 20.3 cm) in diameter (ASCE-WPCF, 1959; Hansen, 1910; Holmes, 1945; Metcalf and Eddy, 1935; Steel, 1960; WPCF, 1959). Unlike the lateral underdrains, the main underdrains were cemented together at the joints. The underdrains had to be large enough to carry away a flow equal to the percolation through the sand filter (ASCE-WPCF, 1959). Metcalf and Eddy (1935) stated that concrete pipe should not be used for underdrains because concrete is subject to attack by acids formed in the sand beds.

These underdrains were surrounded by a layer of gravel approximately 1 foot (30.5 cm) deep. Most designs called for three grades of gravel each 3 - 5 inches (7.6 - 12.7 cm) in depth. Coarse gravel 1½ - 3 inches (3.8 - 7.6 cm) was placed next to the underdrains; gravel 3/4 - 1½ inches (1.9 - 3.8 cm) was placed in the middle layer; and fine gravel 1/4 - 1/2 inch (0.6 - 1.3 cm) was placed in the top layer directly below the bed of sand (ASCE-WPCF, 1959; Babbitt and Baumann, 1958; Fair et al., 1968; Hansen, 1910; Holmes, 1945; Metcalf and Eddy, 1935; Steel, 1960). The function of the gravel was to prevent sand from washing into the underdrains.

Once the underdrains and gravel were in place, the sand had to be placed. The sand had to be a clean well-graded sand free of deleterious material such as organics, dirt fines, clay, loam or cementing material (ASCE-WPCF, 1959; Babbitt and Baumann, 1958; Metcalf and Eddy, 1935; WPCF, 1961). An effective size sand between 0.2 mm (0.00787 inch) and 0.5 mm (0.0197 inch) with a uniformity coefficient of less than 5.0 was recommended (ASCE-WPCF, 1959; Hansen, 1910; Holmes, 1945; Salvato, 1972; Steel, 1960; WPCF, 1961). However, Salvato (1956) has reported that studies performed by Allen Hazen indicate as long as the effective size of the sands remain the same, a sand with a uniformity coefficient of 10 has almost identical hydraulic characteristics as a sand with a uniformity coefficient of 1.0. The additional cost incurred in finding or producing a low uniformity coefficient sand is not warranted.

Sand depths up to 60 inches (1.5 m) have been used in the past (Anonymous, 1912). Metcalf and Eddy (1935) stated that deeper beds hinder proper ventilation of the beds and may cause the bottom layers to turn anaerobic. More recent designs called for a sand depth of not less than 24 inches (0.61 m) and usually not more than 36 inches (0.91 m) (ASCE-WPCF, 1959; Hansen, 1910; Hardenburgh, 1921; Holmes, 1945; Metcalf and Eddy, 1935).

Experiments were undertaken at the University of Florida to determine the critical depth of sand filters by sampling various depths of a 30 inch (0.76 m) bed containing 0.35 mm (0.0143 inch) effective size sand (Furman et al., 1955). The results indicated that the 12 inch (30.5 cm) depth of sand was the critical depth. A five-day biochemical oxygen demand (hereafter referred to as BOD_5) removal of 89 percent was obtained at the 12 inch (30.5 cm) level, while a slight improvement of 95 percent removal of BOD_5 was obtained at the 30 inch (0.76 m) level. Furman et al. (1955) recommended that a 24 inch (0.61 m) sand depth be the normal minimum, but this may be reduced to 18 inches (45.7 cm) under special conditions.

The distribution of the sewage upon the filters is very important, as an even dose over a level bed is needed for proper distribution of the nutrients to the microorganisms. At the beginning of a filter run, uniform distribution is not obtained because the influent will penetrate into the sand near the distribution point. As the surface pores become partially clogged uniform distribution of the sewage improves (Metcalf and Eddy, 1935). For the above reasons Hansen (1910) stated that a maximum of 2,500 square feet (232.3 m^2) of filter area is all that should be served from one discharge point.

Some of the methods used for distribution were: 1) wooden or metal troughs running the full length of the bed; 2) a radiating trough network used in irregular shaped beds; 3) corner or quarter distribution points splashing onto concrete aprons; 4) central manholes surrounded by splash aprons; and 5) overhead perforated pipes either stationary or in the form of rotary distributors (ASCE-WPCF, 1959; Holmes, 1945; Metcalf and Eddy, 1935). These intermittent sand filter plants used manual labor (closing and opening of valves or sluice gates), automatic sump pumps in wet wells, or automatic rotating siphons to distribute the flow (Salvato, 1972). The automatic siphons were designed with a dosage tank to hold enough influent to dose the bed to a depth of 1 - 4 inches (2.5 - 10.2 cm) (Imhoff et al., 1973). Imhoff et al. (1973) and WPCF (1961) stated that the volume of sewage required to reach the desired depth should be distributed to the filter within a 5 to 20 minute period.

Several sources (Furman et al., 1955; Imhoff et al., 1973) mentioned the advantage of using two equal doses daily rather than one daily dose. Furman et al. (1955) at the University of Florida studied the effect of multiple loadings upon BOD removal. The daily loading was applied in two equal doses initially, and later the daily dosage was applied in 24 equal doses. The BOD removal rate increased only slightly by using more than two doses a day for the finer sized sands (0.25 mm; 0.00984 inch and 0.31 mm; 0.0122 inch), but a definite improvement in BOD removal was noted by using multiple doses for the larger sized sands (0.45 mm; 0.0177 inch and 1.04 mm; 0.041 inch).

Metcalf and Eddy (1935) reported that in some cases it has been found to be better to apply a double dosage of sewage on alternate days. The filter thus has a longer time to aerate between doses and may be in service longer. Steel (1960) suggested using beds in rotation. The first bed is dosed the first day, and the second day the second bed is dosed, while the first rests. With three beds, each works one day while resting two days. A fourth bed may be used to allow the other beds to rest for a week or more at a time.

The daily loading rate of sewage on the filters was heavily influenced by the effective size of the sand and the condition of the sewage. As a rule filter effluent quality slightly decreased as larger loading rates are used. Furman et al. (1955) and Grantham et al. (1949) demonstrated the effect of loading rates and sand size upon effluent quality. Table 2 shows the effects of hydraulic loading rates and effective sand size upon BOD removal. As the size of the sand is increased, the BOD removal decreased, but as the loading rates increased, little or no decrease was exhibited in BOD_5 removal.

Raw sewage has been applied to intermittent sand filters at the rate of 30,000 gpad (280.6 m^3/h-d) for small effective size sands ($\leqslant$ 0.20 mm; $\leqslant$ 0.0078 inch) and up to 75,000 gpad (701.5 m^3/h-d) for large effective size sand ($\geqslant$ 0.45 mm; $\geqslant$ 0.018 inch). Primary settled sewage (septic tanks, Imhoff tanks, primary sedimentation) dosage rates have ranged from 40,000 gpad (374.2 m^3/h-d) for small effective size sands up to 200,000 gpad (1,870.8 m^3/h-d) for large effective size sands. Biological treated effluents have been applied at rates of 400,000 gpad (3,741.6 m^3/h-d) to 800,000 gpad (7,483.1 m^3/h-d) (ASCE-WPCF, 1959; Babbitt and Baumann, 1958; Fair et al., 1968; Furman et al., 1955).

Operation

The operation of intermittent sand filters for treating sewage was much like the operation of slow sand filters for treating water except for the intermittent operation. Daniels (1945) has stated that unless they are carefully and intelligently operated, intermittent sand filtration can be a nuisance and even suffer total failure.

Daniels (1945) noted that the term "intermittent" was often overlooked. Many intermittent sand filters were continuously operated, and this had a serious effect upon the bed. The sand filter needs a rest period between applications to keep the bed aerobic and functioning properly because the filtered substances must be mineralized or oxidized within the top layer of sand or the pores will rapidly clog (Fair et al., 1968). Steel (1960) stated that complete resting of the bed is needed if septic conditions are present in parts of the bed. The resting period should be at least one week and two to four weeks if the condition is serious.

An example of a well operated filter was one at the Lawrence Experiment Station (Massachusetts Board of Health, 1912). Sand has not been removed from the surface of this filter in 23 years of operation. It had a surface area of 1/200 acre (0.002 hectare). This (Massachusetts Board of Health, 1912) article stated that within this time 2,395,532 gallons (9,068.1 m^3) of sewage containing about 6,000 lbs (2,727.3 kg) of organic matter had been applied to the intermittent sand filter. This example should demonstrate the potential of intermittent sand filtration for upgrading wastewater treatment plants.

Even when intermittent sand filters were operated properly eventually the filters became plugged and cleaning was necessary. Plugging occurred when the daily dosage of sewage failed to percolate through the filter bed in a 24 hour period. For multiple loadings, a cleaning was necessary when the preceding dosage still covered the surface at the time of the next loading (Furman et al., 1955).

When cleaning was necessary, the bed was taken out of service and allowed to dry. The surface mat of strained solids would crack and curl up. This mat, composed or organics and sand, was then scraped off and wasted or washed to remove the organic portion. An economical number of rakings between scrapings was used to increase filter runs. The amount of sand surface removed depended upon the condition of the influent sewage as well as other external conditions. Usually only a 1/4 - 1/2 inch (0.6 - 1.2 cm) thickness of sand surface needed to be removed, but this was extended to 2 inches (5.1 cm) at times (Babbitt and Baumann, 1958). Cleaning and removing of the sand continued until the minimum depth of filter sand was reached. At this point the intermittent sand filter was thoroughly scraped and clean sand was added.

The winter operation of intermittent sand filters presented special maintenance and operational problems as the sand surface of the filters could not be allowed to freeze. Daniels (1945) discusses three methods of managing intermittent sand filters during the winter. The first method, called the Brockton method, involves furrowing and ridging the beds at the start of winter. When the ice sheets are formed,

Table 2. The effect of hydraulic loading rates and effective size sand upon BOD_5 removals (Furman et al., 1955).

Effective Sand Size (mm)	BOD_5 Removal, %			
	Hydraulic Loading Rates (gpad)			
	150,000 1403.1 m^3/h-d	200,000 1870.8 m^3/h-d	250,000 2338.5 m^3/h-d	300,000 2806.2 m^3/h-d
0.25 (0.0098 in.)	98.1	96.9	97.6	97.5
0.31 (0.0122 in.)	97.5	94.8	95.2	94.7
0.46 (0.0181 in.)	86.2	82.3	82.8	84.3
1.04 (0.0409 in.)	81.6	-	67.4	70.8

Note: All beds contained 30 inches (76.2 cm) of sand.

they would come to rest upon the ridges and eventually would break up. At the start of a cold spell the beds are loaded heavier to provide extra protection against the freezing of the sand surface.

The second method, called the Worcester method, is similar to the Brockton method except that during the last scraping of the filters in the fall the scrapings are heaped into piles. These piles serve as a support for the ice layer and also require much less cleaning and rearrangement when spring comes.

The third method of managing the sand bed is identical to the regular summertime operation. However, much care has to be taken to prevent the ice layer from settling upon the flat sand surface and solidly freezing the surface. If the incoming influent dosage is unable to thaw out the settled ice layer, the filter will be unusable until the spring thaw arrives (Metcalf and Eddy, 1935).

Although more expensive, intermittent sand filters could be covered by wooded planks or plastic covers during the winter to prevent freezing.

Effluent quality

Much of the performance data on intermittent sand filters has already been discussed in the design section.

Furman et al. (1955) reported 80 to 95 percent BOD_5 removals depending upon sand size (refer to Table 2). Other literature sources (ASCE-WPCF, 1959; Babbitt and Baumann, 1958; Imhoff et al., 1973; Salvato, 1972) reported BOD_5 removals from 90 to 95 percent and E-coli removals from 95 to 99 percent. Table 3 shows the performance of selected intermittent sand filters in Ohio loaded with settled sewage at loading rates below 200,000 gpad (1,870.8 m^3/h-d). BOD_5 removals up to 99 percent were obtained, and these sand filters also produced a well oxygenated effluent, from influent devoid in oxygen.

Suspended solids removal performance has not been documented as thoroughly as BOD_5 performance. However Furman et al. (1955) reported BOD_5 removals of 89 to 96 percent for pilot-scale filters at the University of Florida. Salvato (1972) reported BOD_5 removals of 90 to 98 percent, while Babbitt and Baumann (1958) reported BOD_5 removals of only 75 percent for intermittent sand filters.

Intermittent sand filters produce an effluent that is well into the nitrification stage of oxidation. Furman et al. (1955) and Grantham et al. (1949) believed that the five-day BOD test for sand filter effluents measures some nitrogenous BOD_5 as well as carbonaceous BOD_5 demand. Therefore, these University of Florida researchers used the percent oxidation of applied nitrogen as another means to measure the degree of stabilization afforded by intermittent sand filters. They concluded that deeper beds and smaller sands afforded more complete nitrification. The deeper beds (30 inches; 76.2 cm) of the smaller sized sands (0.31 mm; 0.0122 inch and 0.25 mm; 0.00984 inch) turned out effluents that were oxidized 96 and 98 percent, respectively, at a 75,000 gpad (701.5 m^3/h-d) loading rate. The percent oxidation decreased rapidly as the loading rate went up, until at the loading rate of 175,000 gpad (1,636.9 m^3/h-d), the percent oxidation of nitrogen was essentially independent of loading rate. Furman et al. (1955) demonstrated that more complete oxidation of nitrogen is obtained when the sewage is applied in multiple increments. In most cases the total effluent concentration of nitrogen was less than the influent concentration of nitrogen, and Furman et al. (1955) attributed this to either nitrogen loss to the atmosphere or nitrogen build-up in the bed.

Pincince and McKee (1968) determined that the amount of nitrogen oxidized depended upon the aerobic conditions in the top portion of the sand bed. They developed a hypothesis that the nitrate concentration was constant for the entire depth of the bed at the instant that infiltration was complete. Oxygen enters into the top layer of the bed as time passes and oxidizes nitrites to nitrates. When the next dosage of sewage is applied the nitrates formed in the upper layers are forced out of the filter by incoming water. After infiltration was complete the cycle started over.

Table 3. Performance of intermittent sand filters in Ohio (Holmes, 1945, p. 285).

Parameter	Parameter Concentration (mg/l)				
	Plant Number				
	#1	#2	#3	#4	#5
Settleable Solids:					
Applied	2.0	8	4	5	3.0
Final	T	0	T	0	T
Dissolved Oxygen:					
Applied	0.0	0.0	1.9	0.0	0.0
Final	7.9	2.8	8.1	6.5	7.5
BOD_5:					
Applied	425	666	319	555	326
Final	6.7	26	4	18	2

T = Trace

Ecology of intermittent sand filters

Intermittent sand filtration of sewage is a biological as well as mechanical mechanism. Early researchers at the Lawrence Experiment Station realized the importance of a bacteria population within intermittent sand filters (Powell, 1911; Calaway, 1957). The biological process of metazoa and bacteria attacking and decomposing the organic matter in the "schumtzdecke" (filtering skin) was essential for proper intermittent sand filter operation (Calaway, 1957; Metcalf and Eddy, 1935). Calaway et al. (1952) also determined that the degree of BOD_5 removal by intermittent sand filtration was higher than might be expected by mechanical filtration alone. Heterotrophic bacteria were attacking the soluble BOD_5 as the sewage passed through the filters.

Calaway et al. (1957) found principally six groups of bacteria present in intermittent sand filters. They were: 1) Zoogloeal bacteria such as *Zoogloea ramigera;* 2) Aerobic spore formers such as *Bacillus cereus;* 3) Alkali-producing bacteria such as *Alcaligenes faecalis;* 4) Yellow bacteria such as *Flavobacterium;* 5) Soil actinomycetes such as *Nocardia* sp.; and 6) *Streptomyces.* The zoogloea bacteria and two heterotrophic bacteria, *Bacillus* and *Flavobacterium,* were the largest in number. The floc-forming zooglea bacteria are well known for their importance in biological treatment of primary sewage and were found in great abundance in the upper 12 inches (30.5 cm) of the filter. However they were not present below the 12 inch (30.5 cm) level. *Bacillus* and *Flavobacterium* have the ability to decompose organic nitrogen compounds and to destroy carbohydrates. *Flavobacterium* dominated over *Bacillus* at high loading rates (300,000 gpad; 2,806.2 m^3/h-d) but the domination was often reversed at low hydraulic loading rates. In all cases as the loading rate was increased the total bacteria population also increased. Coliforms were found in the heaviest concentration at the surface and decreased rapidly with depth. Calaway et al. (1952) noted that there was essentially no removal of *Fecal Streptococcus* by intermittent sand filters.

Several protozoa, including the ciliates *Colopoda* sp. and *Paramecium* sp., were present at times in the filter, but while the protozoa have the ability to eat bacteria and disintegrate zoogloeal masses, they were never found in large enough numbers to have an influence.

Metazoa perform an important function in intermittent sand filters because they feed on the digest pore clogging material such as zoogloeal slims, cellulose, humus, and other materials. This keeps the bed open and accessible to oxygen because the metazoa excretions are very porous and are easily decomposed by bacteria (Calaway, 1957). Annelid worms, flat worms, nematodes, rotifers, water mites, insects, and insect larvae were all found in the filtering layer by Calaway et al. (1952).

It is quite apparent that the mechanical mechanisms of filtering and adsorption do the majority of the work in sewage purification, but the presence of a well-established stable biological population makes the intermittent sand filter even more effective.

Economics

The Engineering News Record (1975) Cost Indices were used to update reported costs to December 1975 values. The updated values are shown in parentheses following the original cost. The construction costs for intermittent sand filters basically depends upon the cost of the land and the cost of sand of proper specifications.

Powell (1911) and Fuller (1911) reported a construction cost of $3,260 ($47,113.94) per acre in 1903 in Massachusetts. A construction cost of $6,350 ($90,805) per acre was reported by the Baltimore Sewerage Commission in 1906, while Metcalf and Eddy reported a construction cost of $8,850 ($64,638.31) per acre in 1924 at Framingham, Massachusetts. The construction costs of intermittent sand filters at today's prices would range approximately from $47,000 to $91,000 per acre.

Operating costs of $7.75 ($196.04) to $13.82 ($349.59) per million gallons of filtrate have been reported in 1912 at Worcester, Massachusetts. Metcalf and Eddy (1935) reported operating costs varying from $10 ($117.44) to $40 ($469.78) per million gallons of filtrate in 1935. Updated to 1975 values, these operating costs for intermittent sand filters would range approximately from $115 to $470 per million gallons of filtered sewage.

Rapid Sand Filtration and Other Media

Rapid sand filters used for sewage treatment

Streander (1940a,b,c) reported a study of the use of rapid sand filtration to treat settled sewage at Wuppertal, Germany, that indicated that deeper beds gave more uniform results, and that removals of 60 to 90 percent of suspended solids were common with rapid sand filters. He also stated that sewage solids trapped in the filters tended to decompose resulting in bacterial slimes which clogged the filters. Backwashing could remove only 80 to 90 percent of the trapped sewage solids, and the filters could not efficiently handle peak loads.

Lynam et al. (1969) Tebbutt (1971), and Tchobanoglous (1970) used rapid sand filtration as a method of tertiary treatment to upgrade activated sludge plants. Tebbutt (1971) reported suspended solids removals ranging from 38 to 70 percent, and that a fine sand or dual-media bed offered no improvement in suspended solids removal over an anthracite bed. Tchobanoglous (1970) reported that the removal efficiency was primarily a function of the media size, and that turbidity break-throughs using rapid sand filtration was not observed using an 8 - 10 foot (2.44 - 2.88 m) headloss. He also determined that polyelectrolites can be used to achieve varying degrees of removals of suspended solids from secondary effluents.

Streander (1940b) also described a rapid sand filter mechanical sand scraper and washer which may be of interest for intermittent sand filter operations. The common hydraulic backwash method was not in universal use at this time due to surface cleaning problems, and Streander reported on the Laughlin method. The Laughlin method used two separate cleaning units. Laughlin concluded that a majority of the solids were trapped in the upper part of the bed. Therefore the top unit consisted of a cutting blade mounted in a moving chamber. The blade cut into the sand surface 1/2 inch (1.3 cm) and rotating paddles stirred the scrapings to agitate and loosen the sewage solids from the sand. The sand fell back onto the surface while the organic debris was withdrawn by suction pumps. The rest of the bed of sand was then cleaned using the common hydraulic backwash method.

Rapid sand filters and other media used for algae removal

In the past some work has been done on the use of rapid sand filtration to remove algal suspensions. Ives (1961) used two green algae, *Chlorella* sp. and *Scenedesmus* sp., to demonstrate the usefulness of his recently developed filtering equations for describing the performance of pressurized rapid sand filters. He found a rapid buildup of filtered algae at the surface and a near constant algae deposit at a depth of 10 - 12 cm (3.9 - 4.7 inches) and below.

Bochardt and O'Melia (1961) conducted a series of experiments similar to Ives. They found that algae were present at all times in the sand filter effluent even though at times they could be detected only by microscopic examination.

Folkman and Wachs (1970) were concerned with the problem of possible algal contamination of aquifers in Israel, caused by the infiltration of lagoon wastewater through dune sand. Their experimental findings agreed with the findings of Ives (1961) and Bochardt and O'Melia (1961). Accumulation of algae in the upper 5 cm (1.97 inch) of sand caused headloss to increase. Below this point some algae did accumulate but this accumulation did not affect the hydraulic characteristics of the sand.

Intermittent Sand Filtration to Upgrade Lagoon Effluents

General

Intermittent sand filtration to treat wastewater fell into disuse mainly because of the large amounts of land required. However in 1971, PL 92-500 set new standards for wastewater treatment plant effluents. In order to meet these future standards, the development of a simple, economical, low maintenance treatment method to "polish" or "upgrade" lagoon effluents was needed. Many of the small communities using wastewater stabilization ponds are still surrounded by large areas of relatively inexpensive land and have locally available filter media. This need generated two studies at Utah State University to determine the feasibility of using single stage intermittent sand filters to upgrade lagoon effluents. The first study dealt with laboratory scale and pilot scale single stage intermittent sand filters, while the second considered single stage prototype intermittent sand filters.

Single stage pilot scale and laboratory scale intermittent sand filters

Marshall and Middlebrooks (1974) evaluated the performance of laboratory and pilot scale intermittent sand filters to determine if the process was capable of upgrading existing wastewater treatment lagoons in the State of Utah to meet Class "C" water quality standards (see Table 1). The laboratory scale sand filters were 6 inch (15.2 cm) plexiglass columns, while the pilot scale intermittent sand filters were 4 ft. x 4 ft. (1.2 x 1.2 m) plywood boxes. Each contained 30 inches (76.2 cm) of sand. Effective size of the sand, hydraulic loading rate, and algal concentration were the variables studied.

During the study *Chlamydomonas* sp. (a green alga) was the predominant alga in the influent to the filters. *Chlamydomonas* sp. represented a minimum of 70 percent of the algal population throughout the study.

Hydraulic loading rates were found to have little effect upon the parameters studied in the laboratory portion of the study. They did not affect the algae or suspended solids removal efficiency at the 100,000 (935.4 m^3/h-d), 200,000 (1,870.8

m³/h-d), or 300,000 (2,806.2 m³/h-d) gallons per acre-day loadings employed in the laboratory study. However, during the field experiments, when higher loading rates were used, suspended and volatile solids removal appeared to increase with an increase in hydraulic loading rates (Tables 5 and 6). Algal cells were found to pass the entire bed depth although significant amounts of algae were removed by the filters.

Sand size had a profound effect on the quality of the effluent, and also on the time of operation before plugging occurred. Tables 4, 5, and 6 show the monthly BOD_5, suspended solids, and volatile suspended solids performance respectively of the field filters. For all three parameters, the 0.17 mm (0.0067 inch) effective size sand filters produced a better quality effluent than the 0.72 mm (0.0283 inch) effective size sand filters. The monthly BOD_5 removal efficiencies for the 0.17 mm (0.00669 inch) size sand ranged from 93.8 percent for a 400,000 gpad (3,741.6 m³/h-d) hydraulic loading rate to 66.7 percent for a 800,000 gpad (7,483.1 m³/h-d) hydraulic loading rate. The removal efficiencies of the 0.72 mm (0.0283 inch) effective size sand filters were slightly lower. In all cases the 0.17 mm (0.0067 inch) effective size sand filters gave an effluent BOD_5 concentration of less than 5 mg/l. The volatile suspended solids removal performance was similar although it fluctuated greatly.

The effects of hydraulic loading rate on suspended solids removal in the field filters was inconclusive because of the large quantities of fines washed from the filters (Table 5). Immediately before plugging occurred in the laboratory filters, the filter effluent suspended solids concentrations were approximately zero. As the filter operated with time, the suspended solids removal efficiency increased reaching a maximum point at the time of plugging. This phenomena did not occur in the field, but if fines had been washed from the filter before placing it in operation, it is likely that a similar pattern would have occurred.

Table 4. Mean influent and effluent BOD_5 concentrations obtained with each sand size and hydraulic loading rate for the field filters (Middlebrooks and Marshall, 1974, p. 54).

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 5. Mean influent and effluent suspended solids concentrations obtained with each sand size and hydraulic loading rate for the field filters (Middlebrooks and Marshall, 1974, p. 56).

	Mean Monthly Influent 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

At hydraulic loading rates of 0.4 to 0.6 mgad (3,741.6 m^3/h-d - 5,612.4 m^3/h-d), the 0.17 mm (0.0067 inch) effective size sand filters were found to operate approximately 100 days before cleaning was required when receiving a lagoon effluent containing a mean suspended solids concentration of 20 mg/l. At hydraulic loading rates of 0.7 mgad (6,547.7 m^3/h-d) and 0.8 mgad (7,483.1 m^3/h-d), the 0.17 mm (0.0067 inch) filters operated 32 consecutive days before requiring cleaning when receiving lagoon effluent containing a mean suspended solids concentration of 42 mg/l.

Based upon current cost figures in 1972, an effluent polishing intermittent sand filter process could have been constructed and operated for a cost ranging between $15 to $47 per million gallons of filtrate. 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 (Marshall and Middlebrooks, 1974).

Single stage prototype intermittent sand filters

Reynolds et al. (1974) evaluated the feasibility of using prototype 0.17 mm (0.00669 inch) intermittent sand filters to upgrade lagoon effluents. Their experimental facility consisted of six 24 x 36 feet (7.3 m x 11.0 m) intermittent sand filters. Harris et al. (1975) presented a review of one year of data on the filters.

The effluent BOD_5 quality was below 5 mg/l 93 percent of the time at an influent average of 19 mg/l. During the winter one filter was constantly flooded, causing the filter to become anaerobic. This greatly reduced the filter's efficiency (effluent exceeded 5 mg/l 92 percent of the time).

The influent suspended solids averaged 31 mg/l during the year. After start-up of the filter operation, the effluent was relatively high in suspended solids. This was primarily due to the "washing" of inorganic fines from the sand and the filter rock during the start-up period. After this period of washing the filter effluents exceeded 5 mg/l suspended solids only 15 percent of the time. The flooded aerobic filter exceeded 5 mg/l 83 percent of the time (Reynolds et al., 1974).

The influent volatile suspended solids average 23.8 mg/l for the year, while the effluent averages were below 3.4 mg/l. The anaerobic filter however, exceeded 5 mg/l 67 percent of the time (Reynolds et al., 1974).

The results indicated that only a slight amount of phosphorus was removed by the filters. This may have been due to adsorption on the sand particles.

Nitrification occurred within the filters. The effluent ammonia concentration was significantly below the influent concentration, while the nitrate concentration of the effluent was well above the influent nitrate concentration. Nitrification decreased during the winter months when bacterial action slowed down. Also the anaerobic filter showed no nitrification tendencies.

The length of the filter runs varied inversely with the hydraulic loading rates. The average filter runs varied from 8.1 days for a 1.0 mgad (9,353.9 m^3/h-d) hydraulic loading rate to 64 days for a 0.2 mgad (1,870.8 m^3/h-d) hydraulic loading rate. The

Table 6. Mean influent and effluent volatile suspended solids concentrations obtained with each sand size and hydraulic loading rate for the field filters (Middlebrooks and Marshall, 1974, p. 56).

Month	Mean Monthly Influent Conc. as Volatile Suspended Solids (mg/l)	Mean Monthly Effluent Volatile Suspended Solids Concentration 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	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

length of the individual filter runs varied greatly due to two mechanisms: 1) time of day at which the filters were loaded and 2) calcium carbonate precipitation. Reynolds et al. (1974) performed an experiment which showed that when the filters were loaded in the morning, they had influent standing on them throughout the daylight hours. The liquid above the filter experienced tremendous algal growth. The suspended solids concentration had increased from 77.1 mg/l at one hour after loading to 224.4 mg/l at twelve hours after loading. They suggested loading the filters in the evening or covering the filters to prevent photosynthesis.

The second mechanism dealt with the increase in the pH of the water as the algae grow and utilize carbon dioxide in the standing water. As the pH of the wastewater increased, the carbonate-ion concentration eventually exceeded its solubility product and calcium carbonate precipitated out. This calcium carbonate bonded with the sand grains to form a "plaster like" surface which rapidly became impermeable (Harris et al., 1975).

During the winter the beds were rearranged, using different techniques, to prevent the freezing of the sand surface. As described by Harris et al. (1975) all methods proved satisfactory.

Reynolds et al. (1974) concluded that intermittent sand filters could be constructed and operated for \$56 to \$62 per million gallons of filtrate. With federal assistance with the construction costs, the figure drops to \$31 to \$33 per million gallons of filtrate.

Filtering and Clogging Mechanisms of Intermittent Sand Filtration

General

The performance of intermittent sand filtration in the filtering of sewage is well documented. As an intermittent sand filter operates most of the suspended solids that enter the sand will be oxidized (Steel, 1960). However some of the solids are transformed into humus and other material forming an organic mat on the surface. This surface mat is beneficial in removing fine suspended solids from the sewage. As the length of the filter run increases, the rate of removal increases until the filter becomes clogged and the sand below the surface algal mat turns anaerobic. At this point the filter has to be scraped or reconditioned.

The micro-scale aspects of the filtering and clogging mechanisms of intermittent sand filtration will now be presented. However, since the literature contains no information on the micro-scale filtering and clogging mechanisms of intermittent sand filtration, information is borrowed from other sources. The available information should hold true for intermittent sand filters.

Filtering mechanisms

Folkman and Wachs (1970) quoting Ives (1961) describe filtration as a process of three phases: 1) a ripening period where filter efficiency increases; 2) a period of maximum efficiency; and 3) a period of clogging and deterioration in filter efficiency determined by the available pressure head. This filtration process may be true for pressurized rapid sand filters, but slow and intermittent sand filters would probably be considered plugged before deterioration in effluent quality would occur. Unless a large head was used, intermittent sand filters would probably not include a period of deterioration in efficiency during period 3.

Ives (1961), while developing his filter equations, assumed the following three principles of filter behavior: 1) the removal of suspended particles from water is proportional to the quantity of suspended particles in the water; 2) the removal characteristic of the filter depends on the surface available on the filter grains, on the tortuosity of flow within the pores, and the interstitial velocity; 3) the principle force operating to remove suspended particles from the flow stream lines is gravitational, although adsorptive forces become dominant when a particle reaches a grain surface.

McGauhey and Krone (1967), describing soil percolation fields, proposed that particulate matter is removed in the upper 12.5 - 15 cm (4.9 - 5.9 inch) by one or more of the following mechanisms: a) straining at the soil surface where the filtered particles buildup on the sand and become part of the filter; b) bridging when the particles penetrate the sand surface until they reach a pore opening that stops their passage; and c) straining and sedimentation which includes "a" and "b" except that the suspended particles are finer than half of the smallest pore openings.

Table 7 contains a summarization of the most prominent removal mechanisms proposed for rapid sand filters by Tchobanoglous (1970). The first four removal mechanisms are classified as physical mechanisms and are related to various physical parameters such as grain size, porosity, and bed depth. Mechanisms 5 through 8 are chemical mechanisms. Tchobanoglous (1970) proposed that all three basic removal mechanisms (physical, chemical, and biological) occur in the rapid sand filtration of sewage effluents. These three mechanisms should also be present in intermittent sand filtration of sewage lagoon effluents.

Clogging mechanisms

The information on the clogging of intermittent sand filters has been obtained from studies on soil filtration of wastewater and septic tank percolation studies. In most of these studies the experimental setup was almost identical to an experimental intermittent sand filter.

DeVries (1972), using columns containing 0.1 mm - 0.5 mm (0.0039 - 0.020 inch) effective size sand, demonstrated that clogging by wastewater effluent was strictly a surface phenomena. Hydraulic conductivity measurements (coefficient of permeability measurements) showed that clogging took place in the surface pores. DeVries (1972) also showed through oxygen and carbon dioxide measurements that failed filters (plugged) recovered in only 8 days of room temperature.

Thomas et al. (1966) used a round lysimeter with 0.2 mm (0.00787 inch) effective size sand at a hydraulic loading rate of 5 gallons per square foot per day (0.218 mgad; 2,037.29 m^3/h-d). Thomas et al. (1966) showed that clogging occurred in three phases: 1) a gradual reduction in filtration rate; 2) a short period characterized by a rapid decline in filtration rate; and 3) a point where the infiltration rate approached a lower limit. He showed that clogging was essentially a surface phenomena occurring in the top 1 cm (0.3937 inch), but did occur on down to 6 cm (2.4 inches). Thomas et al. (1966) also believed that the amount of organic matter was the significant contributor to clogging. He also reported that phosphate and iron could have contributed to the clogging problem.

Jones and Taylor (1965) used a coarse sand for treating septic tank effluent. They used the Darcy equation to demonstrate the plugging phenomena.

$$V = -'Ki \quad \ldots\ldots\ldots\ldots \quad (1)$$

in which

V = macroscopic flow velocity (discharge velocity)

'K = hydraulic conductivity (coefficient of permeability)

i = hydraulic head gradient

Jones and Taylor described three phases of plugging as follows: 1) an initial period where conductivity declines to 25 percent of the initial value; 2) a second phase where conductivity declines slowly to 10 percent of the original value; and 3) a sharp drop to a conductivity of 1 to 2 percent of the original value. They also showed that the hydraulic loading rate affected the duration of periods 1 and 2 but not period 3. Under an anaerobic environment period 2 was missing. Jones and Taylor (1965) reported that soil clogging occurred 3 to 10 times faster under an anaerobic environment than under an aerobic environment.

Laak (1970) reported using 6 inch (15.2 cm) columns containing garden soil, sandy loam soil, and 0.26 mm (0.01 inch) effective size sand to filter aerobic and anaerobic unit process effluents. He reported that the clogging material between the sand grains consisted of 90 percent or more of bacteria. Laak (1970) also reported that the length of the filter runs was inversely related to the sum of the suspended solids and BOD_5 in the wastewater. He also noted that the hydraulic loading rate is critical. At certain hydraulic loading rates, a 10 percent increase in hydraulic loading rate could reduce the soil system service time (length of filter runs) by 50 percent or more.

Mitchell and Nevo (1964) concluded that there was a high correlation between the accumulation of

Table 7. Sand filtration removal mechanisms (Tchobanoglous, 1970, p. 605).

Variables	Removal Mechanism
1. Filter media grain size, shape, and density	1. Straining: a) Mechanical, b) Chance Contact
2. Filter media porosity	2. Sedimentation
3. Media headloss characteristics	3. Inertial impaction
4. Filter bed depth	4. Interception
5. Filtration rate	5. Chemical adsorption: a) Bonding, b) Chemical interaction
6. Allowable headloss	6. Physical adsorption: a) Electrostatic forces, b) Electrokinetic forces, c) Van der Waals forces
7. Effluent characteristics	7. Adhesion and adhesion forces
8. Chemical treatment	8. Coagulation-flocculation
9. Floc strength	9. Biological growth
10. Filter bed charge	
11. Fluid characteristics	

polysaccharides, with or without glucionic residuals, and soil clogging. The polysaccharide producing bacteria, *Flavobacterium* sp., was found in large numbers in the clogged sand. They also stated that soil clogging was related to a low-oxidation potential in the soil or sand. They suggested that the use of biological oxidizing agents might be used to increase filtering capability.

Aunimelech and Nevo (1964) stated that high carbon to nitrogen ratios induced long-lasting clogging in soil while low carbon to nitrogen ratios caused only short periods of clogging. Aunimelech and Nevo (1964) also stated that slowly decomposible material caused only a slight clogging. This statement seems very inconsistent with the need for resting of septic tank fields as well as the need for scraping of intermittent sand filters.

Harris et al. (1975) concluded that calcium carbonate precipitation, caused by excessive algal growth and the resulting high pH, caused rapid clogging of intermittent sand filters used to upgrade wastewater lagoon effluents by laying down a thin "plaster like" film on the filter. This phenomena took place only during the summer when sunlight was abundant.

In conclusion, it should be stated that additional studies on the mechanisms of filtering and plugging of intermittent sand filters used to "upgrade" wastewater lagoon effluents should be performed in order to verify and update the information in this section.

Mathematical Modeling of Intermittent Sand Filtration

General

Unfortunately because intermittent sand filtration has just been brought back into focus, very little mathematical modeling has been done on intermittent sand filters. Only one attempt at developing theoretical equations for the performance of intermittent sand filters has been documented (Pincince and McKee, 1968). However many models have been proposed for rapid sand filter performance in the removal of algae and suspended matter from water supplies. Ives and Sholji (1965) have indicated that their rapid sand filter equations do not apply to situations where a biologically active "schmutzdecke" (filtering skin) is present. However it still may be possible to utilize the mathematical models of rapid sand filtration for intermittent sand filtration with modifications for biological action. In fact the original theoretical filtering equations were derived by Iwasaki (1937) for slow sand filters used for filtering drinking water. In this section of the literature review rapid sand filtration models will be reviewed first. A review of slow sand filtration and intermittent sand filtration will follow.

Rapid sand filtration models

Most of the mathematical filter equations are of highly empirical simplifications, and involve a process of trial and error. Only two rapid sand filtration models are advanced enough to warrant a "... rational although somewhat empirical application of the basic principles of filtration" (Ott and Bogan, 1970, p. 455). One model is by Ives and Sholji (1965) and the other by Camp (1964). First, however, the original model as proposed by Iwasaki (1937) will be discussed as both the Ives (1961, 1965) and Camp (1964) models originated from Iwasaki's (1937) equations.

Iwasaki model. Iwasaki (1937) developed a model based upon a detailed microscopic examination of the penetration and distribution of microorganisms and particulate matter in a mixed nonuniform size sand. He proposed the following mathematical relationship:

$$\frac{\partial I}{\partial L} = -\lambda I \qquad \text{. . . . . (2)}$$

in which

I = quantity of microscopic material reaching a 1 sq. cm area at a depth L in a unit of time, $L^{-2}\,T^{-1}$

L = depth within the bed of sand, L

λ = impediment modulus or filter coefficient, L^{-1}

Iwasaki (1937) also proposed the following filter equation:

$$\frac{\partial I}{\partial L} + \frac{\partial S}{\partial t} = 0 \qquad \text{. . . . . (3)}$$

in which

S = quantity of microscopic material arrested in 1 cubic centimeter of sand at depth L in a unit of time, L^{-3}

t = time of filtration, T

Because the filter coefficient λ will change with time Iwasaki (1937) proposed an equation to facilitate this change as follows:

$$\lambda = \lambda_o + cS \qquad \text{. . . . . (4)}$$

in which

λ_o = initial filter coefficient, L^{-1}

c = constant

Ives model. Ives (1961) began the first serious attempt at developing rapid sand filter equations using Iwasaki's basic equations with some modifications. Ives (1961) modified Equation 2 by expressing the dependent variable in terms of volumetric concentrations.

$$\frac{\partial C}{\partial L} = -\lambda C \quad \ldots\ldots\ldots\ldots\ldots (5)$$

in which

C	=	concentration of suspended particles at depth L, expressed in terms of volume of particles per volume of water
L	=	depth within filter, L
λ	=	impediment modulus of the filter coefficient, L^{-1}

Ives (1961) as summarized by Ott and Bogan (1970) modified Iwasaki's second equation to the following:

$$\frac{\partial C}{\partial L} = \frac{1 - fG}{v}\frac{\partial G}{\partial T} \quad \ldots\ldots\ldots\ldots (6)$$

in which

G	=	specific deposit or volume of deposit per volume of filter bed
T	=	time, T
v	=	approach velocity, LT^{-1}
fG	=	self-porosity of the deposited solids

This equation assumes that the volume of the particles does not change significantly with time as the particles pass from the influent water above the bed into the bed to be deposited on the sand grains. Therefore the volume of particles removed from the water is equal to the volume deposited in the filter bed. This statement would probably not be true for intermittent sand filtration where filter runs are longer, and there is much decomposible matter present in the influent. The fG (self-porosity) term was included to indicate the effect the deposited solids have on removal.

Ives (1961) also developed a headloss equation as follows:

$$\frac{\partial H}{\partial L} = \left(\frac{dH}{dL}\right)_o + kG \quad \ldots\ldots\ldots\ldots (7)$$

in which

H	=	headloss, L
$(dH/dL)_o$	=	initial headloss per unit depth
k	=	headloss constant

Many researchers (Camp, 1964; Deb, 1969; Fox and Cleasby, 1966; Ives and Sholji, 1965; and others) have tried to develop equations which would better define or predict values for the filter coefficient λ.

Ott and Bogan (1970) gave a critical review of the present methods of finding the filter coefficient λ by theoretical equations. Their suggestion was to find the value of the filter coefficient by experimental methods using the following equation:

$$\lambda = -\frac{1}{\Delta L}\ln\left(\frac{C_n}{C_{n-1}}\right) \quad \ldots\ldots\ldots\ldots (8)$$

in which

n	=	bottom of layer under consideration
n-1	=	top of layer under consideration
λL	=	depth of filter media

They suggested that C_n and C_{n-1} can be found by using counting techniques such as the Coulter counter. Ott and Bogan (1970) came to the conclusion that the initial bed porosity exerted a tremendous influence upon the filter coefficient.

Camp model. Camp's model consists of the basic Ives equation (Equation 5) and a modified form of the Kozeny equation for headloss. Two equations for the hydraulic gradient have been developed by Camp (1964). As summarized by Ott and Bogan (1970), the first equation applies to the situation where the clogging of the filters is due to the formation of sheaths on the sand grains, and the second equation applies to the situation where removal is due to a straining mechanism. For sheath formation:

$$i = \frac{K(1 - P_o + G)^2}{(P_o - G)^3 (D + \Delta D)^2} \quad \ldots\ldots (9)$$

For straining:

$$i = \frac{K(1 - P_o)^2}{D_o^2 P_o^2 (P_o - G)} \quad \ldots\ldots (10)$$

in which

i	=	hydraulic gradient
K	=	overall characteristic of the filter bed
D	=	grain size at the start of filtration, L
ΔD	=	change in grain size at a particular time, L
P_o	=	initial porosity

Slow sand filtration models

Iwasaki (1937) developed the original filtration equations for a slow sand filter at Tokyo, but essentially no further work has been performed on these equations for slow or intermittent sand filtration.

Folkman and Wachs (1970) in Israel were presented with a problem of possible infiltration of algae into the groundwater from waste stabilization ponds. The filter medium was dune sand, and they developed a model to predict the possibility of groundwater contamination. From the results of their experiments they determined that the concentration of algae at a depth just below the sand surface was greater than the initial concentration of algae. They added a coefficient to the solved Iwasaki (1937) equation as modified by Ives (1961).

$$C = B\,C_o\,e^{-\lambda L} \qquad \ldots\ldots\ldots\ldots (11)$$

in which

B = initial increase in algal concentration

Folkman and Wachs (1970) also developed a headloss equation for slow sand filtration

$$H = iL + H_T \qquad \ldots\ldots\ldots\ldots (12)$$

in which

H_T = headloss in the top 5 cm (1.96 inch) of dune sand, L

Intermittent sand filtration models

Pincince and McKee (1968) used a completely different approach to model the physical-chemical and biological processes in intermittent sand filters. This model assumed that "... aerobic bacterial activity within the sand or soil material is limited, mostly by oxygen deficiency, while the surface is ponded with wastewater" (Pincince and McKee, 1968, p. 1117). The model assumes that atmospheric oxygen diffuses into the sand after infiltration ends, but the penetration is limited by anaerobic zones in the lower portion of the filters. Pincince and McKee (1968) assumed that the oxygen content of the system can be described by a quasi-steady-state diffusion equation including an oxygen sink term.

$$-\frac{d}{dz}\left(c\,D\,\frac{dX_A}{dz}\right) = R_A \qquad \ldots\ldots\ldots (13)$$

in which

c = total molar concentration, mols/L^3
D = effective diffusivity, L^2/t
R_A = molar rate of production of gas A(oxygen), mols/tL^3
X_A = mole fraction of gas A(oxygen)
z = soil depth, L

Equation 17 can be integrated once to yield:

$$D_{(z)} = \frac{\int_z^L R_A(\xi)\,d\xi}{c\,\dfrac{dX_A}{dz}} \qquad \ldots\ldots (14)$$

in which

$D_{(z)}$ = effective diffusivity at depth z, L^2/t
ξ = a dummy variable
L = the depth at which the oxygen concentration is zero

Pincince and McKee (1968) state that if diffusivity does not change with time their model should be satisfactory. The values of D and R_A depend upon the amount of bacterial growth in the filter which in turn depends upon such things as the composition and strength of wastewater, depth and frequency of ponding, and soil type. Presently values of D and R_A are unavailable for intermittent sand filters for design purposes.

Oxygen profiles can be calculated by the following:

$$X_A = X_{A_o} + \frac{1}{c}\int_o^z \frac{\int_z^L R_Z(\xi)\,d\xi}{D(\xi)}\,d\xi \quad \ldots (15)$$

in which

X_{A_o} = mole fraction of gas A(oxygen) at surface

Pincince and McKee (1968) using soil moisture analysis from field and laboratory filters substantiated their model. Their results cannot really be used for intermittent sand filter design, but they can be used to improve the operation of an existing operation. They said that the frequency of application should be adjusted so that the influent is completely nitrified, therefore affording complete aerobic treatment for the wastewater.

Filterability index

Hsiung (1972) determined that the filtration process consists of two interacting processes involving: 1) the influent suspension, and 2) the filter media. Hsiung (1972) attempted to develop a filterability index that would relate the filterability of secondary effluents from activated sludge and trickling filter plants to their filterability through granular medias.

Hsiung (1972) reasoned that the filterability of a suspension could be determined by filtering a suspension through a membrane filter twice. The average refiltration rate of the suspension is related to the cake porosity and the specific area of particles in the filter cake which was filtered out from the first filtration. He determined that the filterability of a suspension could be described by two parameters, E and R. He came up with the following equations:

$$VC_o = R - \frac{R}{G}\left(\frac{V}{t}\right)^{1/3} \quad \ldots\ldots\ldots (16)$$

in which

V = sample volume, L^3
C_o = initial suspended solids concentration, M/L^3
R = parameter, M
G = parameter, $(L^3/T)^{1/3}$
t = refiltration time, T

The terms R and G are found by plotting VC_o versus $(V/t)^{1/3}$ and finding the intercepts. Once R and G are found E can be determined from Equation 17.

$$E = (R^{1/3}\,G)^{1/2} \quad \ldots\ldots\ldots\ldots (17)$$

E is a dimensionless parameter which describes the removal efficiency expected for a suspension, since E varies inversely with the surface area of the particles in suspension and directly with the particle sizes. Therefore E should vary directly with removal efficiency.

Hsiung (1972) related the effect of the suspension on the headloss produced in the filter by the parameter R. R was found to be related to the solids loading factor upon the granular filter.

By finding the values of E and R through membrane filtration, the filterability of a secondary effluent suspension through sand filters could be anticipated. The removal efficiency and headloss characteristics of the influent suspension, as estimated by E and R, could be used to design and specify filter media which would meet the effluent quality required at certain headlosses.

Because the use of intermittent sand filtration to upgrade lagoon effluents is a relatively new concept, no modeling techniques have been tried. However, it is hoped that the review of this section will give a better understanding of the theoretical considerations of sand filtration. More study is needed to develop modeling equations which could be used to help in the design of intermittent sand filters.

Literature Review Summary

The use of slow sand filtration to upgrade culinary water began over a century and a half ago. In 1870 the intermittent sand filtration process was developed to treat communities' wastewaters. Both of these sand filtration processes were heavily used by municipalities seeking an economical method of treating culinary water and wastewater. However, their use declined due to the unavailability of inexpensive land. In 1972, new effluent standards initiated studies on the use of intermittent sand filtration to upgrade wastewater lagoon effluents. Many of these wastewater lagoons were located in areas where inexpensive land was still readily available. The use of intermittent sand filtration was shown to be a feasible and economic method of upgrading wastewater lagoo effluents. Presently series intermittent sand filtr. on is being studied as an effluent polishing process.

Much of the information contained in this lengthy and detailed literature review is not directly applicable to intermittent sand filters. However, if one is to thoroughly understand the process known as intermittent sand filtration, all avenues of possible information must be explored.

METHODS AND PROCEDURES

Experimental Design

General

The study was divided into four phases. The first phase involved using pilot scale intermittent sand filters which were placed into operation July 24, 1974, and operated until December 1, 1974, when freezing conditions forced suspension of experimentation. The second phase involved the use of laboratory scale filters which were operated from March 20 to May 3 of 1975. Phase III was based on the information obtained during Phase II and involved operating the Phase I pilot scale filters from July 10, 1975, to August 28, 1975. During Phase III a decreasing hydraulic loading rate was employed through the filter series. During September of 1975, the fourth phase of series operation took place, which involved equal loading to all filters in the series as in Phase I. All operations on the series intermittent sand filtration project ceased on September 30, 1975.

Phase I–field filters during 1974

Nine pilot scale filters, which had been employed by Marshall and Middlebrooks (1974) in a previous study were rearranged into three series operations using three filters each at the Logan Municipal Sewage Lagoon site. These filters were 4 feet by 4 feet (1.2 m x 1.2 m) and 6 feet high (1.83 m). They had been constructed of plywood and lined with fiberglass and resin. All nine filters had three, 4 inch (10.2 cm), layers of graded gravel (Table 8) and 30 inches (76.2 cm) of sand over the gravel. Three series operations using three filters with different effective size sands were connected in series as shown in Figures 1 and 2.

Table 8. Gravel size and depth of layers of underdrain material used for the series operation.

Gravel Size	Thickness of Layer
1/4 inch (6 mm) maximum	4 inches (7.5 cm)
3/4 inch (19 mm) maximum	4 inches (7.5 cm)
1 ½ inch (38 mm) maximum	4 inches (7.5 cm)

The first filter in each series contained 0.72 mm (0.0284 inch) effective size sand; the second filter contained 0.40 mm (0.0158 inch) effective size sand; and the third filter in each series contained 0.17 mm (0.0067 inch) effective size sand. The 0.72 mm (0.0284 inch) and the 0.17 mm (0.0067 inch) effective size sands were the same sands used by Marshall and Middlebrooks (1974). LeGrand Johnson Construction Company of Logan, Utah, had furnished the 0.17 mm (0.0067 inch) effective size sand which was washed bank run sand used primarily as fine aggregate in concrete. Marshall and Middlebrooks (1974) produced the 0.72 mm (0.0284 inch) effective size sand by sieving out the fines from the 0.17 mm (0.0067 inch) effective size sand. The 0.40 mm (0.0158 inch) effective size sand was sand blasting grit sand obtained from Utah Sand and Gravel Company in Salt Lake City, Utah. The 0.72 mm (0.0284 inch) and 0.17 mm (0.0067 inch) effective size sand filters were scraped before operations began because they had been used by Marshall and Middlebrooks (1974).

The sieve analysis of the three sands is shown in Table 9. The sieve analysis, which was run during the summer of 1974, differs slightly from that given by Marshall and Middlebrooks (1974) for the 0.17 mm (0.0067 inch) and 0.72 mm (0.0284 inch) effective size sands. However, the difference is slight and in order to be consistent with previous studies (Marshall and Middlebrooks, 1974), the sand shall hereafter be referred to as 0.72 mm (0.0284 inch) and 0.17 mm (0.0067 inch) effective size sand.

Information obtained from Utah Sand and Gravel Company indicated that their sand blasting grit sand could range in effective size from 0.30 mm (0.0118 inch) to 0.45 mm (0.018 inch) from bag to bag. However, they stated it should average about 0.40 mm (0.0158 inch) effective size overall. The sieve analysis ranged from about 0.33 mm (0.013 inch) to about 0.47 mm (0.0185 inch) for individual samples from the bags with an overall average of four samples of 0.38 mm (0.015 inch) effective size. Thus, this sand shall hereafter be referred to as 0.40 mm (0.0158 inch) effective size sand.

During the 1974 study, secondary effluent was applied to the 0.72 mm (0.0284 inch) effective size sand filter in each series by a calibrated pump[1] operated for a specified period of time. As shown in Figure 1 the loading rates used were 0.5 mgad (4,676.96 m^3/h-d), 1.0 mgad (9,353.9 m^3/h-d), and 1.5 mgad (14,030.9 m^3/h-d). The nine filters contained spreading troughs and drop pipes to distribute the flow evenly and prevent sand surface scouring of the filter surface.

Effluent from the 0.72 mm (0.0284 inch) effective size sand filters were collected into rain gutters. The effluent flowed by gravity to the 0.40 mm (0.01575 inch) effective size sand filters in each series. Effluent from the 0.40 mm (0.0158 inch) effective size sand filters flowed into sumps (55 gallons; 0.21 m^3 barrels) where automatic sump pumps[2] pumped the wastewater to the 0.17 mm (0.0067 inch) effective size sand filters.

During Phase I, plugging was assumed to occur when the daily loading of influent failed to pass

[1]Cole-Parmer Instrument Company, Chicago, Illinois, High Capacity Centrifugal Pumps, Model 7006-10.

[2]Teel Company, Chicago, Illinois, Sump Pump, Model 3P588.

Figure 2. Pilot scale field intermittent sand filters at the Logan Sewage Lagoons.

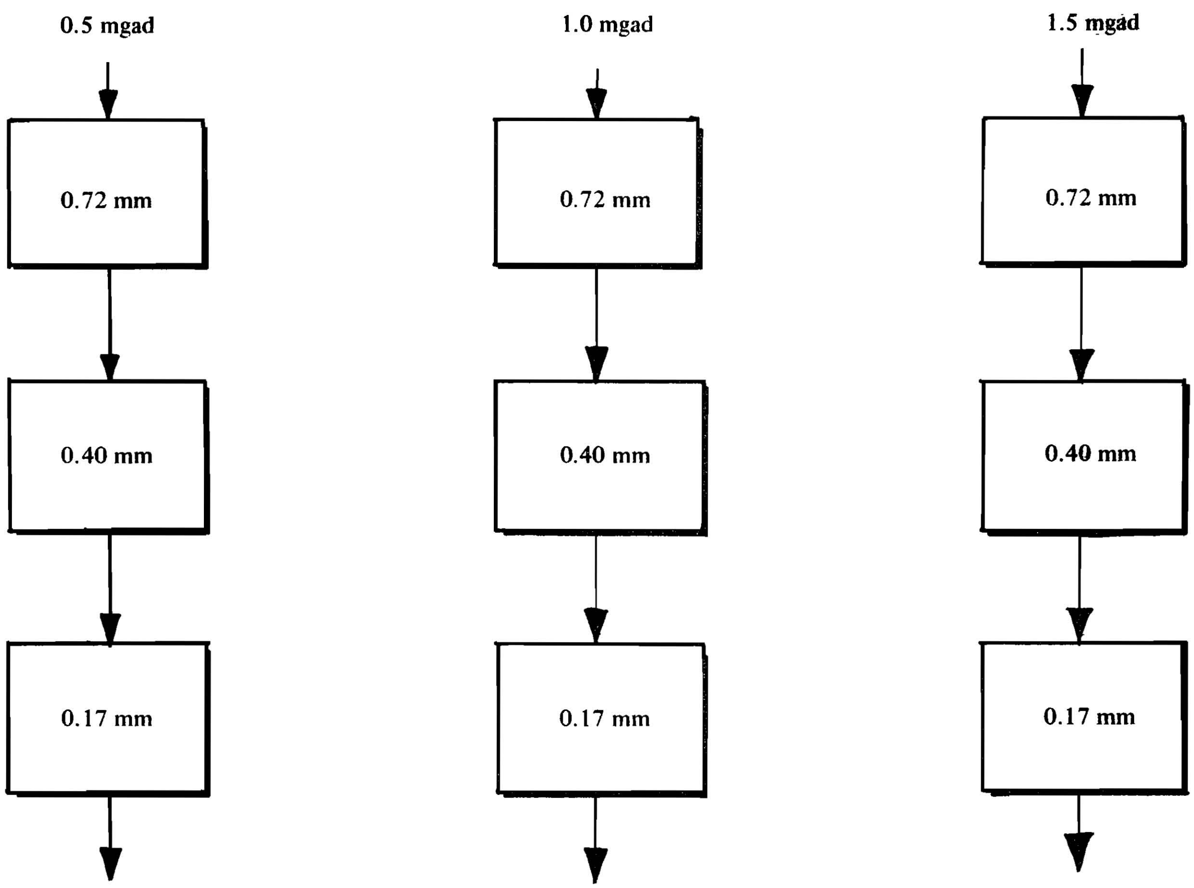

Figure 1. Series intermittent sand filtration operation during the pilot scale field study during 1974 (Phase I).

through the filter series within a 24 hour period. However, during Phase I, plugging did not occur.

Phase II–laboratory filters during 1975

During the field loading phase (Phase I), the filters had not plugged. Therefore, a laboratory scale series intermittent sand filtration operation was set up at the Utah Water Research Laboratory, Logan, Utah, using much higher hydraulic loading rates.

Three filter columns were constructed of 6 inch (15.2 cm) polyvinyl chloride (PVC) pipe. A 1 foot (30.5 cm) clear plexiglass section was attached in the middle of the section in order to observe the sand filter surface. A flanged coupling was provided in the middle of each column to facilitate filter cleaning. The operation is shown in Figure 3. The operation used sump switches, sump barrels, and three small pumps[3] to fill the filters automatically. The underdrain gravel and sand specifications were identical to those of the field study. The sand was compacted by water to a depth of 30 inches (74.4 cm).

An initial loading rate of 12 mgad (112,247.1 m^3/h-d) was used. This was reduced for two filter runs to 8 mgad (74,831.4 m^3/h-d). Phase II was completed using a 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate for the series operation. Logan sewage lagoon secondary effluent was used to load the series operation once daily, usually in the mornings.

During the 12 mgad (112,247.1 m^3/h-d) and the 8 mgad (74,831.4 m^3/h-d) hydraulic loading rates, secondary lagoon effluent was collected once every three days. The secondary effluent was kept in a 24 hour illuminated[4] model stabilization pond during the other two days. A submersible pump[5] was used to keep the contents of the model lagoon continually mixed. Samples of the stored influent were collected and analyzed at least every two days for BOD_5, suspended solids, and volatile suspended solids to determine if storage had a significant effect on the secondary effluent. During the 4 mgad (37,415.7 m^3/h-d) filter run, the secondary effluent was transported daily from the Logan sewage lagoon to the series intermittent sand filter operation.

As in Phase I, plugging was assumed to have occurred when the daily loading of influent failed to pass through the filter series within a 24 hour period.

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[3]Cole-Parmer Instrument Company, Chicago, Illinois, Lab-Puppy Centrifugal Pumps, Model 7097-20.

[4]Western Lighting Corporation, Los Angeles, California, Four Table High Intensity Light Bank, Issue B.

[5]Little Giant Vaporizer Company, Oklahoma City, Oklahoma, Submersible Pump, Model 4.

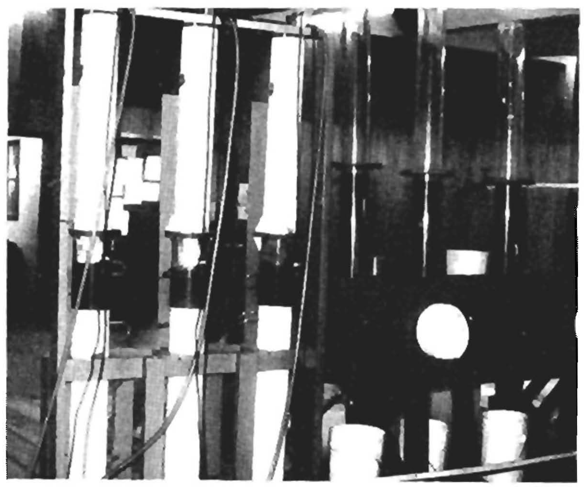


Figure 3. Laboratory scale series intermittent sand filters at the Utah Water Research Laboratory.

Table 9. Sieve analysis of filter sand used for the series operation.

U.S. Sieve Designation Number	Size of Opening (mm)	Large Sized Sand % Passing	Medium Sized Sand % Passing	Small Sized Sand % Passing
3/8 inch	9.423	99.9	100	99.9
4 inch	4.76	99.5	100	99.5
8 inch	2.362	82.1	100	83.4
16 inch	1.168	45.9	94.4	60.9
30 inch	0.59	4.2	33.1	38.2
50 inch	0.295	0.8	4.7	16.9
100 inch	0.149	0.7	1.2	7.8
200 inch	0.074	0.6	-	5.3
Pan	-	-	-	-
		E.S. = 0.71 mm	E.S. = 0.38 mm	E.S. = 0.18 mm
		U.C..= 2.1	U.C. = 1.95	U.C. = 6.2

Sumps (barrels), located before each filter in the series, could hold the days entire loading. Thus each filter could work as fast as it could filter the wastewater. When one filter in the series plugged, all the filters were scraped in order to maintain control. The filters were placed back in operation the day after plugging.

Phase III–field filters decreasing loading during 1975

The 0.17 mm (0.0067 inch) effective size sand filter had plugged first in all cases during Phase II. Therefore, a decreasing hydraulic loading operation was established for the pilot scale intermittent sand filters located at the Logan lagoons. Splitter boxes were used to equally divide the flow from the 0.72 mm (0.0284 inch) and the 0.40 mm (0.0158 inch) effective size sand filters in each series (Figure 4). These splitter boxes were 4 feet by 1 foot by 1 foot (1.2 m x 0.305 m x 0.305 m), and were constructed of plywood lined with fiberglass and resin. These splitter boxes contained two 22 degree V-notch weirs to divide the flow. The succeeding filter in each series received only one-half of the loading of the previous filter. During Phase III only two series operations (six filters) were in operation.

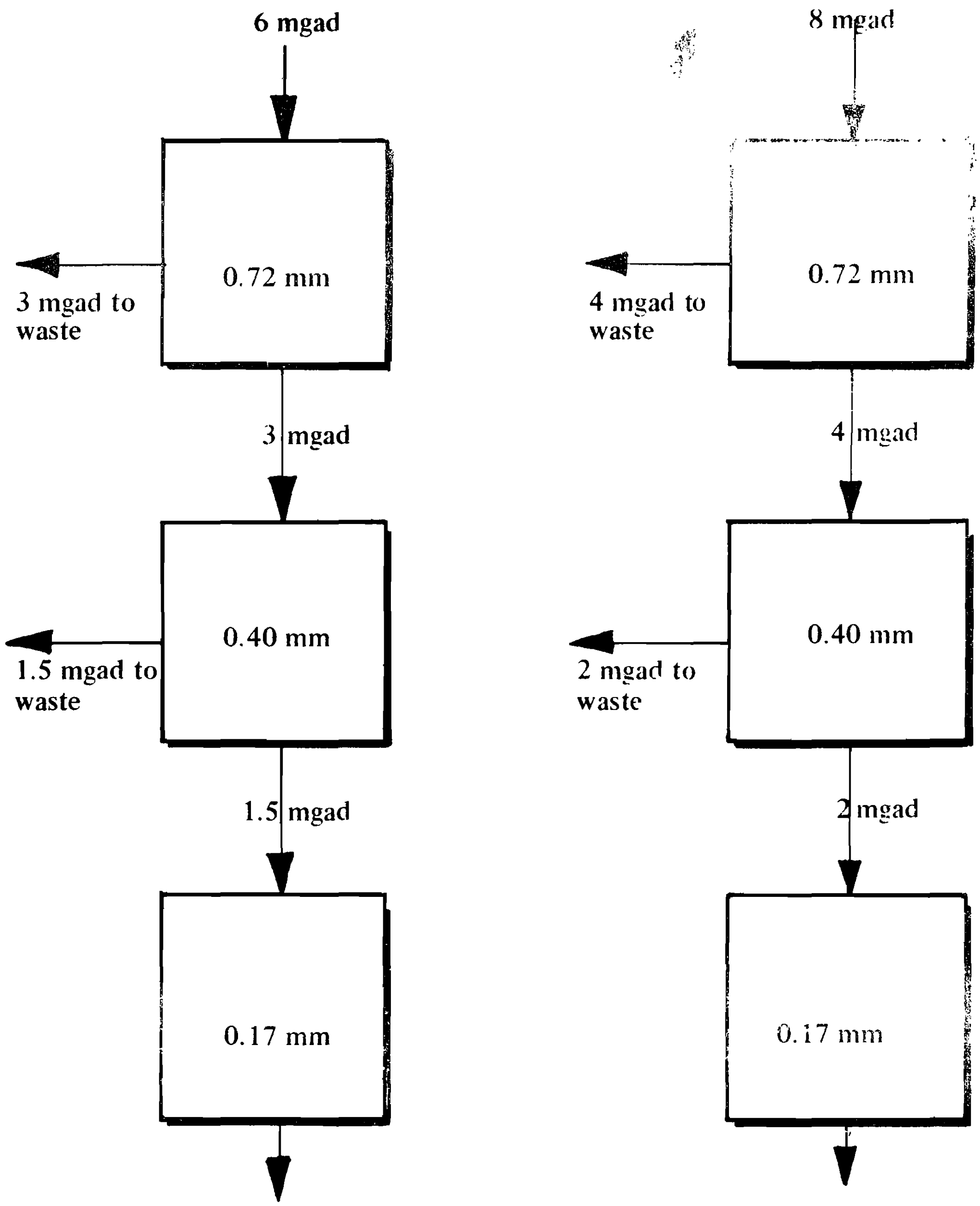


Figure 4. Typical series intermittent sand filtration operation during the decreasing loading period of 1975 (Phase III).

Loading rates of 16 - 8 - 4 mgad (149,662.8 - 74,831.4 - 37,415.7 m^3/h-d), 12 - 6 - 3 mgad (112,247.1 - 56,123.6 - 28,061.8 m^3/h-d), 8 - 4 - 2 mgad (74,831.4 - 37,415.7 - 18,707.9 m^3/h-d), and 6 - 3 - 1.5 mgad (56,123.6 - 28,061.8 - 14,030.9 m^3/h-d) were used during Phase III. Because of the large loadings, automatic timers[6,7] were used to operate the pumps for a certain percentage of time an hour over a specified time interval. As the filters lost head and began to plug the daily loading was accomplished over a longer period of time.

Before series operations were started for the year, the top 2 inches (5.1 cm) of sand were removed from each filter. This removed sand was not replaced.

During Phase III, because of the high hydraulic loading rates and the intermittent automatic loading, plugging was assumed to occur when the filter freeboard could no longer contain the daily loading rate of influent. The 0.72 mm (0.0284 inch) effective size sand filter in each series was in reality a slow sand filter during Phase III. When one filter in a series plugged, all filters in the series were scraped in order to maintain control of the operation.

Phase IV–field filters equal loading during 1975

During September of 1975, all three filters in the two series operations were loaded at the same hydraulic loading rate. Hydraulic loading rates of 4 mgad (37,415.7 m^3/h-d) and 3 mgad (28,061.8 m^3/h-d) were used for the two series operations.

Due to several filter runs during Phase III, the top 6 inches (15.2 cm) of sand were removed from all six filters before starting Phase IV. New clean sand was then added to obtain the original 30 inch (76.2 cm) sand depth.

As in Phase III, automatic timers were used to operate the pumps. However, plugging was assumed to occur when the daily loading of influent failed to pass through the filters within a 24 hour period after initial loading.

Sampling

Pilot scale field filter effluent grab samples from each filter in the series were collected once weekly during Phases I and IV and once per filter run during Phase III. The effluent samples were collected within 45 minutes following the application of the daily loading upon the filters. An influent grab sample was taken the same day as the effluent samples.

During Phase III, grab samples were collected once a filter run. Twice a week sampling was to have been performed; however, the splitting of the daily loading following the 0.72 mm (0.0284 inch) and 0.40 mm (0.0158 inch) effective size sand filters in each series resulted in very small flows to the succeeding filters when the 0.72 mm (0.0284 inch) effective size sand filter started to plug. Therefore, representative sampling of the 0.40 mm (0.0158 inch) and 0.17 mm (0.0067 inch) effective size sand filters effluent was impossible when the 0.72 mm (0.0284 inch) effective size sand filter started to lose head (i.e. plugging started). During the decreasing loading phase (Phase III), the 0.72 mm (0.0284 inch) effective size sand filter plugged first in all cases.

During the early stages of the laboratory scale operation (Phase II), filter effluents were sampled twice weekly for BOD_5 and at least every other day for suspended solids and volatile suspended solids. The influent BOD_5, suspended solids, and volatile suspended solids were sampled at least every two days to monitor changes caused by storage of the secondary lagoon effluent in the laboratory. Effluent grab samples from each filter in the series were taken within 20 minutes after loading, and influent grab samples were collected the same day. During the 4 mgad (37,415.7 m^3/h-d) series operation weekly influent and effluent grab samples were collected.

Analysis

The influent and effluent from each pilot scale filter (Phases I, III, and IV) were analyzed for biochemical oxygen demand, suspended solids, volatile suspended solids, pH, temperature, and dissolved oxygen concentrations. These analyses were performed according to Standard Methods (American Public Health Association, 1971; hereafter referred to APHA, 1971).

Only biochemical oxygen demand, suspended solids, and volatile suspended solids were analyzed in the laboratory portion of the study (Phase II).

Phytoplankton and zooplankton counts on the secondary lagoon effluent were obtained from a prototype single stage intermittent sand filter project using the same influent. Phytoplankton and zooplankton were counted using a Sedwick-Rafter Counting Cell according to Standard Methods (APHA, 1971). Phytoplankton and zooplankton influent counts are available for Phases I and II. During Phase III, the prototype single stage system was not in operation; thus, no plankton counts are available. Phytoplankton counts were available for Phase IV, but zooplankton counts were not available.

[6]Ripley Company, Incorporated, Middletown, Connecticut, TSA-14 Percentage Timer.

[7]Sears and Roebuck Company, Time Switch Model 5870.

Statistical Analysis

The data were analyzed by using the standard analysis of variance techniques as described by Neter and Wasserman (1974). During Phase I, the effluent parameters were compared using a randomized block design where the sample dates were blocked. The data were not statistically analyzed during Phases II, III, and IV or between phases due to differing influent characteristics and/or an insufficient number of data points. Analyses were performed using a Burroughs 6700 computer located at Utah State University. A statistical package, with STATPAC-BASIC (Hurst, no date), was used to perform the randomized block design analysis of variance. If the calculated values of F (treatments mean square divided by the error mean square in the analysis of variance calculations) indicated a significant difference between treatment means, when more than two treatments were being compared, Duncan's Multiple Range Test (Duncan, 1955) was used to determine which means were significantly different.

During all phases, mean and standard deviations were calculated for the data in accordance with procedures described by Ostle (1963).

concentration once and averaged 9.5 mg/l for the 12 mgad (112,247.1 m^3/h-d) hydraulic loading rate. The final effluent BOD_5 concentration averaged 6.9 mg/l for the 8 mgad (74,831.4 m^3/h-d) hydraulic loading rate and 2.3 mg/l for the 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate.

Suspended solids results

Table 21 summarizes the suspended solids results from the laboratory study. The influent suspended solids concentration was very high during the 12 mgad (112,247.1 m^3/h-d) hydraulic loading rate averaging 48.1 mg/l. It decreased to approximately 20 mg/l for the 8 mgad (74,831.4 m^3/h-d) and 4 mgad (37,415.7 m^3/h-d) hydraulic loading rates.

Total percent removal of suspended solids for the series averaged about 70 percent for the 12 mgad (112,247.1 m^3/h-d) and the 8 mgad (74,831.4 m^3/h-d) hydraulic loading rates and 86 percent for the 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate. The series average effluent suspended solids concentration ranged from 2.5 mg/l for a hydraulic loading rate of 4 mgad (37,415.7 m^3/h-d) to 13.2 mg/l for a hydraulic loading rate of 12 mgad (112,247.1 m^3/h-d).

The "filter washing" effect which took place during Phase I did not occur during the laboratory phase because the gravel was thoroughly washed before it was placed in the filter columns.

Volatile suspended solids results

The laboratory study volatile suspended solids results are summarized in Table 22. The influent volatile suspended solids concentration during the 12 mgad (112,247.1 m^3/h-d) hydraulic loading rate was 38.4 mg/l. The influent concentrations were much lower for the 8 mgad (74,831.4 m^3/h-d) and 4 mgad (37,415.7 m^3/h-d) hydraulic loading rates averaging 5.7 mg/l and 1.1 mg/l respectively.

It should be emphasized that removal efficiency of volatile suspended solids improved for all three filters in the series as the hydraulic loading rate was reduced to 4 mgad (37,415.7 m^3/h-d). This improved performance may be due to the lower loading rate, the differing influent population or both. Zooplankton *(Daphnia)* and green algae

Table 21. Average suspended solids concentration for the laboratory scale filters during 1975 (Phase II).

Loading Rate	Applied S.S. (mg/l)	Effluent Suspended Solids (mg/l) — Effective Size of Filter Sand		
		0.72 mm	0.40 mm	0.17 mm
12 mgad (112,247.1 m^3/h-d)	48.1	29.5	17.4	13.2
8 mgad (74,831.4 m^3/h-d)	23.6	16.2	7.8	6.9
4 mgad (37,415.7 m^3/h-d)	18.1	7.7	4.3	2.5

Table 22. Average volatile suspended solids concentration for the laboratory scale filters during 1975 (Phase II).

Loading Rate	Applied VSS (mg/l)	Effluent VSS (mg/l) — Effective Size of Filter Sand		
		0.72 mm	0.40 mm	0.17 mm
12 mgad (112,247.1 m^3/h-d)	38.4	26.1	15.3	9.6
8 mgad (74,831.4 m^3/h-d)	15.1	10.3	5.4	5.7
4 mgad (37,415.7 m^3/h-d)	7.3	2.9	1.7	1.1

(*Micractinium* sp.) were predominant in the influent during the 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate.

Length of filter runs

Table 23 summarizes the average length of the filter run for the laboratory study. Filter runs obtained at the 12 mgad (112,247.1 m^3/h-d) and the 8 mgad (74,831.4 m^3/h-d) hydraulic loading rates are averages of two runs. Length of filter runs for the hydraulic loading rate averaged 3.5 days for the 12 mgad (112,247.1 m^3/h-d) and 5.5 days for the 8 mgad (74,831.4 m^3/h-d).

For the series filter run obtained at the 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate, the length of the run was 16 days.

The length of the filter runs increased as the hydraulic loading rate decreased. However at the 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate, the series plugged in 15 days compared to a 130 day plus filter run for the 1.5 mgad (14,030.9 m^3/h-d) hydraulic loading rate during Phase I.

During the laboratory study, the 0.17 mm (0.0067 inch) effective size sand filter plugged first at all three hydraulic loading rates. When plugging occurred, the top 4 inches (10.2 cm) of sand was removed and replaced in all three filters in the series to maintain control on the operation. The top 2 inches (5.1 cm) of the 0.17 mm (0.0067 inch) filter sand appeared to be slightly cemented together by the trapped solids as mentioned by Marshall and Middlebrooks (1974).

At times during the laboratory study, algae were observed, by means of a greenish tint to the sand, at the 4 inch (10.2 cm) depth in the 0.17 mm (0.0067 inch) and 0.40 mm (0.0158 inch) effective size sand filter columns. Because of the high loading rates, a head of 1.5 feet (45.7 cm) to 2.5 feet (76.2 cm) existed on the filters the majority of the time during daily loading. This high head may have forced some of the algae (especially blue-greens) deeper into the filter media. The upper 8 inches (20.4 cm) of the sand bed were contained in the clear plexiglass sections of the filter columns. Light may have stimulated algal growth within the sand in the upper 4 inches (10.2 cm) of the bed. The algal penetration and possible growth may have contributed to the failure of the 0.17 mm (0.0067 inch) effective size sand filter to perform as expected. It may also account for the somewhat shorter than expected filter run exhibited during the 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate.

Table 23. Average length of filter runs for laboratory scale filters during 1975 (Phase II).

Hydraulic Loading Rate	Length of Filter Runs (Days)	Filter in Series Which Plugged
12 mgad (112,247.1 m^3/h-d)	3.5[a]	0.17 mm
8 mgad (74,831.4 m^3/h-d)	5.5[a]	0.17 mm
4 mgad (37,415.7 m^3/h-d)	16	0.17 mm

[a]Average of 2 runs.

Jones and Taylor (1965) reported that soil clogging took place 3 to 10 times faster under anaerobic conditions than under aerobic conditions. The high hydraulic loading rates used during the initial part of Phase II combined with the high BOD_5 concentration, may have caused anaerobic conditions to exist in the 0.17 mm (0.0067 inch) effective size sand filter which may account for the poor performance and early plugging.

Field Filters Decreasing Loading Results During 1975 (Phase III)

General

The results for BOD_5, suspended solids, volatile suspended solids, temperature, pH, and dissolved oxygen are tabulated in Tables A-13, A-14, and A-15 in Appendix A. As mentioned in the "Methods and Procedures" section of this paper, sampling problems were encountered during Phase III. Therefore the data collected may not be representative of actual performance.

Influent algae and zooplankton

Algae and zooplankton counts were not performed during the decreasing loading phase; however, general visual observations indicated that zooplankton *(Daphnia)* were present in very high concentrations during the early part of Phase III. The secondary effluent did not exhibit a green tint (indicating algae) during this period of the study. Algae dominated the secondary lagoon effluent during the last stages of Phase III; however, zooplankton *(Daphnia)* were still insignificant concentrations.

The influent plankton population during Phase III was much different than the population during Phases I and II where blue-green algae dominated the influent plankton population. It also differed from results reported by Marshall and Middlebrooks (1974) where green algae (mainly *Chlamydomonas* sp.) dominated.

During the later stages of Phase III, suspended solids analysis revealed that a small number of algae had passed through all three filters in the series. This observation was also reported for the field study of 1974 (Phase I) and the laboratory study (Phase II).

BOD results

BOD_5 removal performance of the pilot scale field filters during the decreasing loading phase (Phase III) are summarized in Table 24. The influent BOD_5 concentration ranged from 30.6 mg/l for the 16-8-4 mgad (149,662.8 - 74,831.4 - 37,415.7 m^3/h-d) and 12-6-3 mgad (112,247.1 - 56,123.6 - 28,061.8 m^3/h-d) hydraulic loading rates to 10.7 mg/l for the 8-4-2 mgad (74,831.4 - 37,415.7 - 18,707.9 m^3/h-d) hydraulic loading rate. The final series effluent BOD_5 concentration averaged 6.9 mg/l for the 16-8-4 mgad (149,662.8 - 74,831.4 - 37,415.7 m^3/h-d) hydraulic loading rate and 7.1 mg/l for the 12-6-3 mgad (112,247.1 - 56,123.6 - 28,061.8 m^3/h-d) hydraulic loading rate. However this consisted of one data point and may not be truly representative. The series effluent BOD_5 concentration averaged 2.4 mg/l for the 8-4-2 mgad (74,831.4 - 37,415.7 - 18,707.9 m^3/h-d) hydraulic loading rate and 2.3 mg/l for the 6-3-1.5 mgad (56,123.6 - 28,061.8 - 14,030.9 m^3/h-d) hydraulic loading rate.

Suspended solids results

Table 25 summarizes the suspended solids performance of this phase. Influent suspended solids concentration ranged from 51.0 mg/l at the 16-8-4 mgad (149,662.8 - 74,831.4 - 37,415.7 m^3/h-d) and 12-6-3 mgad (112,247.1 - 56,123.6 - 28,061.8 m^3/h-d) hydraulic loading rates to 24.2 mg/l for the 8-4-2 mgad (74,831.4 - 37,415.7 - 18,707.9 m^3/h-d) hydraulic loading rate. The final series effluent suspended solids concentration ranged from 4 to 5 mg/l at the 16-8-4 mgad (149,662.8 - 74,831.4 - 37,415.7 m^3/h-d) and 12-6-3 mgad (112,247.1 - 56,123.6 - 28,061.8 m^3/h-d) hydraulic loading rates to over 8 mg/l for the 8-4-2 mgad (74,831.4 - 37,415.7 - 18,707.9 m^3/h-d) and 6-3-1.5 mgad (56,123.6 - 28,061.8 - 14,030.9 m^3/h-d) hydraulic loading rates.

The 0.72 mm (0.0284 inch) effective size sand filters experienced a large suspended solids (as well as volatile suspended solids) removal during the 16-8-4 mgad (149,662.8 - 74,831.4 - 37,415.7 m^3/h-d) and 12-6-3 mgad (112,247.1 - 56,123.6 - 28.061.8 m^3/h-d) hydraulic loading rates. This was due to the almost total domination of the influent plankton population by *Daphnia*. Referring to Tables A-14 and A-15 in Appendix A, the last two sample dates (August 21 and 28) were taken when the algae dominated the influent. The efficiency of suspended solids (as well as volatile suspended solids) removal was somewhat lower during these dates than earlier in the phase when *Daphnia* was in large numbers. During the part of Phase III when algae dominated the influent, the final series effluent suspended solids concentrations ranged from 8.5 mg/l for the 6-3-1.5 mgad (56,123.6 - 28,061.8 - 14,030.9 m^3/h-d) hydraulic loading rate on August 28 to 23.9 mg/l for the 8-4-2 mgad (74,831.4 - 37,415.7 - 18,707.9 m^3/h-d) hydraulic loading rate on August 21. Thus, the nature of the influent appears to have a significant effect on the quality of the filter effluent.

Table 24. Average BOD_5 concentrations for pilot scale field filters during the decreasing loading period of 1975 (Phase III).

Loading Rate	Applied BOD_5 (mg/l)	Effluent BOD_5 (mg/l) — Effective Size of Filter Sand		
		0.72 mm	0.40 mm	0.17 mm
16-8-4 mgad (149,662.8-74,831.4-37,415.7 m^3/h-d)	30.6	10.5	9.3	6.9
12-6-3 mgad (112,247.1-56,123.6-28,061.8 m^3/h-d)	30.6	9.5	6.6	7.1
8-4-2 mgad (74,831.4-37,415.7-18,707.9 m^3/h-d)	10.7	7.4	4.9	2.4
6-3-1.5 mgad (56,123.6-28,061.8-14,030.9 m^3/h-d)	11.1	4.6	3.5	2.3

"Filter washing" (washing of inert fines from the filters) did not occur during Phase III or the following phase because these field filters were the same filters used during the initial field study (Phase I). "Filter washing" was accomplished during the first phase.

Volatile suspended solids

The decreasing loading phase volatile suspended solids results are summarized in Table 26. The influent volatile suspended solids concentrations ranged from 31.0 mg/l at the 16-8-4 mgad (149,662.8 - 74,831.4 - 37,415.7 m^3/h-d) and 12-6-3 mgad (112,247.1 - 56,123.6 - 28,061.8 m^3/h-d) hydraulic loading rates to 15.9 mg/l at the 8-4-2 mgad (74,831.4 - 37,415.7 - 18,707.9 m^3/h-d) hydraulic loading rate.

The final effluent volatile suspended solids concentration was below 2 mg/l for the 16-8-4 mgad (149,662.8 - 74,831.4 - 37,415.7 m^3/h-d) and 12-6-3

Table 25. Average suspended solids concentrations for the pilot scale field filters during the decreasing loading period of 1975 (Phase III).

Loading Rate	Applied S.S. (mg/l)	Effluent Suspended Solids (mg/l) Effective Size of Filter Sand		
		0.72 mm	0.40 mm	0.17 mm
16-8-4 (149,662.8-74,831.4-37,415.7 m^3/h-d)	51.0	5.7	4.1	5.1
12-6-3 mgad (112,247.1-56,123.6-28,061.8 m^3/h-d)	51.0	5.7	3.8	4.4
8-4-2 mgad (74,831.4-37,415.7-18,707.9 m^3/h-d)	24.2	9.2	7.4	8.0
6-3-1.5 mgad (56,123.6-28,061.8-14,030.9 m^3/h-d)	35.3	12.9	10.9	8.5

Table 26. Average volatile suspended solids concentrations for the pilot scale field filters during the decreasing loading period of 1975 (Phase III).

Loading Rate	Applied VSS (mg/l)	Effluent VSS (mg/l) Effective Size of Filter Sand		
		0.72 mm	0.40 mm	0.17 mm
16-8-4 mgad (149,662.8-74,831.4-37,415.7 m^3/h-d)	31.0	2.9	1.9	2.0
12-6-3 mgad (112,247.1-56,123.6-28,061.8 m^3/h-d)	31.0	2.5	1.2	1.1
8-4-2 mgad (74,831.4-37,415.7-18,707.9 m^3/h-d)	15.9	6.6	6.2	4.5
6-3-1.5 mgad (56,123.6-28,061.8-14,030.9 m^3/h-d)	23.4	10.0	8.2	6.1

mgad (112,247.1 - 56,123.6 - 28,061.8 m^3/h-d) hydraulic loading rates. The final series effluent volatile suspended solids concentrations averaged 4.5 mg/l for the 8-4-2 mgad (74,831.4 - 37,415.7 - 18,707.9 m^3/h-d) hydraulic loading rate and 6.1 mg/l for the 6-3-1.5 mgad (56,123.6 - 28,061.8 - 14,030.9 m^3/h-d) hydraulic loading rate.

Length of filter runs

As reported earlier the 0.72 mm (0.0284 inch) effective size sand filter plugged first during all filter runs during Phase III. Table 27 summarizes the lengths of the filter runs. The two series loaded at the 16-8-4 mgad (149,662.8 - 74,831.4 - 37,415.7 m^3/h-d) and 12-6-3 mgad (112,247.1 - 56,123.6 - 28,061.8 m^3/h-d) hydraulic loading rates plugged after only one day. The 8-4-2 mgad (74,831.4 - 37,415.7 - 18,707.9 m^3/h-d) series operation averaged a 7.5 day length of filter run, while the 6-3-1.5 mgad (56,123.6 - 28,061.8 - 14,030.9 m^3/h-d) series operation averaged a 9.5 day length of filter run. The influent plankton characteristics seemed to have little effect upon the length of the filter runs. The length of the filter runs at the 8-4-2 mgad (74,831.4 - 37,415.7 - 18,707.9 m^3/h-d) hydraulic loading rate were 7, 7, 7, and 9 days. *Daphnia* were dominant during the earlier three runs, while algae were dominant during the last filter run. However *Daphnia* was still in significant concentrations during the last filter run.

Table 27. Average length of filter runs for the pilot scale field filters during the decreasing loading period of 1975 (Phase III).

Hydraulic Loading Rate	Length of Filter Runs (Days)	Filter in Series Which Plugged
16-8-4 mgad (149,662.8 - 74,831.4 - 37,415.7 m^3/h-d)	1	0.72 mm
12-6-3 mgad (112,247.1 - 56,123.6 - 28,061.8 m^3/h-d)	1	0.72 mm
8-4-2 mgad (74,831.4 - 37,415.7 - 18,707.9 m^3/h-d)	7.5[a]	0.72 mm
6-3-1.5 mgad (56,123.6 - 28,061.8 - 14,030.9 m^3/h-d)	9.5[b]	0.72 mm

[a]Average of 4 runs.
[b]Average of 2 runs.

Because of the large *Daphnia* population in the influent during the decreasing loading phase, a paper thin filtering skin formed on the 0.72 mm (0.0284 inch) effective size sand filter in each series. Filtering skins were not noticed on the 0.40 mm (0.0158 inch) or 0.17 mm (0.0067 inch) effective size sand filters at any time during the study. As the 0.72 mm (0.0284 inch) effective size sand filter in each series started to plug, the filtering skin became very impermeable. The 0.72 mm (0.0284 inch) effective size sand filter lost head slowly during the runs until plugging was near at which time head loss sharply increased. This seems to agree with observations reported by Thomas et al. (1966) in describing the clogging of soils by wastewater effluents.

As the 0.72 mm (0.0284 inch) effective size sand filter was allowed to dry between filter runs, the filtering skin cracked, curled up, and gave off a putrid odor. Scraping the filter involved only removing the thin filtering skin.

Because of the large and intermittent loadings used during Phase III, the 0.72 mm (0.0284 inch) effective size sand filters were operated much like slow sand filters. Perhaps if the daily loading could have been applied to the 0.72 mm (0.0284 inch) effective size sand filter as a "slug" dose, true intermittent operation would have been achieved and slightly longer filter runs may have resulted.

Field Filters Equal Loading Results During 1974 (Phase IV)

General

Table A-16 in Appendix A lists the BOD_5, suspended solids, volatile suspended solids, temperature, pH, and dissolved oxygen results for the field filter equal loading phase of 1975 (Phase IV). The series operation using the 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate had plugged two days before the final sampling date (September 26); however, the series was loaded on September 26 to obtain another data point.

Influent algae and zooplankton

Table 28 shows the characteristics of the influent algal population during Phase IV. Throughout Phase IV, *Palmella* sp. (a clumped green alga) was the dominant alga in the influent. The diatoms, *Melosira* sp. and *Navicula* sp., were also in significant numbers during the equal loading phase. The unknown blue-green alga present on September 18 was unidentified. Although zooplankton were not counted during Phase IV, they were visually observed in the influent especially during the later stages of the period.

The *Palmella* sp. (green alga) dominated influent of Phase IV was different from previous phases. *Daphnia* dominated the influent during Phase III, and blue-green algae dominated the influent plankton population during Phases I and II. Previous studies by Marshall and Middlebrooks (1974) reported using a *Chlamydomonas* sp. (green alga) dominated influent.

As reported in the previous phases, suspended solids analysis revealed that a small number of algae had passed through all the filters in the series operations.

BOD results

The BOD_5 results for Phase IV are summarized in Table 29, and the removal efficiency is plotted in Figure 9. During the period the influent BOD_5 concentration was low averaging only 7.0 mg/l. The series effluent BOD_5 concentration averaged 2.0 mg/l for a 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate and 1.4 mg/l for a 3 mgad (28,061.8 m^3/h-d) hydraulic loading rate. The final series filter effluent BOD_5 concentration was always well below 5 mg/l for the operation during Phase IV.

Better quality BOD_5 effluent was obtained for all three effective size sand filters at the 3 mgad (28,061.8 m^3/h-d) hydraulic loading rate than at the 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate. An analysis of variance comparison of effluent quality for the two different loading rates revealed no significant difference (1 percent level). However, there were not enough data points available to reach any definite conclusions.

Suspended solids results

The field filter equal loading period (Phase IV) performance of the two series for suspended solids is summarized in Table 30 and the removal efficiency is plotted in Figure 10.

The influent suspended solids concentration averaged 28.4 mg/l and ranged from 15.8 mg/l to 49.6 mg/l for the phase. The effluents suspended solids concentration averaged 5.8 mg/l for the 4 mgad

Table 28. Algae counts for influent to the pilot scale field filters during the equal loading period of September 1975 (Phase IV).

Algal Type	Cells Per ml			
	Sept. 2	Sept. 11	Sept. 18	Sept. 24
CHLOROPHYTA:				
(Green Algae)				
Chlamydomonas sp.	-	1,960	-	-
Chlorella sp.	-	-	3,528	392
Palmetta sp.	313,600	1,281,840	1,176	134,848
Schroederia sp.	-	-	784	-
Total Green Algae	313,600	1,283,800	5,488	135,240
CYANOPHYTA:				
(Blue-Green Algae)				
Gloeothece sp.	-	-	-	1,568
Microcystis sp.	8,820	6,860	11,956	9,212
Unknown Blue-Green	-	-	243,628	-
Total Blue-Greens	8,820	6,860	255,584	10,780
CHRYSOPHYTA:				
(Yellow-Green Diatoms)				
Melosira sp.	392	-	-	-
Navicula sp.	980	16,660	11,760	19,600
Total Diatoms	1,372	16,660	11,760	19,600
OTHER ALGAE:				
Cryptomonas sp.	588	-	392	-
TOTAL ALGAE	324,380	1,307,320	273,224	165,620

(37,415.7 m^3/h-d) hydraulic loading rate and 4.2 mg/l for the 3 mgad (28,061.8 m^3/h-d) hydraulic loading rate.

The removal efficiency of the three filters in series for the two loading rates increased as time of the filter runs progressed. This was due to the "schmutzdecke" (filtering skin) buildup on all of the filters. At the beginning of the runs, total suspended solids removal was approximately 55 percent for the 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate and 59 percent for the 3 mgad (28,061.8 m^3/h-d) hydraulic loading rate. These removal efficiencies improved to approximately 91 percent for the two operations at the end of the filter runs.

Volatile suspended solids removal

Influent and effluent volatile suspended solids performance for the experimental period are shown

Table 29. Average BOD_5 concentrations for the pilot scale field filters during the equal loading period of 1975 (Phase IV).

Loading Rate	Applied BOD_5 (mg/l)	Effluent BOD_5 (mg/l) — Filter in Series: 0.72 mm	0.40 mm	0.17 mm
4 mgad (37,415.7 m^3/h-d)	7.0	4.7	4.1	2.0
3 mgad (28,061.8 m^3/h-d)	7.0	4.3	2.9	1.4

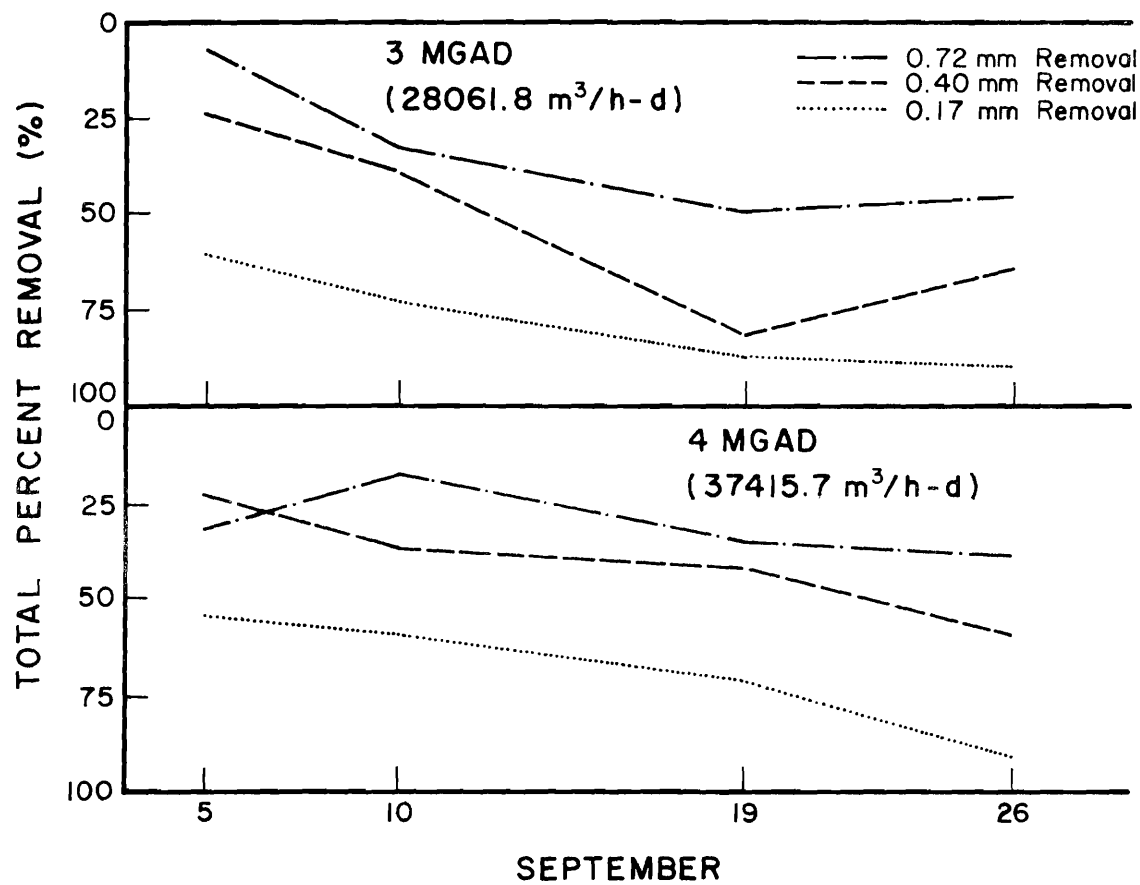


Figure 9. BOD_5 total percent removal by the pilot scale field filters during the equal loading period of 1975 (Phase IV).

in Table 31 and Figure 11. The influent volatile suspended solids concentration averaged 19.6 mg/l and ranged from 10.8 mg/l to 36.5 mg/l. For the 3 mgad (28,061.8 m^3/h-d) hydraulic loading rate, the effluent averaged 3.3 mg/l, and for the 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate, the effluent averaged 3.6 mg/l.

The volatile suspended solids total removal efficiency for the filters was almost identical to the suspended solids removal efficiency. Efficiency of removal improved as the "schmutzdecke" (filtering skin) built up on the filters. The two series filter operations averaged approximately 82 percent removal of volatile suspended solids for the experimental period.

pH, temperature, and dissolved oxygen results

The pH, temperature, and dissolved oxygen results for Phase IV are tabulated in Table A-16 in

Table 30. Average suspended solids concentrations for the pilot scale field filters during the equal loading period of 1975 (Phase IV).

Loading Rate	Applied Suspended Solids (mg/l)	Effluent Suspended Solids (mg/l) Filter in Series 0.72 mm	0.40 mm	0.17 mm
4 mgad (37,415.7 m^3/h-d)	28.4	10.0	8.8	5.8
3 mgad (28,061.8 m^3/h-d)	28.4	10.0	8.7	4.2

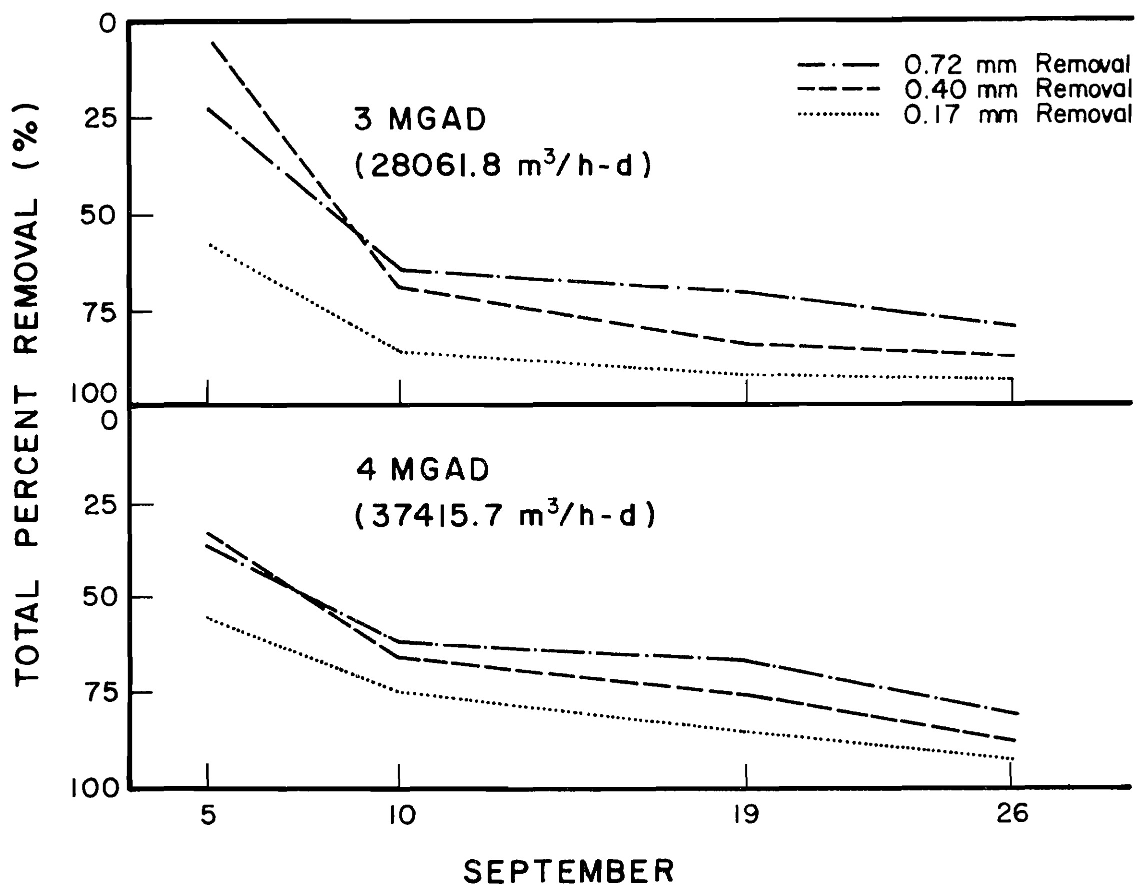


Figure 10. Suspended solids total percent removal by the pilot scale field filters during the equal loading period of 1975 (Phase IV).

Appendix A. The pH of the wastewater continually decreased as it passed through each filter in the series. During the period, the influent pH was greater than 9.0 for all sample periods, and the final series effluent pH averaged about 8.5.

The temperature of the wastewater decreased slightly during the experimental period of September as the climate started to turn colder. Except on one occasion, dissolved oxygen concentrations were always greater than 5.5 mg/l for all influents and effluents during this phase. The 0.72 mm (0.0284 inch) effective size sand filter effluent for the 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate had a dissolved oxygen concentration of 4.10 mg/l on the last sample date. However, this sample was collected after the 0.72 mm (0.0284 inch) effective size sand filter had plugged. The influent dissolved oxygen concentration was 13.1 mg/l on this sample date. It appears that the organic mat on the plugged filter

Table 31. Average volatile suspended solids concentrations for the pilot scale field filters during the equal loading period of 1975 (Phase IV).

Loading Rate	Applied VSS (mg/l)	Effluent VSS (mg/l) Filter in Series		
		0.72 mm	0.40 mm	0.17 mm
4 mgad (37,415.7 m^3/h-d)	19.6	6.1	4.8	3.6
3 mgad (28,061.8 m^3/h-d)	19.6	6.8	4.8	3.3

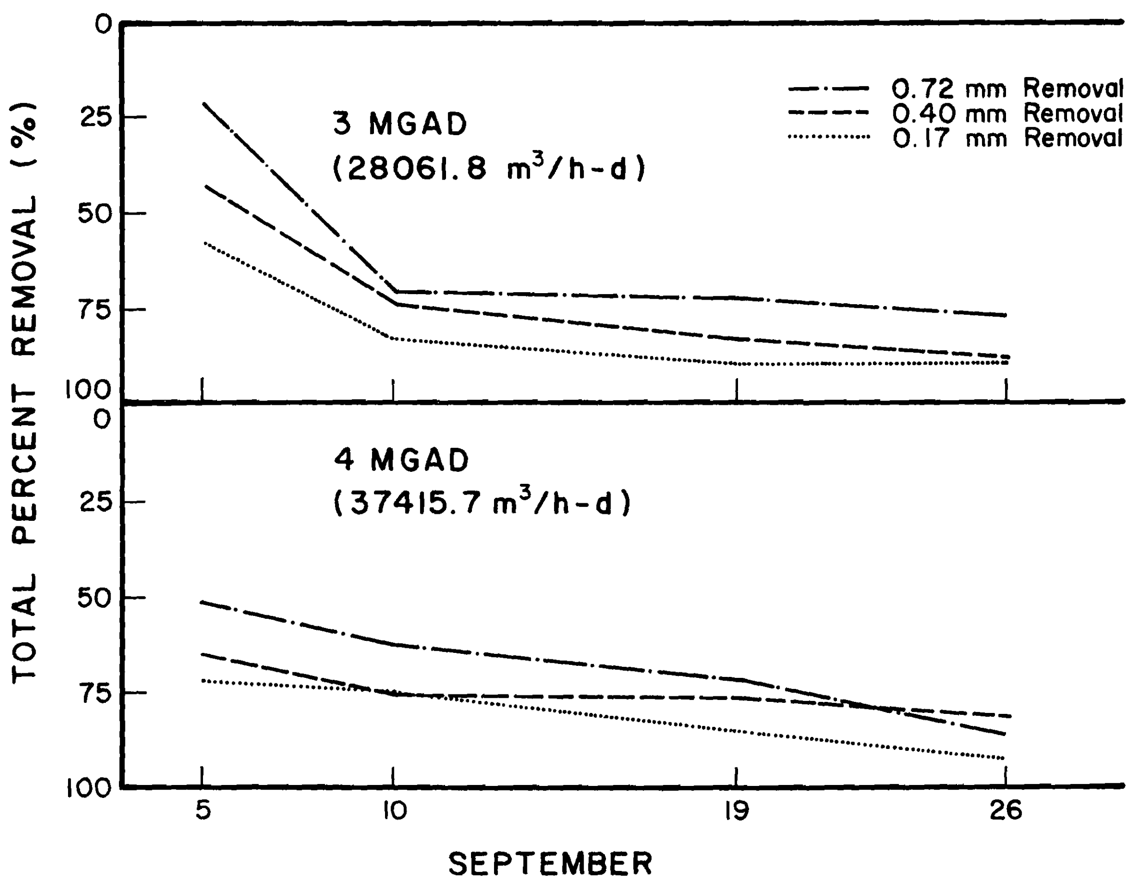


Figure 11. Volatile suspended solids total percent removal by the pilot scale field filters during the equal loading period of 1975 (Phase IV).

surface had a large oxygen demand due to decomposition of the trapped organic matter.

Pollutant mass loading of filters

Another attempt was made during Phase IV to determine which filter or filters in the series operation removed the majority of the BOD_5, suspended solids, and volatile suspended solids. The mass removals are tabulated in Tables 32, 33, and 34. These pollutant mass removals are the total mass removed by each filter in the series until one filter in the series plugged. The 0.72 mm (0.0284 inch) effective size sand filter plugged first for the two hydraulic loading rates.

During Phase IV, the series operations removed approximately 16,000 lbs/acre (17,934 kg/hectare) of suspended solids and 11,000 lbs/acre (12,329 kg/hectare) of volatile suspended solids before one filter in the series plugged. The amount of removal was almost identical at the two hydraulic loading

Table 32. Suspended solids mass removal for the pilot scale field filters during the equal loading period of 1975 (Phase IV).

Loading Rate[a]	Mass Removed (lbs/acre)			
	Effective Size of Filter Sand			Total
	0.72 mm	0.40 mm	0.17 mm	
4 mgad (37,415.71 m^3/h-d)	12,891	855	2,082	15,828
3 mgad (28,061.78 m^3/h-d)	12,437	885	3,008	16,330

[a]The 0.72 mm effective size filters plugged first.

Table 33. Volatile suspended solids mass removal for the pilot scale field filters during the equal loading period of 1975 (Phase IV).

Loading Rate[a]	Mass Removed (lbs/acre)			
	Effective Size of Filter Sand			Total
	0.72 mm	0.40 mm	0.17 mm	
4 mgad (37,415.71 m^3/h-d)	9,491	862	897	11,250
3 mgad (28,061.78 m^3/h-d)	8,686	1,345	1,021	11,052

[a]The 0.72 mm effective size filter plugged first.

Table 34. BOD_5 mass removal for the pilot scale field filters during the equal loading period of 1975 (Phase IV).

Loading Rate[a]	Mass Removed (lbs/acre)			
	Effective Size of Filter Sand			Total
	0.72 mm	0.40 mm	0.17 mm	
4 mgad (37,415.71 m^3/h-d)	1,591	456	1,444	3,491
3 mgad (28,061.78 m^3/h-d)	1,791	953	1,041	3,785

[a]The 0.72 mm effective size filters plugged first.

rates. The 0.72 mm (0.0284 inch) effective size sand filters removed 12,891 lbs/acre (14,449 kg/hectare) of suspended solids at the 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate, and 12,437 lbs/acre (13,940 kg/hectare) of suspended solids at the 3 mgad (28,061.8 m^3/h-d) hydraulic loading rate. For a hydraulic loading rate of 4 mgad (37,415.7 m^3/h-d), 9,491 lbs/acre (10,638 kg/hectare) of volatile suspended solids mass was removed by the 0.72 mm (0.0284 inch) effective size sand filter, and for a hydraulic loading rate of 3 mgad (28,061.8 m^3/h-d), 8,686 lbs/acre (9,736 kg/hectare) of volatile suspended solids mass was removed by the 0.72 mm (0.0284 inch) effective size sand filter. The above mass removals by the 0.72 mm (0.0284 inch) effective size sand filters represent approximately 80 percent of the total suspended solids and volatile suspended solids that were removed by the series operations.

The removal percentage by the 0.72 mm (0.0284 inch) effective size sand filters of suspended solids and volatile suspended solids during Phase IV was greater than the removal percentage by the 0.72 mm (0.0284 inch) effective size sand filters of suspended solids and volatile suspended solids during Phase I. During Phase I the 0.72 mm (0.0284 inch) effective size sand filters experienced some "filter washing" which explains why suspended solids removals were greater during Phase IV. This higher rate of removal during Phase IV could also be attributed to the differing influent populations. Green algae dominated the secondary effluent during Phase IV, while blue-green algae dominated the effluent during Phase I. The overall efficiency of removal for the series system was almost identical for Phases I and IV. A suspended solids and volatile suspended solids removal efficiency of 75 to 85 percent was obtained for the two phases.

The total series mass removal of BOD_5 was 3,491 lbs/acre (3,913 kg/hectare) for the 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate, and 3,785 lbs/acre (4,242 kg/hectare) for the 3 mgad (28,061.8 m^3/h-d) hydraulic loading rate.

During Phase IV, the 0.72 mm (0.0284 inch) effective size sand filter removed slightly more BOD_5 mass than the 0.17 mm (0.0067 inch) effective size sand filter at each hydraulic loading rate. For a hydraulic loading rate of 4 mgad (37,415.7 m^3/h-d), the 0.72 mm (0.0284 inch) effective size sand filter in the series removed 1,591 lbs/acre (1,783 kg/hectare) of BOD_5 while the 0.17 mm (0.0067 inch) effective size sand filter in the series removed 1,444 lbs/acre (1,619 kg/hectare) of BOD_5. For the 3 mgad (28,061.8 m^3/h-d) hydraulic loading rate, the 0.72 mm (0.0284 inch) effective size sand filter removed 1,791 lbs/acre (2,007 kg/hectare) of BOD_5 and the 0.17 mm (0.0067 inch) effective size sand filter removed 1,041 lbs/acre (1,167 kg/hectare) of BOD_5 mass. During Phase I, the 0.17 mm (0.0067 inch) effective size sand filter removed the majority of the BOD_5, however the 0.72 mm (0.0284 inch) effective size sand filter removed more BOD_5 during the warmer months of the experimental period (Phase I).

It is worthwhile to mention that the total pollution mass removals using hydraulic loading rates of 3 mgad (28,061.8 m^3/h-d) and 4 mgad (37,415.7 m^3/h-d) (Phase IV) were much less than pollutant removals using hydraulic loading rates ranging from 0.5 mgad (4,676.96 m^3/h-d) to 1.5 mgad (14,030.9 m^3/h-d) (Phase I). It must be remembered that the series operations used during Phase I did not plug. More pollutant mass removal was accomplished at the lower hydraulic loading rate (1.5 mgad; 14,030.9 m^3/h-d) because the lower hydraulic loading rate gave the filters more time to aerobically recover between daily applications. The lower hydraulic loading rates also meant lower daily BOD_5 and suspended solids loading to the filters. The intermittent sand filters biological population could decompose the organics in the "schmutzdecke" (filtering skin) more efficiently at the lower hydraulic and organic (BOD_5 and suspended solids) loading rates.

Length of filter runs

The length of the two filter runs during Phase IV are tabulated in Table 35. The 4 mgad (37,415.7 m^3/h-d) series operation plugged in 21 days, and the 3 mgad (28,061.8 m^3/h-d) series operation plugged in 26 days. The 0.72 mm (0.0284 inch) effective size sand filter plugged first in both cases.

It appeared at the start of the filter runs that the 0.17 mm 0.0067 inch) effective size sand filters would plug first. The loading time had to be continually increased to accommodate the 0.17 mm (0.0067 inch) effective size sand filters. The 0.17 mm (0.0067 inch) effective size sand filters seemed to lose head until reaching a semi-constant head loss. The 0.72 mm (0.0284 inch) effective size sand filters lost head slowly at first and then later in the study, the

Table 35. Length of filter runs for the pilot scale field filters during the equal loading period of 1975 (Phase IV).

Hydraulic Loading Rate	Length of Filter Runs (Days)	Filter in Series Which Plugged
4 mgad (37,415.7 m^3/h-d)	21	0.72 mm
3 mgad (28,061.8 m^3/h-d)	27	0.72 mm

0.72 mm (0.0284 inch) effective size sand filters lost head rapidly until plugging occurred. It may be reasonable to assume that the 0.17 mm (0.0067 inch) effective size sand filters were close to plugging at the time the 0.72 mm (0.0284 inch) effective size sand filters plugged.

Final Sand Analysis

The sand in the pilot-scale field filters was analyzed for effective size and uniformity coefficient at the end of operations in September 1975. The sieve analysis indicated that the 0.72 mm (0.0284 inch) and 0.17 mm (0.0158 inch) sand had not changed significantly over the two years of operation. However the 0.40 mm (0.0158 inch) effective size sand was found to have an actual effective size of 0.33 mm (0.013 inch) and a uniformity coefficient of 2.2. At the start of operations the 0.40 mm (0.0158 inch) effective size sand had an actual effective size of 0.38 mm (0.015 inch) and a uniformity coefficient of 1.95.

All three 0.40 mm (0.0158 inch) effective size sand filters were sampled and the results were almost identical for each. One of the 0.40 mm (0.0158 inch) effective size sand filters had not been used during the 1975 season and its size characteristics were almost identical to the other two field 0.40 mm (0.0158 inch) effective size sand filters.

The reason for the discrepancy is unknown. Perhaps the 0.40 mm (0.0158 inch) effective size sand filters trapped inert solids washed from the 0.72 mm (0.0284 inch) effective size sand filter in each series until the 0.40 mm (0.0158 inch) effective size sand filters reached an "equilibrium" point among the particles of sand in the filters. Another plausible reason is that the overall average effective size of the sand was actually about 0.33 mm (0.013 inch) instead of 0.40 mm (0.0158 inch). Analysis on the new sand in 1974 indicated an effective size range of about 0.33 mm (0.013 inch) to about 0.47 mm (0.0185 inch) for different samples, and the actual mean value could have been on the lower side of this range. However Utah Sand and Gravel, who furnished the sand, said that it should average approximately 0.40 mm (0.0158 inch) effective size. In summary, no definite conclusions can be made concerning this phenomena.

Performance Summary

The pilot scale series filter operation during Phase I indicates that an effluent with an average BOD_5 of less than 3.0 mg/l, an average suspended solids concentration of less than 9.0 mg/l, and an average volatile suspended solids concentration of less than 4.0 mg/l can be consistently produced with a hydraulic loading rate of 1.5 mgad (14,030.9 m^3/h-d) and lower. In addition, filter run lengths of 130 days were achieved.

At hydraulic loading rates of between 3 mgad (28,061.8 m^3/h-d) and 4 mgad (37,415.7 m^3/h-d), effluents with an average BOD_5 concentration of less than 3 mg/l, average suspended solids concentration of less than 6 mg/l, and volatile suspended solids concentration of less than 4 mg/l were produced. Length of filter runs were 27 days at a hydraulic loading rate of 3 mgad (28,061.8 m^3/h-d) and 21 days at a hydraulic loading rate of 4 mgad (37,415.7 m^3/h-d).

During Phases II and III, final series filter effluents of less than 10 mg/l BOD_5 concentration and 10 mg/l suspended solids concentration were consistently produced. However the length of the filter runs were shortened considerably at these higher hydraulic loading rates. Filter runs varied from 1 day up to 16 days for hydraulic loading rates ranging from 4 mgad (37,415.7 m^3/h-d) to 16 mgad (149,662.8 m^3/h-d).

Figure 12 shows a summary of the length of filter runs for all phases. The hydraulic loading rate appears to be critical. Laak (1970) described the clogging of soils (including 0.26 mm; 0.01 inch effective size sand) in the filtering of effluent from aerobic and anaerobic unit processes. He noted that a 10 percent increase in hydraulic loading rate could reduce the length of filter runs by 50 percent or more. This appears to be true for a series intermittent sand filtration operation as well. The curve seems to extend asymptotically along the X-axis (length of filter runs) at the lower hydraulic loading rates. It is unfortunate that Phase I could not be extended to determine the length of the filter runs used. Extremely lengthy filter runs may have been obtained.

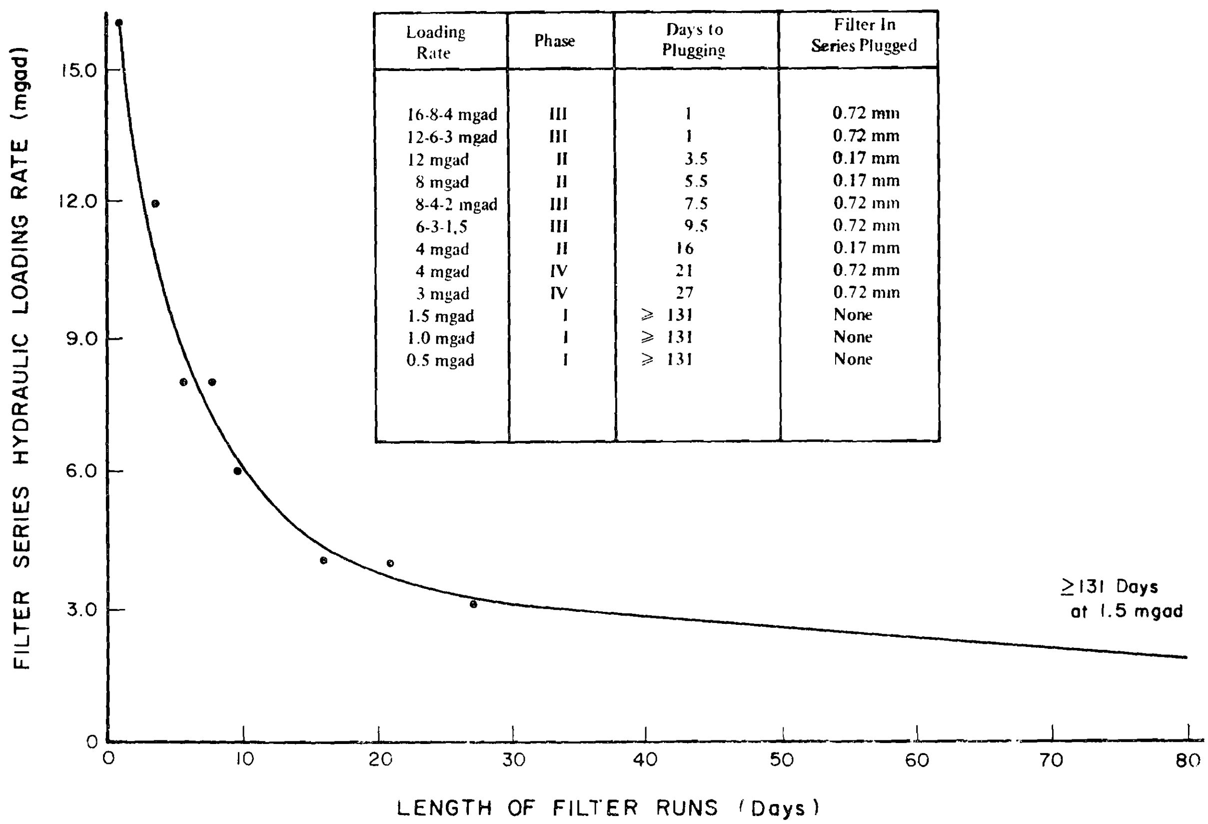
Previous investigators (Reynolds et al., 1974) indicate that a single stage prototype intermittent sand filter operated for 42 days at a 0.2 mgad (1,872 m^3/h-d) hydraulic loading rate and 12 days at a 1.2 mgad (16,837.1 m^3/h-d) hydraulic loading rate for a 0.17 mm (0.0067 inch) effective size sand filter while producing a high quality effluent ($BOD_5 < 20$ mg/l and suspended solids <20 mg/l).

During August, September, and October of 1975, the single stage prototype intermittent sand filtration operation, located at the Logan Municipal Sewage Lagoons was in operation using larger effective size sands (0.72 mm; 0.0284 inch and 0.40 mm; 0.0158 inch). Filip (1975) reported filter runs of 12 days at a 3.0 mgad (28,061.8 m^3/h-d) hydraulic loading rate and 50 days at a 1.5 mgad (14,030.9

m^3/h-d) hydraulic loading rate for the 0.72 mm (0.0284 inch) effective size sand filters. Filip (1975) also reported filter runs of 6 days at a 1.5 mgad (14,030.9 m^3/h-d) hydraulic loading rate and 45 days at a 1.0 mgad (9,353.9 m^3/h-d) hydraulic loading rate for the 0.40 mm (0.0158 inch) effective size sand filters. Hbwever these single stage filters using the larger sized sands tend to have a lower quality effluent.

From the information available from the single stage prototype intermittent sand filter operation, it appears that a prototype series opeation would have filter runs approximately one-half the length of the pilot scale series operation. However Reynolds et al. (1974) and Harris et al. (1975) established that the prototype single stage filter runs were shortened considerably by algae growth in the standing water above the filter. This algae growth also led to the raising of the pH in the standing water. Eventually, the calcium carbonate in the standing water precipitated out forming a "plaster like" surface on the filter which rapidly became impermeable. The pilot scale filters used for the series intermittent sand filtration experiment were shaded from the sun by the high freeboard on the filters. Sunlight was unavailable for algae reproduction. Perhaps if the prototype filters had been loaded at night or had been covered, similar filter runs might have occurred. An algacide used daily might also have increased the prototype scale filter runs. However algacides may be detrimental to the bacterial population in the filters.

A series intermittent sand filtration operation is capable of producing the same quality effluent as a single stage operation using small effective size sands. However length of filter runs appears to be greatly increased by a series operation, and much higher hydraulic loading rates also appear to be possible with a series operation.



Loading Rate	Phase	Days to Plugging	Filter In Series Plugged
16-8-4 mgad	III	1	0.72 mm
12-6-3 mgad	III	1	0.72 mm
12 mgad	II	3.5	0.17 mm
8 mgad	II	5.5	0.17 mm
8-4-2 mgad	III	7.5	0.72 mm
6-3-1.5	III	9.5	0.72 mm
4 mgad	II	16	0.17 mm
4 mgad	IV	21	0.72 mm
3 mgad	IV	27	0.72 mm
1.5 mgad	I	⩾ 131	None
1.0 mgad	I	⩾ 131	None
0.5 mgad	I	⩾ 131	None

Figure 12. Composite filter plugging chart for all phases.

DESIGN CRITERIA AND ECONOMIC ANALYSIS

General

Based upon the data presented in this paper, tentative design criteria have been established. It should be pointed out that the data in this paper were collected using a pilot scale filtration system, and therefore, caution should be exercised in scaling up to a prototype system.

Hydraulic Loading Rates

Because intermittent sand filtration is a biological treatment method, future research may dictate that loading rates be expressed in terms of BOD_5 and suspended solids loading rather than hydraulic loading rates. However for the purpose of this study, a hydraulic loading rate will be used for design purposes.

For a series intermittent sand filter system, a hydraulic loading rate of 1.0 mgad (9,353.9 m^3/h-d) to 1.5 mgad (14,030.9 m^3/h-d) is probably acceptable for most applications. A large influent suspended solids concentration may require lower hydraulic loading rates, and a small influent suspended solids concentration may be loaded at slightly higher rates. In addition, it may be advisable to adjust the filter's hydraulic loading rate to the characteristics of the lagoon effluent at different times of the year.

Filter Sand

If three filters are used in a series intermittent sand filter, the first filter sand in the series should have an effective size of 0.65 to 0.75 mm (0.026 to 0.030 inch), while the second filter sand in the series should have an effective size of 0.35 mm to 0.45 mm (0.014 to 0.018 inch). The filter sand in the final filter should have an effective size of 0.15 mm to 0.25 mm (0.006 to 0.010 inch). The uniformity coefficient for all sands may range from 2 to 10.

If two filters are used in the series operation, the first filter should have a sand with an effective size between 0.45 to 0.65 mm (0.018 to 0.0256 inch). The final filter in the series should have 0.15 mm to 0.25 mm (0.006 to 0.010 inch) effective size sand.

Experience indicates that the smaller effective size sands ($\leqslant$ 0.25 mm; $\leqslant$ 0.010 inch) are generally available as pit run concrete sand. However the larger effective size sands ($\geqslant$ 0.30 mm; $\geqslant$ 0.012 inch) will require special processing.

Filter Bed

The filter bed should consist of 36 inches (91.4 cm) of filter sand and underlain with 1 foot (30.5 cm) of 1/2 to 1 inch (1.3 to 2.5 cm) diameter concrete aggregate. The filter drain pipes should be embedded in the gravel. This gravel should be thoroughly washed to shorten the "wash out" period required during the initial filter start-up period.

Previous reports indicate that a satisfactory effluent may be produced with an 18 inch (45.7 cm) sand depth (Furman et al., 1955); however, a minimum of 24 inches (61.0 cm) of filter sand should be maintained at all times. An initial sand depth of 36 inches (91.4 cm) would allow sand replacement every six to twelve months.

No single filter should be larger than 3/4 to 1 acre (0.030 to 0.405 hectares) in size as recommended by Metcalf and Eddy (1935) and Steel (1960) in order to facilitate flexible operation and filter cleaning. Two or more filters should be provided for each effective size sand.

Sand Cleaning

Provisions should be made for either disposal or cleaning and replacement of clogged filter sand. For large installations economic considerations indicate that sand should be cleaned rather than disposed of in landfills. Cleaning may be accomplished by hydraulic "backwash" of the sand in a conventional sand washer. The organic matter washed from the sand could be recycled to the primary cell of the lagoon system. Once the filter sand has been cleaned, it may be replaced on the filters.

The sand washing equipment need not be large or elaborate. Rather, the sand washer should be sized to accommodate the sand washing over several days at a time. This will provide a more economical sand washing operation.

Embankment

Filter embankment construction should be similar to that used in the lagoon system. Embankments are usually designed with side slopes from 6:1 to 2:1 with 3:1 being the most common. Embankment top width should be at least 10 feet (3.05 m) and provide a 12 inch (0.305 cm) thick all-weather gravel road. Road surfaces should be crowned to assure rainwater runoff and minimum erosion.

The interior embankment should be impervious to prevent excessive water seepage. Most states have a minimum seepage loss requirement. Interior slopes should also be designed to prevent erosion due to wave action. Erosion protection can be provided by cobbles, broken or cast-in-place concrete, wooden bulkheads, or asphalt strips. Emphasis should be placed on shoreline control and reduction of aquatic weed growths. Exterior slopes should be seeded with native or cultured grasses to enhance the aesthetic nature of the installation and to prevent erosion.

A ramp should be provided in each filter for easy access by mechanical cleaning equipment. These ramps should be paved to prevent erosion and weed growths. In addition, each filter should be provided with boat launch facilities for routine maintenance of the system.

Embankments should provide at least 3 feet (0.91 m) of head on the filter. In addition, 2 to 3 feet (0.61 to 0.91 m) of freeboard should be provided to prevent wave action from washing over the dike.

Influent System

The influent system may be either gravity flow or utilize a pump depending on the geographical nature of the site. Influent lines should be designed to accomplish complete daily loading in less than 6 hours. Influent velocities should be sufficient to prevent settling of solids in the line.

The influent distribution system need not be elaborate. Simple channels which overflow at regular intervals across the filter bed are sufficient. Discharge velocity from the channels onto the filter sand should be small enough to prevent serious sand erosion. Splash aprons may also be used to dissipate the energy from the wastewater flow. Hansen (1910) recommended that one discharge point should not serve more than 2500 square feet (232.3 m^2) of filter surface.

The influent system should be automated using sump pumps or automatic rotating syphons and dosage tanks so that the system can be used day or night. Manual overrides should be provided in case of power failure.

Drain System

The filter drain system may consist of perforated plastic drain pipe (similar to that used for irrigation drainage) placed at regular intervals across the filter. The drain pipe should be placed with the bottom gravel layer in the filter to collect the water as it infiltrates through the sand. Slopes on the drain pipes should be sufficient to produce "scour" velocity so that these pipes will be self-cleaning.

Flexibility

Flexibility should be designed into the system so that each filter may operate independently of the other. Series filter operations should be designed to permit single stage operation if necessary. Spare pumps and metering systems should be provided. In general, the same degree of flexibility should be provided for an intermittent sand filter system as that found in a conventional treatment plant.

Operation Modes

Length of filter runs may be increased if the filter influent suspended solids are low. Therefore, it may be advantageous to hold lagoon effluents during high algal peak periods and discharge during periods of low algal growths (i.e., early spring and late fall). These periods may not result in a high quality of lagoon effluent in terms of BOD, but the filter is capable of significantly reducing the BOD. Therefore, the traditional practice of lagoon discharge during summer months may be modified to discharging during spring and fall months and holding the effluent during the summer and winter months. With such a discharge scheme, higher hydraulic loading rates may be possible. This could result in a considerable reduction in capital costs.

If it is necessary to hold the pond effluent during the winter or for some other period of time, it will be necessary to design the filter operation for a higher hydraulic flow rate. For example it will be necessary to remove the total volume of wastewater produced by the community in nine months instead of the normal 12. Therefore, the design flow rate for the filters would be 1.33 times greater than the discharge from the community. Capital costs would increase approximately in proportion to the increase in flow rate and operating costs would remain essentially constant.

The modification of an existing multi-cell lagoon system to provide for multiple storage periods would provide only a modification in operating procedure. The intermittent sand filters would have to be designed to handle flow rates in proportion to the storage volume provided.

During the winter, many lagoon systems do not discharge effluents; however if they do, studies have shown that winter operation of intermittent sand filters is no problem (Harris et al., 1975). Preventive maintenance, such as furrowing or staking the bed of sand, is needed to prevent the ice layer from settling on the sand surface and solidly freezing the surface.

Loading the filters at night rather than during the day has also increased the length of filter runs. This is due to the reduction of algal growth on the filter itself during dark hours. Covering the filter or using an algacide immediately before the daily loading may be used to stop the algal growth. If an algacide is used, it should not be detrimental to the biological population in the filter.

Economic Considerations of Series Intermittent Sand Filtration

A general approach was taken in the preparation of the cost estimates 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. However, the estimates of operational expenses are based upon experience to date with the prototype units (Harris et al., 1975; Reynolds et al., 1974).

The in-place total construction cost estimates were prepared through the aid of a local consulting engineering firm and construction companies. The construction cost estimates were obtained in November 1974 and were updated by the Engineering News Record (1975) Cost Indexes to December 1975 values.

Economic evaluations of series operations using two filters in series and three filters in series are shown in Table 36. The 0.17 mm (0.0067 inch) effective size sand was locally available, and prices were obtained to prepare 0.40 mm (0.0158 inch) and 0.72 mm (0.0284 inch) effective size sand that would be used in the series operations. The specially prepared media (0.40 mm; 0.0158 inch and 0.72 mm; 0.0284 inch effective size) was found to be only 1.2 times more costly than the locally available media in bulk quantities.

The construction costs determined in Estimates I and II (see Tables B-1 and B-2 in Appendix B for details), reflect: 1) a paired series filter operation utilizing three filters (0.72 mm; 0.0284 inch–0.40 mm; 0.0158 inch–0.17 mm; 0.0067 inch effective size) in series and designed at a 1.5 mgad (14,030.9 m^3/h-d) hydraulic loading rate; and 2) a paired series filter operation utilizing two filters (0.72 mm; 0.0284 inch or 0.40 mm; 0.0158 inch–0.17 mm; 0.0067 inch effective size) in series at a 1.0 mgad (9,353.9 m^3/h-d) hydraulic loading rate. The pumps were designed large enough to apply the influent to the filters in approximately three hours. It was assumed that in a municipal construction project such as this one, at least 75 percent of the construction cost would be funded by federal construction grants. Also, costs without federal funding are reported.

The construction costs shown in Table 36 for Estimates III and IV (see Tables B-3 and B-4 in Appendix B) represent series filter systems utilizing the final cell of a multicell lagoon instead of purchasing additional land. It was assumed that all costs would remain constant with the exception being the purchase of land and the construction of dikes. For a series operation approximately 25 percent of the dike work would be eliminated under Estimates III and IV.

From the present understanding of the operation of effluent polishing intermittent sand filters, a cost ranging between \$39 to \$89 per million gallons of filtrate can be assumed to be representative of this process. The most economical alternative seemed to be a series operation utilizing two filters in series and built in an existing cell of the lagoon system. The cost of construction ranged from \$54,928 per acre to \$63,100 per acre without federal assistance.

A comparison of alternative methods to meet new water quality standards cannot be made because of the unavailability of cost data for other processes. In 1972 Middlebrooks et al. (1973) compared the costs of several likely candidate processes to polish wastewater lagoon effluents and the intermittent sand filter was found to be very competitive. It is likely that all prices have increased proportionally over the past three years, and it would be expected that the relative position of the intermittent sand filter has remained constant and favorable.

Table 36. **Estimated cost per million gallons of filtrate produced by various designs of an effluent polishing series intermittent sand filtration operation.**

Application Conditions	Design Flow Rate	Design Hydraulic Loading Rate	Effective Sand Size			Cost With Federal Assistance $/10^6 Gallons	Cost Without Federal Assistance $/10^6 Gallons
			First	Second	Third		
Estimate I: Three filter series operation	0.5 mgd (4,676.96 m^3/h-d)	1.5 mgad (14,030.9 m^3/h-d)	0.72 mm	0.40 mm	0.17 mm	$44	$89
Estimate II: Two filter series operation	0.5 mgd (4,676.96 m^3/h-d)	1.0 mgad (9,353.9 m^3/h-d)	0.72 mm or 0.40 mm	0.17 mm	-	$41	$82
Estimate III: Modification of lagoon using three filters in series	0.5 mgd (4,676.96 m^3/h-d)	1.5 mgad (14,030.9 m^3/h-d)	0.72 mm	0.40 mm	0.17 mm	$43	$84
Estimate IV: Modification of lagoon using two filters in series	0.5 mgd (4,676.96 m^3/h-d)	1.0 mgad (9,353.9 m^3/h-d)	0.72 mm or 0.40 mm	0.17 mm	-	$39	$78

SUMMARY AND CONCLUSIONS

This study was designed to evaluate the performance of series intermittent sand filters employed to upgrade wastewater lagoon effluents to meet stringent discharge requirements. In addition, a comparison between the performance of single stage intermittent sand filters and series intermittent sand filtration was performed. The results of the study were combined with previous research to develop design criteria and the cost of construction and operation of a series intermittent sand filter installation.

A pilot scale series intermittent sand filter operation was operated for one long phase during 1974 and two shorter phases of 1975. A short laboratory scale series operation was also analyzed during 1975. BOD_5, suspended solids, volatile suspended solids, pH, temperature, and dissolved oxygen were the major parameters measured.

It appears that the use of series intermittent sand filtration can substantially increase the length of filter runs while producing a high quality effluent capable of meeting future effluent standards. Higher hydraulic loading rates also seem possible with the use of series intermittent sand filtration.

Based upon the findings in this study, the following conclusions can be made:

1. Hydraulic loading rate had no effect upon the BOD_5 removal for the 0.72 mm (0.0284 inch) and 0.40 mm (0.0158 inch) effective size sand filters in the series operation during Phase I. The final series BOD_5 effluent from the 0.17 mm (0.0067 inch) effective size sand filters was significantly higher at the 1.5 mgad (14,030.9 m^3/h-d) hydraulic loading rate than at the 1.0 mgad (9,353.9 m^3/h-d) and 0.5 mgad (4,676.96 m^3/h-d) hydraulic loading rate.
2. At an average influent BOD_5 concentration of 10.7 mg/l, the final series effluents ranged from 1.8 mg/l at the 0.5 mgad (4,676.96 m^3/h-d) hydraulic loading rate up to 2.3 mg/l for the 1.5 mgad (14,030.9 m^3/h-d) hydraulic loading rate during Phase I.
3. At the beginning of filter runs, the final series effluent suspended solids concentration was dependent upon the initial influent concentration. As the "schmutzedecke" (filtering layer) built up, final effluent concentration turned independent of influent suspended solids concentration. This was true of volatile suspended solids also.
4. Washout was accomplished faster from the 0.72 mm (0.0284 inch) and 0.40 mm (0.0158 inch) effective size sand filters at the 1.5 mgad (14,030.9 m^3/h-d) hydraulic loading rate than at the lower hydraulic loading rates during Phase I.
5. At an average influent suspended solids concentration of 32.4 mg/l, the series operation produced average effluent suspended solids concentrations of less than 9 mg/l during Phase I.
6. At an average influent volatile suspended solids concentration of 21.9 mg/l, the series operation produced average effluent volatile suspended concentrations of less than 3.3 mg/l during Phase I.
7. The pH of the wastewater continually drops as it flows through the filters in each series.
8. The 0.72 mm (0.0284 inch) effective size sand filter in each series removed the majority of the suspended solids and volatile suspended solids from the wastewater.
9. The 0.72 mm (0.0284 inch) effective size sand filter in each series removes slightly more BOD_5 from the wastewater than the 0.17 mm (0.0067 inch) effective size sand filter during the warmer months. During the colder months the 0.72 mm (0.0284 inch) effective size sand filter is very ineffective at BOD_5 removal. Most of the BOD_5 removal is then accomplished by the 0.17 mm (0.0067 inch) effective size sand filter in each series followed closely by the 0.40 mm (0.0158 inch) effective size sand filter.
10. Pilot scale series intermittent sand filter run lengths of 130 days or greater were accomplished at hydraulic loading rates ranging from 0.5 mgad (4,676.96 m^3/h-d) up to 1.5 mgad (14,030.9 m^3/h-d).
11. Overloading produced detrimental effects to the 0.17 mm (0.0067 inch) effective size sand filter effluent, and length of filter runs during

the laboratory scale series intermittent sand filtration study (Phase II) were significantly shortened.

12. The 0.17 mm (0.0067 inch) effective size sand filter plugged first in the series operation during Phase II. The average length of the filter runs during the laboratory scale phase ranged from 3.5 days for a 12 mgad (112,247.1 m^3/h-d) hydraulic loading rate up to 16 days for a 4 mgad (37,415.7 m^3/h-d) hydraulic loading rate.
13. During the decreasing loading phase (Phase III) the 0.72 mm (0.0284 inch) effective size sand filter plugged first for all hydraulic loading rates. The average length of the filter runs ranged from 1 day for a 16-8-4 mgad (149,662.8 - 74,831.4 - 37,415.7 m^3/h-d) hydraulic loading rate up to 9.5 days for a 6-3-1.5 mgad (56,123.6 - 28,061.8 - 14,030.9 m^3/h-d) hydraulic loading rate.
14. At an average influent suspended solids concentration of 28.4 mg/l, the series operation produced average effluent suspended solids concentration of less than 6 mg/l during the equal loading period of 1975 (Phase IV). No "filter washing" of inert fines took place during Phase IV.
15. At hydraulic loading rates of 3 mgad (28,061.8 m^3/h-d) and 4 mgad (37,415.7 m^3/h-d), pilot scale series intermittent sand filters operated for 26 and 21 days respectively before plugging occurred. The 0.72 mm (0.0284 inch) effective size sand filters plugged first for the two hydraulic loading rates of Phase IV.
16. If operated and loaded properly the use of a series intermittent sand filtration operation could substantially increase the length of the filter runs while producing a high quality effluent capable of meeting future standards.
17. It appears that a series intermittent sand filtration operation used to upgrade wastewater lagoon effluents can be constructed and operated at costs ranging from \$39 to \$89 per million gallons of filtrate at December 1975 cost figures.

RECOMMENDATIONS FOR FUTURE STUDY

1. A two filter series operation may be more practical and economical than a three filter series operation. A prototype series intermittent sand filtration operation should be designed and operated using two filters in series. A 0.72 mm (0.0284 inch) or 0.40 mm (0.0158 inch) effective size sand filter followed by a 0.17 mm (0.0067 inch) effective size sand filter would be the most practical design.
2. A study should be instigated to determine the type and size of algae which appear in the effluent from each filter in a series operation.
3. A detailed study is needed to investigate the actual filtering and clogging mechanisms of intermittemt sand filters used to polish wastewater lagoon effluents. The possibility of penetration of algae into the filters should be looked at closely, especially under high head conditions.
4. Another study is needed to determine the most economical method of scraping, reconditioning, and washing the sand from the intermittent sand filters after plugging has occurred.

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APPENDIXES

Appendix A

Tabulation Results of all Phases

Table A-1. Pilot scale field filters BOD_5 results during 1974 (Phase I).

Sample Date	Applied BOD mg/l	4,676.96 m^3/h-d 0.5 mgad			9,353.9 m^3/h-d 1.0 mgad			14,030.9 m^3/h-d 1.5 mgad		
		0.72 mm mg/l	0.40 mm mg/l	0.17 mm mg/l	0.72 mm mg/l	0.40 mm mg/l	0.17 mm mg/l	0.72 mm mg/l	0.40 mm mg/l	0.17 mm mg/l
7/26	6.6	7.9	5.6	3.1	7.8	6.4	1.4	8.1	5.0	3.1
7/31	14.3	7.7	10.2	1.9	8.3	5.4	2.3	6.5	5.9	3.6
July Ave.	10.5	7.8	7.9	2.5	8.0	5.9	1.8	7.3	5.5	3.3
Stand. Dev.	5.4	0.1	3.2	0.9	0.4	0.8	0.7	1.1	0.7	0.3
8/3	6.6	5.4	6.4	1.3	4.4	5.5	3.3	4.1	4.6	3.6
8/7	5.9	8.2	7.5	3.4	8.3	5.7	4.2	6.3	6.8	4.1
8/10	11.5	5.8	5.2	1.3	6.3	4.4	2.5	4.9	4.8	3.2
8/14	12.9	4.7	4.1	2.1	5.2	4.7	2.0	5.9	5.8	3.1
8/21	6.3	4.7	4.4	2.3	3.2	3.9	3.1	4.2	3.2	2.7
8/29	11.5	7.3	4.4	2.6	7.7	5.2	2.6	4.8	4.9	2.8
Aug. Ave.	9.1	6.1	5.1	2.3	5.8	4.9	3.0	5.0	5.0	3.2
Stand Dev.	3.2	1.4	1.4	0.8	2.0	0.7	0.8	0.9	1.2	0.5
9/4	-	-	-	-	-	-	-	-	-	-
9/11	6.9	5.1	4.6	2.6	4.2	4.4	2.9	4.1	4.0	2.1
9/18	6.4	3.5	3.0	1.8	3.1	2.8	2.2	3.2	3.0	1.9
9/26	4.1	3.7	2.1	1.3	4.0	2.7	1.5	4.2	3.0	1.8
Sept. Ave.	5.8	4.1	3.2	1.9	3.8	3.3	2.2	3.8	3.3	1.9
Stand. Dev.	1.5	0.8	1.3	0.7	0.6	0.9	0.7	0.6	0.6	0.2
10/3	6.0	4.8	3.4	0.8	4.3	3.5	1.3	4.8	4.3	3.0
10/10	7.8	6.9	4.9	1.4	6.8	4.7	1.2	6.4	5.9	1.1
10/17	10.4	9.1	2.8	0.6	8.7	4.3	1.1	8.8	5.5	1.3
10/24	24.0	21.2	6.4	1.5	18.9	9.3	3.3	23.9	12.5	2.3
10/31	23.4	20.9	6.3	1.7	20.4	12.6	1.7	20.6	12.8	1.6
Oct. Ave.	14.3	12.6	4.8	1.2	11.8	6.9	1.7	12.9	8.2	1.8
Stand. Dev.	8.7	7.9	1.6	0.5	7.3	3.9	0.9	8.8	4.1	0.8
11/7	12.0	15.4	4.9	1.4	10.6	6.6	1.0	15.1	6.7	1.4
11/14	21.9	16.8	5.6	0.7	17.8	6.2	1.7	18.8	10.9	0.6
11/21	8.7	11.0	3.4	1.0	7.9	4.6	1.1	7.7	5.4	1.2
11/26	13.3	8.7	5.4	1.0	10.0	5.9	1.1	9.8	5.7	1.0
Nov. Ave.	14.0	13.0	4.8	1.0	11.6	5.8	1.2	12.9	7.2	1.1
Stand. Dev.	5.6	3.8	1.0	0.3	4.3	0.9	0.4	5.0	2.5	0.3
Overall Monthly Ave.	10.7	8.7	5.2	1.8	8.2	5.4	2.0	8.4	5.8	2.3

Table A-2. Pilot scale field filters BOD_5 total percent removals during 1974 (Phase I).

Sample Date	4,676.96 m^3/h-d 0.5 mgad			9,353.9 m^3/h-d 1.0 mgad			14,030.9 m^3/h-d 1.5 mgad		
	0.72 mm % Rem.	0.40 mm % Rem.	0. 17 mm % Rem.	0. 72 mm % Rem.	0.40 mm % Rem.	0.17 mm % Rem.	0.72 mm % Rem.	0.40 mm % Rem.	0.17 mm % Rem.
7/26	-19	15	53	-18	3	79	-22	25	53
7/31	46	29	87	42	63	84	55	59	75
July Ave. Removal	25	24	76	23	44	82	30	48	68
8/3	19	3	80	34	16	50	37	30	46
8/7	-39	-28	42	-41	3	28	- 8	-15	30
8/10	50	55	89	45	62	78	58	59	73
8/14	64	68	84	60	63	84	54	55	76
8/21	25	30	64	50	38	51	33	49	57
8/29	36	62	78	34	55	77	58	57	76
Aug. Ave. Removal	33	44	74	36	46	68	45	45	65
9/4	-	-	-	-	-	-	-	-	-
9/11	27	33	62	39	37	58	40	42	70
9/18	45	53	72	52	56	66	51	53	71
9/26	9	49	56	2	34	63	- 3	26	56
Sept. Ave. Removal	29	44	68	35	43	62	34	43	67
10/3	21	43	87	29	42	79	20	29	50
10/10	12	37	83	13	40	85	18	25	87
10/17	13	74	94	17	59	90	16	47	88
10/24	12	74	94	21	61	86	- 4	48	90
10/31	11	73	92	13	46	93	12	45	93
Oct. Ave. Removal	12	67	92	18	52	88	10	43	87
11/7	-28	60	88	12	45	92	-26	44	88
11/14	23	74	97	19	72	92	14	50	93
11/21	-26	61	89	10	47	87	12	38	86
11/26	35	59	93	25	56	92	26	58	93
Nov. Ave. Removal	7	66	93	17	58	91	8	49	92
Overall Monthly Ave. Removal	19	52	83	24	50	82	22	46	79

Table A-3. Pilot scale field filters suspended solids results for 1974 (Phase I).

Sample Date	Applied Suspended Solids mg/l	4,676.96 m^3/h-d 0.5 mgad			9,353.9 m^3/h-d 1.0 mgad			14,030.9 m^3/h-d 1.5 mgad		
		0.72 mm mg/l	0.40 mm mg/l	0.17 mm mg/l	0.72 mm mg/l	0.40 mm mg/l	0.17 mm mg/l	0.72 mm mg/l	0.40 mm mg/l	0.17 mm mg/l
7/26	12.5	22.7	23.2	6.8	26.0	39.6	15.5	12.9	37.3	10.2
7/31	16.8	8.8	26.5	7.7	7.5	8.8	4.2	6.0	5.6	3.5
July Ave.	14.7	15.8	24.9	7.3	16.8	24.2	9.8	9.5	21.5	6.8
Stand. Dev.	3.0	9.9	2.3	0.6	13.1	21.8	8.0	4.9	22.4	4.8
8/3	-	18.4	32.7	5.9	19.5	16.5	5.8	13.3	13.2	7.3
8/7	52.6	37.0	35.5	9.7	35.7	30.0	14.3	29.4	34.8	12.8
8/10	51.9	29.6	48.9	16.7	30.3	25.3	30.4	26.9	34.9	19.1
8/12	61.5	39.9	68.2	48.6	37.6	29.6	24.1	27.7	26.9	24.7
8/14	55.7	29.4	54.7	19.7	29.8	24.1	22.9	28.4	24.6	19.1
8/21	56.9	24.1	51.4	12.2	31.6	27.1	14.2	25.6	20.1	13.5
8/29	69.4	28.7	31.2	16.5	36.1	25.2	18.6	29.9	24.2	15.2
Aug. Ave.	58.0	29.6	46.1	18.5	31.5	25.4	18.6	25.9	25.5	16.0
Stand. Dev.	6.6	7.3	13.6	14.1	6.1	4.5	8.1	5.8	7.7	5.6
9/4	50.5	28.1	19.1	18.2	36.9	31.9	12.4	33.1	28.7	9.6
9/11	30.8	15.6	40.3	7.2	14.5	12.6	5.0	11.1	9.4	5.9
9/18	14.0	8.5	8.6	3.6	8.7	7.2	3.9	11.4	5.7	4.3
9/26	22.7	13.6	17.0	5.0	15.0	5.3	1.9	8.8	4.6	2.4
Sept. Ave.	29.5	16.4	22.7	8.5	18.8	14.2	5.8	16.1	12.1	5.6
Stand. Dev.	15.6	8.3	13.5	6.7	12.4	12.2	4.6	11.4	11.3	3.0
10/3	16.5	6.8	5.7	5.8	7.2	4.4	1.9	11.7	6.4	1.9
10/10	23.2	12.0	9.4	11.1	11.5	5.8	4.7	10.6	5.4	4.3
10/17	28.9	11.5	10.5	3.6	10.0	8.6	1.5	13.7	8.8	2.5
10/24	43.5	23.5	8.7	3.5	18.9	9.3	3.3	22.7	13.0	1.3
10/31	33.3	16.4	8.0	3.4	19.7	12.3	1.3	20.4	12.6	1.4
Oct. Ave.	29.1	14.0	8.5	5.5	13.4	8.1	2.5	15.8	9.2	2.3
Stand. Dev.	10.3	6.0	1.8	3.3	5.5	3.1	1.4	5.4	3.5	1.2
11/7	26.1	12.3	4.7	3.2	12.3	6.2	1.3	11.2	5.8	0.8
11/14	40.8	-	-	-	-	-	-	-	-	-
11/21	22.7	9.8	4.9	2.4	10.5	4.4	3.0	9.6	3.4	1.7
11/26	32.8	14.6	5.6	3.6	16.4	7.9	2.4	14.9	6.3	1.2
Nov. Ave.	30.6	12.2	5.1	3.1	13.1	6.2	2.2	11.9	5.1	1.2
Stand. Dev.	5.1	2.4	0.5	0.6	3.0	1.8	0.9	2.7	1.6	0.5
Overall Monthly Ave.	32.4	17.6	21.4	8.6	18.7	15.6	7.8	15.8	14.7	6.4

Table A-4. Pilot scale field filters suspended solids total percent removal for 1974 (Phase I).

Sample Date	4,676.96 m^3/h-d 0.5 mgad			9,353.9 m^3/h-d 1.0 mgad			14,030.9 m^3/h-d 1.5 mgad		
	0.72 mm % Rem.	0.40 mm % Rem.	0.17 mm % Rem.	0.72 mm % Rem.	0.40 mm % Rem.	0.17 mm % Rem.	0.72 mm % Rem.	0.40 mm % Rem.	0.17 mm % Rem.
7/26	-81	-85	46	-108	-216	-23	- 3	-198	19
7/31	48	-58	54	55	48	75	64	67	80
July Ave. Removal	- 7	-70	51	-14	-65	33	36	-47	53
8/3	-	-	-	-	-	-	-	-	-
8/7	30	33	82	32	43	73	44	34	76
8/10	43	6	68	42	51	41	48	33	75
8/12	35	-11	21	39	52	61	55	56	60
8/14	47	2	65	47	57	59	49	56	66
8/21	58	10	79	45	52	75	55	65	76
8/29	59	55	76	48	64	73	57	65	78
Aug. Ave. Removal	49	21	68	46	56	68	55	56	73
9/4	44	62	64	27	37	75	34	43	81
9/11	49	-31	77	53	59	84	64	70	81
9/18	40	39	74	38	49	72	19	60	69
9/26	40	25	78	34	77	92	61	80	89
Sept. Ave. Removal	44	23	71	36	52	80	46	59	81
10/3	59	66	65	56	73	89	29	61	88
10/10	48	59	52	50	75	80	54	77	82
10/17	60	64	88	65	70	95	53	70	92
10/24	46	80	92	57	79	92	48	70	97
10/31	51	76	90	41	63	96	39	62	96
Oct. Ave. Removal	52	71	81	54	72	91	46	68	92
11/7	53	82	88	53	76	95	57	78	97
11/14	-	-	-	-	-	-	-	-	-
11/21	57	78	89	54	81	87	58	85	92
11/26	56	83	89	50	76	93	55	81	97
Nov. Ave. Removal	60	84	90	57	80	93	61	83	96
Overall Monthly Ave. Removal	46	34	74	42	52	76	51	55	80

Table A-5. Pilot scale field filters volatile suspended solids results for 1974 (Phase I).

Sample Date	Applied Volatile Suspended Solids mg/l	4,676.96 m^3/h-d 0.5 mgad			9,353.9 m^3/h-d 1.0 mgad			14,030.9 m^3/h-d 1.5 mgad		
		0.72 mm mg/l	0.40 mm mg/l	0.17 mm mg/l	0.72 mm mg/l	0.40 mm mg/l	0.17 mm mg/l	0.72 mm mg/l	0.40 mm mg/l	0.17 mm mg/l
7/26	7.3	4.2	3.3	1.1	5.5	3.6	1.8	3.3	5.0	2.2
7/31	11.5	2.6	3.1	0.9	2.8	1.7	0.8	2.6	1.6	0.7
July Ave.	9.4	3.4	3.2	1.0	4.1	2.7	1.3	3.0	3.3	1.5
Stand. Dev.	3.0	1.1	0.1	0.1	1.9	1.3	0.7	0.5	2.4	1.1
8/3	-	2.8	4.7	1.0	7.6	4.4	1.5	7.6	5.0	1.9
8/7	34.5	6.9	10.8	3.2	21.6	8.8	8.8	18.7	14.2	8.5
8/10	39.4	18.4	15.6	8.4	30.3	10.2	10.3	13.5	8.1	8.5
8/12	36.5	18.8	18.7	16.2	19.8	14.8	11.4	18.2	13.5	13.4
8/14	44.3	17.7	20.5	13.5	17.9	12.8	14.6	18.4	14.4	13.7
8/21	36.6	15.7	13.5	6.6	13.6	10.0	7.7	13.6	8.9	7.3
8/29	67.6	22.7	20.6	12.6	17.2	23.7	12.9	22.6	22.2	9.6
Aug. Ave.	43.2	14.7	14.9	8.8	18.3	12.1	9.6	16.1	12.3	9.0
Stand. Dev.	12.4	6.2	5.8	5.6	7.0	6.0	4.3	4.9	5.6	4.0
9/4	39.1	27.3	11.8	12.8	23.1	24.2	9.8	29.4	28.7	11.3
9/11	20.8	6.7	12.4	3.6	4.9	5.4	2.9	4.9	2.9	3.3
9/18	8.6	3.1	2.9	1.4	3.0	2.9	1.6	2.8	1.8	2.1
9/26	11.5	3.0	2.9	1.4	3.0	1.7	0.9	2.4	1.5	0.9
Sept. Ave.	20.0	10.0	7.5	4.8	8.5	8.5	3.8	9.9	8.7	4.4
Stand. Dev.	13.7	11.7	5.3	5.4	9.8	10.6	4.1	13.1	13.4	4.7
10/3	6.0	3.7	1.3	0.8	3.1	1.7	0.7	3.5	2.3	0.8
10/10	11.4	6.1	2.8	1.2	5.6	2.8	0.8	4.9	2.9	1.0
10/17	12.5	8.1	3.5	1.1	7.3	4.6	1.0	8.3	5.4	1.0
10/24	33.5	19.0	6.3	1.0	16.6	7.2	0.8	18.3	10.4	0.8
10/31	25.4	14.2	5.3	1.1	16.8	9.3	0.6	15.5	10.6	1.2
Oct. Ave.	17.8	10.2	3.8	1.0	9.9	5.1	0.8	10.1	6.3	1.0
Stand. Dev.	11.3	6.3	2.0	0.2	6.4	3.1	0.2	6.5	4.0	0.2
11/7	18.2	9.9	3.9	1.0	8.9	4.9	0.6	9.3	5.2	0.7
11/14	29.5	-	-	-	-	-	-	-	-	-
11/21	11.5	6.2	2.7	0.7	6.7	2.8	0.5	5.9	2.2	0.5
11/26	18.2	8.5	3.2	1.0	10.1	5.2	0.8	9.6	4.6	0.8
Nov. Ave.	19.4	8.2	3.2	0.9	8.6	4.3	0.7	8.3	4.0	0.7
Stand. Dev.	7.5	1.8	0.6	0.2	1.7	1.3	0.2	2.1	1.6	0.2
Overall Monthly Ave.	21.9	9.3	6.5	3.3	9.9	6.6	3.2	9.5	6.9	3.3

Table A-6. Pilot scale field filters volatile suspended solids total percent removal for 1974 (Phase I).

Sample Date	4,676.96 m^3/h-d 0.5 mgad			9,353.9 m^3/h-d 1.0 mgad			14,030.9 m^3/h-d 1.5 mgad		
	0.72 mm %Rem.	0.40 mm % Rem.	0.17 mm % Rem.	0.72 mm % Rem.	0.40 mm % Rem.	0.17 mm % Rem.	0.72 mm % Rem.	0.40 mm % Rem.	0.17 mm % Rem.
7/26	43	55	85	24	50	75	55	31	70
7/31	77	73	92	76	85	93	77	86	94
July Ave. Removal	64	66	89	56	72	86	68	65	84
8/3	-	-	-	-	-	-	-	-	-
8/7	80	69	91	38	74	75	46	59	75
8/10	53	60	79	23	74	74	66	80	78
8/12	48	49	56	46	59	69	50	63	63
8/14	60	54	70	60	71	67	59	68	69
8/21	57	63	82	63	73	79	63	76	80
8/29	66	70	81	75	65	81	67	67	86
Aug. Ave. Removal	66	65	70	58	72	78	63	72	79
9/4	30	60	67	41	38	75	25	27	71
9/11	68	40	83	77	74	86	77	86	84
9/18	64	67	84	65	66	81	67	79	76
9/26	74	75	88	74	85	92	79	87	92
Sept. Ave. Removal	50	63	76	58	58	81	51	56	78
10/3	38	79	87	48	72	88	42	62	87
10/10	47	76	90	51	76	93	57	75	91
10/17	36	72	91	42	63	92	34	57	92
10/24	43	81	97	51	79	98	45	69	98
10/31	44	79	96	34	63	98	39	58	95
Oct. Ave. Removal	43	79	94	44	71	96	43	65	95
11/7	46	46	95	51	73	97	49	71	96
11/14	-	-	-	-	-	-	-	-	-
11/21	46	77	94	42	76	96	49	81	96
11/26	53	82	95	45	71	96	47	75	96
Nov. Ave. Removal	58	83	95	56	80	97	57	79	97
Overall Monthly Ave. Removal	58	70	85	55	70	85	57	68	85

Table A-7. Pilot scale field filters pH results for 1974 (Phase I).

Sample Date	Applied pH	4,676.96 m^3/h-d 0.5 mgad			9,353.9 m^3/h-d 1.0 mgad			14,030.9 m^3/h-d) 1.5 mgad		
		0.72 mm	0.40 mm	0.17 mm	0.72 mm	0.40 mm	0.17 mm	0.72 mm	0.40 mm	0.17 mm
7/26	8.5	8.4	8.4	8.2	8.4	8.3	8.2	8.4	8.3	8.2
7/31	8.5	8.4	8.2	8.0	8.4	8.3	8.1	8.3	8.3	8.1
July Ave.	8.5	8.4	8.3	8.1	8.4	8.3	8.2	8.4	8.3	8.2
8/3	8.9	8.8	8.6	8.3	8.9	8.7	8.4	8.9	8.7	8.4
8/10	9.2	9.0	8.7	8.3	9.0	8.9	8.4	9.0	8.8	8.3
8/14	9.2	9.1	8.9	8.4	9.1	9.1	8.4	9.1	9.0	8.3
8/21	9.1	9.0	8.8	8.4	9.1	8.9	8.5	9.1	9.0	8.5
8/29	9.3	9.3	8.9	8.5	9.3	9.2	8.6	9.3	9.2	8.5
Aug. Ave.	9.1	9.1	8.6	8.4	9.0	8.9	8.5	9.0	8.9	8.4
9/4	8.8	8.8	8.4	8.0	8.7	8.6	8.0	8.8	8.5	8.2
9/11	8.7	8.6	8.2	7.9	8.6	8.4	8.0	8.7	8.6	8.0
9/18	8.7	8.5	8.3	8.0	8.7	8.7	7.9	8.7	8.6	-
9/26	8.4	8.2	7.9	7.9	8.3	8.3	8.0	8.4	8.4	8.0
Sept. Ave.	8.6	8.4	8.2	7.9	8.6	8.5	8.0	8.6	8.5	8.1
10/3	8.2	8.3	8.0	7.8	8.3	8.0	7.6	8.3	8.2	7.6
10/10	8.3	8.3	8.2	8.1	8.2	8.1	7.9	8.0	8.1	7.7
10/17	8.4	8.4	8.0	7.9	8.6	8.3	7.8	8.5	8.4	7.7
10/24	8.4	8.5	7.9	8.0	8.4	7.9	7.9	8.6	8.1	7.7
10/31	8.2	8.5	7.9	8.0	8.6	8.4	7.9	8.5	8.2	7.7
Oct. Ave.	8.3	8.4	8.0	7.9	8.4	8.1	7.8	8.3	8.2	7.7
11/7	8.2	8.3	8.1	7.9	8.5	8.2	7.8	8.5	8.1	7.7
11/14	8.7	8.4	7.8	7.9	8.4	7.9	7.9	8.6	8.0	7.6
11/21	8.7	8.5	7.9	8.1	8.7	8.2	8.1	8.6	8.1	7.9
11/26	8.3	8.2	7.8	7.8	8.3	8.1	7.9	8.2	7.6	7.6
Nov. Ave.	8.4	8.3	7.9	7.9	8.5	8.1	7.9	8.5	7.9	7.7
Overall Monthly Ave.	8.5	8.5	8.2	8.0	8.5	8.3	8.0	8.5	8.3	7.9

Table A-8. Pilot scale field filters dissolved oxygen results for 1974 (Phase I).

Sample Date	Applied Dissolved Oxygen mg/l	4,676.96 m^3/h-d 0.5 mgad			9,353.9 m^3/h-d 1.0 mgad			14,030.9 m^3/h-d 1.5 mgad		
		0.72 mm mg/l	0.40 mm mg/l	0.17 mm mg/l	0.72 mm mg/l	0.40 mm mg/l	0.17 mm mg/l	0.72 mm mg/l	0.40 mm mg/l	0.17 mm mg/l
7/26	3.5	6.8	6.8	7.0	6.9	6.8	6.8	6.9	7.0	6.9
7/31	3.4	6.5	6.5	6.4	6.5	6.4	6.4	6.5	6.5	6.4
July Ave.	3.5	6.7	6.7	6.7	6.7	6.6	6.6	6.7	6.8	6.7
8/5	-	6.5	6.7	7.1	6.8	6.8	7.0	6.8	6.8	6.7
8/10	12.5	7.4	7.8	7.5	7.3	7.4	7.4	7.2	7.8	7.3
8/14	10.9	8.2	8.1	8.1	8.4	8.1	7.9	8.2	8.2	8.1
8/21	5.9	-	-	-	-	-	-	-	-	-
8/29	10.7	6.9	6.8	6.7	7.0	6.9	6.5	6.9	7.0	6.6
Aug. Ave.	10.0	7.3	7.4	7.4	7.4	7.3	7.2	7.3	7.4	7.2
9/4	12.5	8.2	7.9	8.5	7.8	8.0	8.2	8.2	8.0	8.4
9/11	10.5	7.5	7.6	8.0	7.6	7.8	8.0	7.4	7.6	7.5
9/18	14.7	8.4	8.3	8.5	8.3	8.5	8.5	8.0	8.4	-
9/26	7.1	8.6	8.3	8.6	8.7	8.7	8.3	8.5	8.5	8.0
Sept. Ave.	11.2	8.2	8.1	8.4	8.1	8.3	8.2	8.0	8.1	8.0
10/3	7.2	7.3	7.4	7.2	7.6	7.6	7.3	7.5	7.2	7.4
10/10	5.8	10.4	10.9	8.9	9.3	11.2	9.4	9.1	9.1	7.6
10/17	10.6	7.3	7.6	7.9	7.8	8.1	7.7	7.7	7.7	7.6
10/24	8.1	8.2	8.1	7.0	6.5	6.7	7.9	7.1	6.9	7.3
10/31	8.0	9.0	9.2	9.2	9.4	9.3	8.8	9.1	9.0	7.9
Oct. Ave.	7.9	8.4	8.6	8.0	8.1	8.6	8.2	8.1	8.0	7.6
11/7	8.2	11.7	10.0	7.7	7.6	8.0	7.5	9.1	8.5	7.2
11/14	12.9	7.7	7.5	11.8	12.1	11.7	9.2	6.6	10.3	10.0
11/21	9.8	14.4	12.4	12.2	10.7	12.8	13.4	11.2	9.8	10.6
11/26	11.8	11.2	12.1	12.8	11.2	12.5	12.1	12.1	11.3	11.9
Nov. Ave.	10.7	11.3	10.5	11.1	10.4	11.3	10.5	9.8	10.0	9.9
Overall Monthly Ave.	8.7	8.3	8.2	8.3	8.1	8.4	8.2	9.8	8.1	7.9

Table A-9. Pilot scale field filters temperature results for 1974 (Phase I).

Sample Date	Applied Temperature °C	4,676.96 m^3/h-d 0.5 mgad			9,353.92 m^3/h-d 1.0 mgad			14,030.89 m^3/h-d 1.5 mgad		
		0.72 mm °C	0.40 mm °C	0.17 mm °C	0.72 mm °C	0.40 mm °C	0.17 mm °C	0.72 mm °C	0.40 mm °C	0.17 mm °C
7/26	26.0	21.2	21.5	22.6	25.0	24.5	24.0	26.8	25.2	26.2
7/31	23.7	23.2	23.2	22.6	23.4	23.0	23.0	23.0	23.0	23.0
July Ave.	24.9	22.2	22.4	22.6	24.2	23.8	23.5	24.9	24.1	24.6
8/5	-	-	-	-	-	-	-	-	-	-
8/10	21.6	19.8	20.2	19.5	21.3	20.7	20.1	20.2	19.8	20.2
8/14	19.9	20.7	20.8	20.6	18.3	19.8	20.5	20.0	20.6	20.7
8/21	19.0	-	-	-	-	-	-	-	-	-
8/29	20.7	20.0	21.0	20.4	20.2	19.7	20.2	20.2	19.8	19.7
Aug. Ave.	16.2	20.2	20.7	20.2	19.9	20.1	20.3	20.1	20.1	20.2
9/4	19.2	18.5	18.5	16.7	19.0	18.1	18.5	19.0	18.8	17.2
9/11	19.0	17.9	17.8	17.8	17.9	17.9	16.7	18.8	18.3	16.8
9/18	17.1	16.8	17.1	16.4	17.2	16.7	15.7	17.8	16.4	-
9/26	16.8	16.2	16.3	16.3	15.2	15.8	16.8	16.0	15.4	16.6
Sept. Ave.	18.0	17.4	17.4	16.8	17.3	17.1	16.9	17.9	17.2	16.9
10/3	15.1	14.9	15.0	15.1	14.2	14.1	15.0	14.3	14.9	14.5
10/10	13.0	12.6	11.0	13.0	12.4	12.0	12.6	12.9	12.0	13.2
10/17	11.9	10.8	10.8	11.6	11.4	10.6	11.2	11.3	10.7	10.7
10/24	11.9	12.1	10.3	10.7	12.3	10.2	10.4	12.9	10.1	10.2
10/31	9.5	7.8	7.6	7.7	6.9	7.6	7.6	7.2	7.4	7.8
Oct. Ave.		11.6	10.9	11.6	11.4	10.9	11.4	11.7	11.0	11.3
11/7	7.1	7.7	5.9	5.6	6.4	4.7	5.5	6.3	4.6	5.2
11/14	5.8	5.2	4.9	5.6	5.1	4.0	4.4	4.9	3.9	4.0
11/21	5.1	4.8	4.0	4.9	4.2	3.2	4.2	4.7	2.8	3.8
11/26	4.1	3.8	3.9	4.3	3.7	3.7	4.3	3.4	2.8	4.2
Nov. Ave.		5.4	4.7	5.1	4.9	3.9	4.6	4.8	3.5	4.3
Overall Monthly Ave.		15.4	15.2	15.3	15.5	15.2	15.3	15.9	15.2	15.5

Table A-10. Laboratory scale filter results for the 12 mgad (112,247.1 m^3/h-d) loading during 1975 (Phase II).

Sample Date	Applied Influent	(112,247.1 m^3/h-d) 12 mgad					
		0.72 mm		0.40 mm		0.17 mm	
		Eff. Conc.	% Rem.	Eff. Conc.	% Rem.	Eff. Conc.	% Rem.
		Biochemical Oxygen Demand (mg/l)					
3/22	21.1						
3/23	15.1						
3/24	27.3	13.4	51	10.8	61	8.3	70
3/25	19.9						
3/28	16.7	13.6	18	8.9	47	10.6	36
Overall Ave.	20.0	13.5	33	9.8	51	9.5	53
Stand. Dev.	4.7	0.1		1.3		1.6	
		Suspended Solids (mg/l)					
3/22	71.0	40.4	43	26.6	63	12.6	82
3/23	58.0	44.5	23	28.7	50.4	11.5	80
3/24	N.A.	22.2	-	14.8	-	16.8	-
3/28	41.6	26.4	37	10.1	75.6	13.0	69
3/29	22.0	13.9	37	7.0	68.1	12.0	46
Overall Ave.	48.1	29.5	39	17.4	63.7	13.2	73
Stand. Dev.	21.2	12.8		9.2		2.1	
		Volatile Suspended Solids (mg/l)					
3/22	50.9	36.8	28	24.3	52	5.3	90
3/23	45.0	36.1	20	23.4	48	7.2	84
3/24	N.A.	17.8	-	13.0	-	11.8	-
3/28	35.9	25.8	28	8.9	75	11.8	67
3/29	22.0	13.9	37	7.0	68	12.0	46
Overall Ave.	38.4	26.1	32	15.3	60	9.6	75
Stand. Dev.	12.6	10.4		8.1		3.2	

Note: During this loading period filter runs of 2 days and 5 days were obtained. In both runs the 0.17 mm filter plugged.

Table A-11. Laboratory scale filter results for the 8 mgad (74,831.4 m^3/h-d) loading during 1975 (Phase II).

Sample Date	Applied Influent	0.72 mm Eff. Conc.	0.72 mm % Rem.	0.40 mm Eff. Conc.	0.40 mm % Rem.	0.17 mm Eff. Conc.	0.17 mm % Rem.
		(74,831.4 m^3/h-d) 8 mgad					
		Biochemical Oxygen Demand (mg/l)					
3/31	17.2	14.7	15	8.6	50	9.7	44
4/3	20.9	17.3	17	12.6	40	4.8	77
4/8	26.9	14.9	44	15.1	44	9.0	67
4/10	19.2	14.4	25	11.5	40	4.1	79
Overall Ave.	21.1	15.3	27	12.0	43	6.9	67
Stand. Dev.	4.2	2.3		2.7		2.9	
		Suspended Solids (mg/l)					
3/31	39.0	28.3	27	10.0	74	N.A.	-
4/2	20.7	15.9	23	7.5	64	14.6	30
4/3	36.0	29.2	19	13.8	62	9.0	75
4/7	11.4	11.1	3	9.2	19	5.9	48
4/8	32.1	24.3	24	9.0	72	4.7	85
4/9	19.6	14.1	28	8.8	55	5.1	74
4/10	13.8	7.8	43	4.9	65	3.7	73
4/11	23.8	13.0	45	4.7	80	N.A.	-
4/12	17.4	9.3	47	4.1	77	N.A.	-
4/13	21.8	9.4	57	6.1	72	5.4	75
Overall Ave.	23.6	16.2	31	7.8	67	6.9	71
Stand. Dev.	9.3	8.1		3.0		3.8	
		Volatile Suspended Solids (mg/l)					
3/31	N.A.	N.A.		N.A.		N.A.	
4/2	17.0	13.3	21	5.9	65	12.5	26
4/3	28.2	23.7	16	10.1	64	6.5	77
4/7	3.7	10.2	-174	2.1	44	5.6	-50
4/8	34.8	15.7	55	9.9	72	N.A.	-
4/9	13.3	9.8	26	8.0	40	3.1	77
4/10	10.9	6.1	44	5.1	54	3.7	66
4/11	8.7	6.3	27	2.1	76	N.A.	-
4/12	8.3	3.4	59	2.1	75	N.A.	-
4/13	11.2	4.6	59	3.0	73	2.6	77
Overall Ave.	15.1	10.3	32	5.4	65	5.7	62
Stand. Dev.	10.1	6.4		3.3		3.7	

Note: During this loading period filter runs of 4 days and 7 days were obtained. In both runs the 0.17 mm filter plugged.

Table A-12. **Laboratory scale filter results for the 4 mgad (37,415.7 m^3/h-d) loading during 1975 (Phase II).**

Sample Date	Applied Influent	(37,415.7 m^3/h-d) 4 mgad					
		0.72 mm		0.40 mm		0.17 mm	
		Eff. Conc.	% Rem.	Eff. Conc.	% Rem.	Eff. Conc.	% Rem.
		Biochemical Oxygen Demand (mg/l)					
4/22	11.4	6.4	44	4.4	61	2.1	82
4/26	15.4	9.5	39	7.6	50	3.2	79
4/29	10.8	7.2	33	6.5	40	1.6	85
Overall Ave.	12.5	7.7	39	6.2	51	2.3	82
Stand. Dev.	2.5	1.6		1.6		0.8	
		Suspended Solids (mg/l)					
4/22	17.4	6.4	63	3.6	80	1.9	89
4/26	18.0	8.0	56	4.5	75	2.9	84
4/29	19.0	8.7	54	4.8	75	2.7	86
Overall Ave.	18.1	7.7	58	4.3	76	2.5	86
Stand. Dev.	0.8	1.2		0.6		0.5	
		Volatile Suspended Solids (mg/l)					
4/22	7.0	2.0	72	1.6	77	0.8	89
4/26	6.3	3.1	51	1.7	73	1.6	74
4/29	8.7	3.5	60	1.7	80	0.9	90
Overall Ave.	7.3	2.9	61	1.7	77	1.1	85
Stand. Dev.	1.3	0.8		0.1		0.5	

Note: During this loading period a filter run of 16 days was obtained. The 0.17 mm filter plugged.

Table A-13. Pilot scale field filters results for the 16-8-4 mgad (149,662.8 - 74,831.4 - 37,415.7 m^3/h-d) and 12-6-3 mgad (112,247.1 - 56,123.6 - 28,061.8 m^3/h-d) decreasing loading during 1975 (Phase III).

Sample Date	Applied Influent	(149,662.8 m^3/h-d) 16 mgad 0.72 mm		(74,831.4 m^3/h-d) 8 mgad 0.40 mm		(37,415.7 m^3/h-d) 4 mgad 0.17 mm	
		Eff. Conc.	% Rem.	Eff. Conc.	% Rem.	Eff. Conc.	% Rem.
		Biochemical Oxygen Demand (mg/l)					
7/10	30.6	10.5	66	9.3	70	6.9	77
		Suspended Solids (mg/l)					
7/10	51.0	5.7	89	4.1	92	5.1	90
		Volatile Suspended Solids (mg/l)					
7/10	31.0	2.9	91	1.9	94	2.0	94

Sample Date	Applied Influent	(112,247.1 m^3/h-d) 12 mgad 0.72 mm		(56,123.6 m^3/h-d) 6 mgad 0.40 mm		(28,061.8 m^3/h-d) 3 mgad 0.17 mm	
		Eff. Conc.	% Rem.	Eff. Conc.	% Rem.	Eff. Conc.	% Rem.
		Biochemical Oxygen Demand (mg/l)					
7/10	30.6	9.5	69	6.6	71	7.1	77
		Suspended Solids (mg/l)					
7/10	51.0	5.7	89	3.8	93	4.4	92
		Volatile Suspended Solids (mg/l)					
7/10	31.0	2.5	92	1.2	96	1.1	97

Note: During these filter runs, runs of only 1 day was obtained for each. The 0.72 mm filter plugged.

Table A-14. **Pilot scale field filters results for the 8-4-2 mgad (74,831.4 - 37,415.7 - 18,707.9 m^3/h-d) decreasing loading during 1975 (Phase III).**

Sample Date	Applied Influent	(74,831.4 m^3/h-d) 16 mgad 0.72 mm		(37,415.7 m^3/h-d) 8 mgad 0.40 mm		(18,707.9 m^3/h-d) 4 mgad 0.17 mm	
		Eff. Conc.	% Rem.	Eff. Conc.	% Rem.	Eff. Conc.	% Rem.
		Biochemical Oxygen Demand (mg/l)					
7/17	9.3	4.9	47	5.3	43	3.1	67
7/23	7.4	6.7	9	4.8	35	0.5	93
7/31	10.8	10.2	6	6.7	38	1.9	83
8/7	13.5	8.9	34	3.1	77	N.A.	-
8/21	12.6	6.2	51	4.6	64	4.2	67
Overall Ave.	10.7	7.4	31	4.9	54	2.4	77
Stand. Dev.	2.5	2.1		1.3		1.6	
		Suspended Solids (mg/l)					
7/17	24.1	2.1	91	3.2	87	3.6	85
7/23	16.7	1.9	89	2.1	88	0.7	96
7/31	14.8	4.7	68	2.5	83	3.9	74
8/7	15.3	3.5	77	2.9	81	N.A.	-
8/21	50.1	33.8	33	26.2	48	23.9	52
Overall Ave.	24.2	9.2	62	7.4	70	8.0	67
Stand. Dev.	14.9	13.8		10.5		10.6	
		Volatile Suspended Solids (mg/l)					
7/17	13.5	0.8	94	1.5	89	0.6	95
7/23	10.8	1.4	87	1.7	85	0.4	97
7/31	10.0	2.9	71	1.6	84	1.2	88
8/7	9.0	2.6	71	1.8	80	N.A.	-
8/21	36.0	25.4	29	24.2	33	15.8	56
Overall Ave.	15.9	6.6	58	6.2	61	4.5	72
Stand. Dev.	11.4	10.5		10.1		7.5	
		pH					
7/17	8.0	8.0	-	7.9	-	7.7	-
7/23	7.9	7.4	-	7.6	-	7.7	-
7/31	8.2	7.7	-	7.8	-	7.8	-
8/7	N.A.	N.A.	-	N.A.	-	N.A.	-
8/21	9.2	9.1	-	9.1	-	8.7	-
Overall Ave.	8.2	7.8	-	7.9	-	7.8	-
		Dissolved Oxygen (mg/l)					
7/17	0.9	6.2	-	7.0	-	6.8	-
7/23	1.2	6.9	-	7.7	-	8.1	-
7/31	2.7	5.2	-	7.2	-	7.0	-
8/7	N.A.	N.A.		N.A.		N.A.	
8/21	13.8	7.7	-	7.9	-	8.5	-
Overall Ave.	4.7	6.5	-	7.4	-	7.6	-
		Temperature of Wastewater (°C)					
7/17	23.2	22.2	-	22.2	-	20.2	-
7/23	23.2	23.0	-	22.3	-	22.4	-
7/31	22.0	20.3	-	20.0	-	21.4	-
8/7	22.2	22.4	-	20.7	-	N.A.	-
8/21	19.8	19.9	-	19.6	-	18.6	-
Overall Ave.	22.1	21.6	-	21.0	-	20.7	-

Note: During this loading period filter runs of 7, 7, 7, and 9 days were obtained. In all instances the 0.72 mm filter plugged.

Table A-15. **Pilot scale field filters results for the 6-3-1.5 mgad (56,123.6 - 28,061.8 - 14,030.9 m^3/h-d) decreasing loading during 1975 (Phase III).**

Sample Date	Applied Influent	(56,123.6 m^3/h-d) 6 mgad 0.72 mm		(28,061.8 m^3/h-d) 3 mgad 0.40 mm		(14,030.9 m^3/h-d) 1.5 mgad 0.17 mm	
		Eff. Conc.	% Rem.	Eff. Conc.	% Rem.	Eff. Conc.	% Rem.
		Biochemical Oxygen Demand (mg/l)					
7/17	9.3	5.1	45	4.2	56	2.2	76
8/7	13.5	1.2	91	0.7	95	N.A.	-
8/21	12.6	7.5	41	6.4	50	2.8	78
8/28	9.0	4.7	48	2.9	68	1.9	79
Overall Ave.	11.1	4.6	58	3.5	68	2.3	79
Stand. Dev.	2.3	2.6		2.4		0.5	
		Suspended Solids (mg/l)					
7/17	24.1	2.9	88	2.7	89	3.1	87
8/7	15.3	1.3	91	0.6	96	N.A.	-
8/21	50.1	32.1	36	28.2	44	13.7	73
8/28	51.6	15.2	71	12.1	77	8.5	84
Overall Ave.	35.3	12.9	64	10.9	69	8.5	76
Stand. Dev.	18.3	14.2		12.6		5.3	
		Volatile Suspended Solids (mg/l)					
7/17	13.5	1.2	91	0.8	94	0.6	96
8/7	9.0	0.9	91	0.5	95	N.A.	-
8/21	36.0	25.1	30	21.0	42	10.8	70
8/28	35.3	12.8	64	10.4	71	7.0	80
Overall Ave.	23.4	10.0	57	8.2	65	6.1	74
Stand. Dev.	14.2	11.5		9.7		5.2	
		pH					
7/17	8.0	7.9	-	7.9	-	7.9	-
8/7	N.A.	N.A.	-	N.A.	-	N.A.	-
8/21	9.2	9.1	-	9.0	-	8.4	-
8/28	9.4	8.9	-	8.6	-	8.3	-
Overall Ave.	8.4	8.3	-	8.3	-	8.2	-
		Dissolved Oxygen (mg/l)					
7/17	0.9	7.1	-	7.1	-	7.3	-
8/7	N.A.	N.A.		N.A.		N.A.	
8/21	13.8	7.5	-	8.0	-	8.4	-
8/28	12.9	5.0	-	7.9	-	7.8	-
Overall Ave.	9.2	6.5	-	7.7	-	7.8	-
		Temperature of Wastewater (°C)					
7/17	23.2	21.8	-	21.7	-	20.7	-
8/7	22.2	20.8	-	19.0	-	N.A.	-
8/21	19.8	19.8	-	19.4	-	18.5	-
8/28	19.0	19.0	-	18.7	-	18.2	-
Overall Ave.	21.1	20.4	-	19.7	-	19.1	-

Note: Immediately after the 7/17 sample date mechanical malfunctions caused operations to cease. However during later filter runs, runs of 9 and 10 days, were obtained. In all instances the 0.72 mm filter plugged.

Table A-16. Pilot scale field filters results for the equal loading period of 1975 (Phase IV).

Sample Date	Applied Influent	(37,415.7 m^3/h-d) 4.0 mgad						(28,061.8 m^3/h-d) 3.0 mgad					
		0.72 mm		0.40 mm		0.17 mm		0.72 mm		0.40 mm		0.17 mm	
		Eff. Valve	% Rem.	Eff. Valve	% Rem.	Eff. Valve	% Rem.	Eff. Valve	% Rem.	Eff. Valve	% Rem.	Eff. Valve	% Rem.
						Biochemical Oxygen Demand (mg/l)							
9/5	4.7	3.3	30	3.6	23	2.1	55	4.4	7	3.6	24	1.9	60
9/10	5.4	4.4	18	3.4	37	2.2	60	3.7	32	3.4	38	1.5	72
9/19	10.1	6.5	36	6.1	40	3.0	71	5.1	50	1.9	81	1.3	87
9/26	7.7	4.7	39	3.1	59	0.7	91	4.2	46	2.8	63	0.8	90
Overall Ave.	7.0	4.7	33	4.1	42	2.0	71	4.3	38	2.9	58	1.4	80
Stand. Dev.	2.5	1.3		1.4		0.9		0.6		0.8		0.5	
						Suspended Solids (mg/l)							
9/5	15.8	9.9	37	10.3	35	7.0	55	12.1	23	15.0	5	6.5	59
9/10	24.1	9.3	61	8.9	63	6.4	73	8.0	67	7.3	70	3.5	86
9/19	49.6	16.1	68	12.8	74	7.8	84	14.9	70	8.8	82	4.9	90
9/26	24.0	4.7	80	3.1	87	2.0	92	5.0	79	3.6	85	2.1	91
Overall Ave.	28.4	10.0	65	8.8	69	5.8	80	10.0	65	8.7	69	4.2	85
Stand. Dev.	14.7	4.7		4.1		2.6		4.4		4.8		1.9	
						Volatile Suspended Solids (mg/l)							
9/5	10.8	5.4	51	3.8	65	3.2	70	8.4	22	6.2	43	4.5	59
9/10	15.7	5.9	62	3.7	76	4.1	74	4.7	70	4.3	73	2.9	82
9/19	36.5	10.8	71	8.7	76	5.8	84	10.3	72	6.8	81	3.9	89
9/26	15.4	2.2	86	3.1	80	1.2	93	3.5	77	1.8	89	1.8	89
Overall Ave.	19.6	6.1	69	4.8	75	3.6	82	6.8	66	4.8	76	3.3	83
Stand. Dev.	11.5	3.5		2.6		1.9		3.2		2.3		1.2	
						pH							
9/5	9.1	9.0		9.0		8.8		9.0		8.9		8.6	
9/10	9.2	9.1		9.0		8.8		9.1		9.0		8.6	
9/19	N.A.	N.A.		N.A.		N.A.		N.A.		N.A.		N.A.	
9/26	9.3	8.9		8.4		8.4		8.9		8.5		8.3	
Overall Ave.	9.2	9.0		8.7		8.5		9.0		8.8		8.5	
Stand Dev.													

Table A-16. Continued.

Sample Date	Applied Influent	(37,415.7 m³/h-d) 4.0 mgad 0.72 mm Eff. Valve	% Rem.	0.40 mm Eff. Valve	% Rem.	0.17 mm Eff. Valve	% Rem.	(28,061.8 m³/h-d) 3.0 mgad 0.72 mm Eff. Valve	% Rem.	0.40 mm Eff. Valve	% Rem.	0.17 mm Eff. Valve	% Rem.
						Dissolved Oxygen, (mg/l)							
9/5	12.8	7.8	-	7.8	-	8.2	-	7.5	-	7.9	-	7.1	-
9/10	7.9	6.8	-	7.5	-	7.0	-	6.9	-	7.5	-	6.5	-
9/19	N.A.	N.A.		N.A.		N.A.		N.A.		N.A.		N.A.	
9/26	13.1	4.1	-	7.6	-	7.1	-	7.5	-	7.7	-	6.1	-
Overall Ave.	11.3	6.2	-	7.6	-	7.4	-	7.3	-	7.7	-	6.6	-
						Temperature of Wastewater (°C)							
9/5	18.8	19.0	-	19.0	-	18.9	-	18.7	-	19.2	-	18.8	-
9/10	N.A.	N.A.		N.A.		N.A.		N.A.		N.A.		N.A.	
9/19	N.A.	N.A.		N.A.		N.A.		N.A.		N.A.		N.A.	
9/26	16.0	15.3	-	15.9	-	16.9	-	15.1	-	15.4	-	16.9	-
Overall Ave.	17.40	17.15	-	17.45	-	17.9	-	16.9	-	17.3	-	17.4	-

Note: A filter run of 21 days was obtained for the 4 mgad series while a filter run of 27 days was obtained for the 3 mgad series. The 0.72 mm filter plugged in both cases. The 0.72 mm filter of the 4 mgad series plugged on September 24, however the series was still loaded and sampled on September 26.

Appendix B

Cost Estimating

Table B-1. Cost estimate I–three intermittent sand filters operated in series (duplicate facilities).

Design flow rate: 0.5 mgd
Design hydraulic loading rate: 1.5 mgad (each filter area = 0.33 acres)
Three effective size sand filters employed in the following order: 0.72 mm effective size, 0.40 mm effective size, and 0.17 mm effective size @ 30 inch bed depth for each filter
Interest rate: 7%
Economic life:

Land	100 yrs.
Embankments	50 yrs.
Pumps	10 yrs.
Sand	20 yrs.
Gravel	50 yrs.
Other	50 yrs.

Initial construction cost (in-place):

	Quantity	Unit Cost	Total Cost
Granular media			
0.17 mm e.s.	2,689 yd^3	$4.83	$12,988
0.40 mm e.s.	2,689 yd^3	$5.77	$15,516
0.72 mm e.s.	2,689 yd^3	$5.77	$15,516
Gravel	3,227 yd^3	$4.83	$15,586
Lateral drains (10 ft spacings) 6 inch	8,700 ft.	$1.10	$ 9,570
Collection pipe (10 inch)	1,550 ft.	$2.20	$ 3,410
Distribution pipe (10 inch)	1,000 ft.	$2.20	$ 2,200
Ductile iron pipe	500 ft.	$10.44	$ 5,220
Excavation and embankments	14,600 yd^3	$1.50	$21,900
Distribution system	6	$549	$ 3,294
Pumps (2,800 gpm, 1 pump per two filters plus 1 standby)	4	$3500	$14,000
Land	5 acres	$1000	$ 5,000
Building	1	$2000	$ 2,000
Total capital cost			$126,200

Amortization:

Land $5,000 (0.07008)	$ 350
Pipe ($9,570 + $3,410 + $2,200 + $5,220) (0.07246)	$ 1,478
Sand ($12,988 + $15,516 + $15,516) (0.09439)	$ 4,155
Gravel ($15,586)(0.07246)	$ 1,129
Excavation and embankment $21,900 (0.07246)	$ 1,587
Distribution system $3,294 (0.07246)	$ 239
Pumps $14,000 (0.14238)	$ 1,993
Building $2,000 (0.07246)	$ 145
Total amortization	$11,076

Table B-1. Continued.

	Total Cost
Annual operating and maintenance costs:	
Maintenance costs	$ 1,000
Manpower cost (1/4 man year @ $10,000/yr)	$ 2,500
Power 22 hp or 16 kw (16 kw/pump) (3 hr/day) (365 days) (3 pumps) ($0.03/kw-year)	$ 1,577
Total annual operating and maintenance costs	$ 5,077
Total annual cost:	$16,153
Amortization with federal assistance: Federal government would pay for 75% of construction cost; remaining 25% financed at 7% for 20 years	
Total amortization with federal aid $126,200 (0.25) (0.09439)	$ 2,978
Total annual cost with federal aid:	$ 8,055

Cost per million gallons:

With federal aid

$$\frac{\text{Total Annual Cost}}{\text{Total Annual Flow}} = \frac{\$8,055}{182.5\text{ mg}} = \$44/\text{million gallons}$$

Without federal aid

$$\frac{\text{Total Annual Cost}}{\text{Total Annual Flow}} = \frac{\$16,153}{182.5\text{ mg}} = \$89/\text{million gallons}$$

Construction cost per acre:

$$\frac{\$126,200}{2\text{ acres}} = \$63,100/\text{acre}$$

Table B-2. Cost estimate II–two intermittent sand filters operated in series (duplicate facilities).

Design flow rate: 0.5 mgd
Design hydraulic loading rate: 1.0 mgad (each filter area = 0.50 acres)
Two effective size sand filters employed in the following order: 0.72 mm or 0.40 mm effective size followed by a 0.17 mm effective size @ 30 inch bed depth for each filter
Interest rate: 7%
Economic life:

Land	100 yrs
Embankments	50 yrs
Pumps	10 yrs.
Sand	20 yrs.
Gravel	50 yrs.
Other	50 yrs.

Initial construction cost (in-place):

	Quantity	Unit Cost	Total Cost
Granular media			
0.17 mm e.s.	4,033 yd^3	$4.83	$19,479
0.40 mm e.s. or 0.72 mm e.s.	4,033 yd^3	$5.77	$23,270
Gravel	3,227 yd^3	$4.83	$15,586
Lateral drains (10 ft. spacings) 6 inch	8,700 ft.	$1.10	$ 9,570
Collection pipe (10 inch)	1,550 ft.	$2.20	$ 3,410
Distribution pipe (10 inch)	1,000 ft.	$2.20	$ 2,200
Ductile iron pipe	500 ft.	$10.44	$ 5,220
Excavation and embankments	14,600 yd^3	$1.50	$21,900
Distribution system	4	$549	$ 2,196
Pumps (2,800 gpm, 1 pump per two filters plus 1 standby)	3	$3500	$10,500
Land	5 acres	$1000	$ 5,000
Building	1	$2000	$ 2,000
Total capital cost			$120,331

Amortization:

Land $5,000 (0.07008)	$ 350
Pipe ($9,570 + $3,410 + $2,200 + $5,220) (0.07246)	$ 1,478
Sand ($19,479 + $23,270) (0.09439)	$ 4,035
Gravel $15,586 (0.07246)	$ 1,129
Excavation and embankment $21,900 (0.07246)	$ 1,587
Distribution system $2,196 (0.07246)	$ 159
Pumps $10,500 (0.14238)	$ 1,495
Building $2,000 (0.07246)	$ 145
Total amortization	$10,378

Annual operating and maintenance costs:

Maintenance costs	$ 1,000
Manpower cost (1/4 man year @ $10,000/yr)	$ 2,500
Power 22 hp or 16 kw (16 kw/pump) (3 hr/day) (365 days) (2 pumps) ($0.03/kw-year)	$ 1,051
Total annual operating and maintenance costs	$ 4,551
Total annual cost:	$14,929

Amortization with federal assistance:
Federal government would pay 75% of construction cost; remaining 25% financed at 7% for 20 years

Table B-2. Continued.

	Total Cost
Total amortization with federal aid $120,331 (0.25) (0.09439)	$ 2,840
Total annual cost with federal aid:	$ 7,391

Cost per million gallons:

With federal aid

$$\frac{\text{Total Annual Cost}}{\text{Total Annual Flow}} = \frac{\$7{,}391}{182.5\text{ mg}} = \$41/\text{million gallons}$$

Without federal aid

$$\frac{\text{Total Annual Cost}}{\text{Total Annual Flow}} = \frac{\$14{,}929}{182.5\text{ mg}} = \$82/\text{million gallons}$$

Construction cost per acre:

$$\frac{\$120{,}331}{2\text{ acres}} = \$60{,}166/\text{acre}$$

Table B-3. Cost estimate III–modification of existing lagoon system to accommodate three intermittent sand filters in series in one of the existing cells (duplicate facilities).

All considerations would be the same as Estimate I with the exception being the elimination of land costs and approximately 25% of the embankment requirements.

Total capital cost $126,200 - (0.25) ($21,900) - $5,000	$115,725
Total amortization $11,076 - (0.25) ($1,587) - $350	$ 10,329
Total annual operating and maintenance costs	$ 5,077
Total annual cost	$ 15,406
Total amortization with federal aid $115,725 (0.25) (0.09439)	$ 2,731
Total annual cost with federal aid	$ 7,808

Cost per million gallons:

With federal aid

$$\frac{\text{Total Annual Cost}}{\text{Total Annual Flow}} = \frac{\$7{,}808}{182.5\text{ mg}} = \$43/\text{million gallons}$$

Without federal aid

$$\frac{\text{Total Annual Cost}}{\text{Total Annual Flow}} = \frac{\$15{,}406}{182.5\text{ mg}} = \$84/\text{million gallons}$$

Construction cost per acre:

$$\frac{\$115{,}725}{2\text{ acres}} = \$57{,}863/\text{acre}$$

Table B-4. Cost estimate IV–modification of existing lagoon system to accommodate two intermittent sand filters in series in one of the existing cells (duplicate facilities).

All considerations would be the same as Estimate II with the exception being the elimination of land costs and approximately 25% of the embankment requirements.

Total capital cost $120,331 - (0.25) ($21,900) - $5,000	$109,856
Total amortization $10,378 - (0.25) ($1,587) - $350	$ 9,631
Total annual operating and maintenance costs	$ 4,551
Total annual cost	$ 14,182
Total amortization with federal aid $109,856 (0.25) (0.09439)	$ 2,592
Total annual cost with federal aid	$ 7,143

Cost per million gallons:

With federal aid

$$\frac{\text{Total Annual Cost}}{\text{Total Annual Flow}} = \frac{\$7,143}{182.5 \text{ mg}} = \$39/\text{million gallons}$$

Without federal aid

$$\frac{\text{Total Annual Cost}}{\text{Total Annual Flow}} = \frac{\$14,182}{182.5 \text{ mg}} = \$78/\text{million gallons}$$

Construction cost per acre:

$$\frac{\$109,856}{2 \text{ acres}} = \$54,928/\text{acre}$$