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# Evaluation of the Addition of Granular Media Filtration to Wastewater Treatment Plants to Meet New Standards

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# EVALUATION OF THE ADDITION OF GRANULAR MEDIA FILTRATION TO WASTEWATER TREATMENT PLANTS TO MEET NEW STANDARDS

by

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## ABSTRACT

A dual media filtration study was carried out at the Central Weber Wastewater Treatment Plant in Ogden, Utah, to evaluate its feasibility as a tertiary treatment to meet new effluent quality standards. A review of the literature indicated that dual media filters were more efficient than conventional single media sand filters because of the "in depth" filtration achieved by dual media filters.

An experimental filter was operated at four different hydraulic loading rates, ranging from 3 to 6 gpm/ft$^2$ (122.10 to 244.20 l/min/m$^2$), to evaluate its effects on effluent quality. Hydraulic loading rate was shown not to affect suspended solids removal. The experimental filter produced excellent suspended solids removal; however, $BOD_5$ removal efficiency was relatively poor because the influent to the filter contained high concentrations of soluble $BOD_5$ and colloidal organic solids. Filter effluent quality met State of Utah standards of 10 mg/l of $BOD_5$ at the hydraulic loading rate of 3 gpm/ft$^2$ and exceeded the standard by less than 2 mg/l at loading rates of 4, 5, and 6 gpm/ft$^2$. Filter cycle durations were very short at higher filtration rates due to removal of influent suspended solids in the intermixed portion of the filter media. Biological growth within the filter media was a major problem in the operation of the filter. The filter influent was chlorinated to prevent this growth.

The study indicated that dual media filtration of trickling filter plant effluent to meet new effluent quality standards is economically feasible and can produce an effluent which meets state and federal standards.

## ACKNOWLEDGMENTS

The financial support and assistance provided by the management and staff of the Central Weber Wastewater Treatment Plant is gratefully acknowledged. Sincere appreciation also goes to Eimco Corporation and Utah Water Research Laboratory for their cooperation.

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# INTRODUCTION

The 1972 Federal Water Pollution Control Act (PL 92-500) has generated more discussion than any other topic in the field of water quality (WPCF, 1973b). The ultimate goal of the Act is to achieve "zero discharge" by 1985 (WPCF, 1973b). The Act requires all publicly owned wastewater treatment plants to achieve secondary treatment by 1977 (WPCF, 1973a). "Secondary treatment" for the State of Utah has been defined in terms of effluent biochemical oxygen demand ($BOD_5$), suspended solids (SS), total and fecal coliform bacteria and pH. The monthly average of effluent $BOD_5$ or SS should not exceed 25 mg/l, and that of total coliform and fecal coliform bacteria in the effluent should not surpass 2000 per 100 ml or 200 per 100 ml, respectively. The effluent pH should be between 6.5 and 9.0 (State of Utah, 1974). By 1980, all wastewater treatment plants in the State of Utah are required to treat the wastewater to the extent that the monthly average effluent $BOD_5$ and SS are not greater than 10 mg/l, the monthly average of total coliform and fecal coliform bacteria count is less than 200 per 100 ml or 20 per 100 ml, respectively, and the effluent pH is within the limits of 6.5 and 9.0.

Many existing wastewater treatment plants are not capable of meeting the more stringent performance levels required by today's water quality standards. Primary and secondary treatment is incapable of producing an effluent which meets these strict effluent water quality standards. Filters, microstrainers, activated carbon adsorption processes and polishing lagoons are a few examples of the processes and operations which have been successfully used to polish secondary treatment plant effluents.

Wastewater treatment facilities construction in Utah has been restricted because of the limited support available from the U.S. Environmental Protection Agency (EPA) construction grants program. It will be impossible for most of the wastewater treatment plants in Utah to comply with the federal effluent quality standards before the deadlines imposed by the 1972 act. The Utah State Water Pollution Committee and the Utah State Board of Health have decided to extend the deadline of the 1972 act by three years. Thus, all the wastewater treatment plants in the State of Utah will be required to meet the effluent quality standards as specified by PL 92-500, not later than June 30, 1983 (State of Utah, 1974).

## Nature of the Problem

The Central Weber Wastewater Treatment Plant located northwest of Ogden, Utah, receives wastewater from the communities of Pleasant View, North Ogden, Ogden, South Ogden, Washington Terrace, Harrisville, Riverdale, and part of the unincorporated county. The authorities of the treatment plant are faced with the problem of upgrading the existing treatment plant in order to comply with PL 92-500. The main goal is to reduce $BOD_5$ and SS in the final effluent to meet the new standards. A dual media, SVG pilot scale filter has been set up at the treatment plant to evaluate its efficiency in upgrading the plant effluent.

## Filtration

Sand filtration is one of the oldest and most common methods used in wastewater purification. The first application of filtration as a means of removing debris from liquids is lost in antiquity. The first public works application is usually considered to have occurred in 1852 when the City of London was required by Parliament to filter its water through sand filters. The first successful filters in the United States were installed in Poughkeepsie, New York, in 1872 (Fair, Geyer, and Okun, 1968). Filtration, in the early days, had been used in potable water production for the removal of solids. The same principle was later applied to treat wastewater. As technology progressed through the years, more efficient filters have been developed by modifying the original sand filters. Dual media filters with sand and anthracite coal and multimedia filters with sand, anthracite coal and garnet have been found to be more efficient than the original sand filters which contained unstratified and later stratified sands. Filters with upward flow direction have been tried with some success, but the conventional downflow unit is still more popular because of its simplicity (Tebbutt, 1971). However, Russia and other European countries use upflow and dual flow filter systems rather extensively and claim excellent effluents and ease of operation.

## Objectives

The general objectives of this investigation were:

1. To evaluate the SVG dual media filter[1] as a means of upgrading the secondary effluent from the Central Weber Wastewater Treatment Plant to meet the new effluent quality standards as required under PL 92-500.

2. To determine operational and maintenance expenses that would be incurred on large scale filters.

To accomplish the general objectives, it was necessary to:

1. Establish the quality of influent to the filter. Concentration, size distribution, and other characteristics of suspended solids in the influent have a direct bearing on length of the filter cycle and hence the quantity of net filtered water production. Higher suspended solids concentrations in the influent would lead to shorter filter runs under identical conditions. The $BOD_5$ measures the amount of biologically oxidizable organic matter that is present in a wastewater. The $BOD_5$ test has long been used in stream pollution control activities to determine the rates at which oxidation will occur in receiving bodies of water. Since $BOD_5$ and SS concentrations were the two parameters controlled by PL 92-500 exceeded in the effluent from the Central Weber Plant, the quality of influent to the filter was evaluated by measuring the $BOD_5$ and SS concentrations.

2. Establish the quality of effluent from the filter to evaluate the effectiveness of the filter media used. Attainment of the required effluent quality was one of the main goals of this experiment. The efficiency of the experimental unit was determined by comparing the qualities of influent and effluent. Hence, the effluent quality was also based on its $BOD_5$ and SS concentrations.

3. Arrive at an optimum hydraulic loading rate. Filtration rate is an important criteria in the design of granular media filters. High filtration rates cause short filter cycles, thus, increasing the quantity of backwash water to be recycled. Long filter cycles could be achieved by low filtration rates, but this would require more filters to treat the same quantity of wastewater which means higher capital, operational and maintenance costs. If the filtration rate is too low and the filter cycle too long, putrescible solids, which are entrained within the filter media, could cause rapid depletion of chlorine residual in the influent and increase the chances of biological growth in the media, thus, ruining the filter media (Davidson, 1975). Another disadvantage of very long filter cycles is the high $BOD_5$ of backwash water which in the absence of a dirty-backwash water storage tank, would impose a heavy slug on the raw wastewater when backwash water is recycled (Davidson, 1975). Thus, selection of filtration rate is an important design consideration.

4. Determine the loading rate in terms of pounds of incoming suspended solids per cubic foot of filter media per day (#SS/ft$^3$-day). Headloss development within the filter media and hence the length of the filter cycle depends on suspended solids accumulations and distribution in the filter media. This parameter is more important than filtration rate because selection of the design filtration rate is judged by the length of the filter cycle which in turn depends on the accumulation and distribution of suspended solids in the filter bed (Davidson, 1975). Comparison of pound of influent SS/ft$^3$-day with pounds of SS removed by the filter/ft$^3$-day would help determine the effectiveness of the media used in the experiment.

5. Establish the effectiveness of coagulants and coagulant aids to help improve the effluent quality. Aluminum and iron salts have been widely used in the past as coagulants and flocculants in wastewater treatment. Synthetic polymers in small dosages have proven to be important coagulant aids where aluminum and iron salts are used as coagulants. The polymers help in forming tougher flocs which can withstand high velocities in the filter media. Aluminum and iron salts in wastewater treatment, are mainly used for phosphorus removal by forming insoluble phosphorus complexes which can be removed by the filter media. Pilot plant tests should be conducted in order to determine the required dosages of coagulant and coagulant aids, because overdosing causes post-flocculation and hence floc carry-over. Also when coagulants and coagulant aids are used prior to infiltration, reduced net filtered water production results. Addition of coagulants and coagulant aids shorten the length of the filter cycle for the same hydraulic loading rate. This would require more filters and hence more capital for the same quantity of wastewater. Chemicals and chemical feeders add to the cost of tertiary treatment.

[1] **Manufactured by the Eimco Division of ENVIRO-TECH, Salt Lake City, Utah.**

# LITERATURE REVIEW

## Need for Advanced Treatment

Proper treatment of the wastewater protects the health of man, fish and wildlife, preserves the aesthetic value of receiving waters, thus, increasing its potential for reuse. Hence, proper treatment of wastewater is very important in view of the present concern with the adequacy of the national water resources (Culp, 1963). Secondary effluent from most wastewater treatment plants normally contains oxygen demanding organic materials which may be harmful to the fish and other beneficial uses of the receiving stream (Smith and Gregorio, 1970). Many existing wastewater treatment plants are unable to meet the present more stringent standards set forth by the regulatory agencies (U.S. EPA, 1971). Secondary wastewater treatment will not be adequate for our future water quality requirements (Culp, 1963; Smith and Gregorio, 1970). There is a need to develop new methods of wastewater treatment to produce superior quality effluents. Water reuse and advanced wastewater treatment will be necessary in the future because it is projected that by 1980, daily consumption of water by U.S. industry, agriculture, and municipalities will exceed the fresh water resources of 600 billion gallons per day (Smith and Gregorio, 1970).

## A Look at the Possible Solution

Tertiary treatment of wastewater seems to be the most plausible solution clean water deficit in most cases. Most tertiary treatment processes are capable of removing suspended solids that account for 35 to 80 percent of organic pollutants (Smith and Gregorio, 1970) that remain in wastewater after secondary treatment. The main factors to be considered in the selection of a tertiary treatment system are:

1. The substance or substances to be removed.
2. The required removal efficiency of the unit.
3. Disposal of the substance or substances to be removed.
4. The economic feasibility (Kreissl, 1974, Tchobanoglous, 1970).

As mentioned earlier, various tertiary treatment methods such as granular media filtration, activated carbon adsorption, microstrainers, polishing lagoons, etc., have so far been studied to upgrade secondary effluents. Of the various methods available for tertiary treatment, granular media filtration has found the most application in large works (Tebbutt, 1971). Different types of filters such as mixed media beds with uni-size or graded media and upflow units have been studied to accomplish more efficient operation. In many cases, however, the conventional downflow unit is used because of its simplicity (Tebbutt, 1971). A moving bed filter has been tested at East Hyde Works of Luton County Borough (Holding, 1972) but final results are not available.

## Filter Design

The design of the filter is said to be optimum when available head is exhausted exactly at the time the suspended solids in the effluent are in excess of the desired effluent quality (Baumann and Huang, 1974). Due to interdependency of the process variables, it is impossible to arrive at an optimum design (Baumann and Huang, 1974). At present no mathematical model is available that would lead to optimum tertiary filter design (Baumann and Huang, 1974). Hence, one must rely on pilot-plant test data to arrive at the most efficient filter design for the wastewater tested (Baumann and Huang, 1974; Kreissl, 1974; Tebbutt, 1971).

## Filtration

A conventional single medium sand filter normally is not very efficient for upgrading secondary effluent; however, intermittent sand filtration has been found effective and economical for smaller systems (Marshall and Middlebrooks, 1974). If secondary effluent is passed through a conventional single medium, graded sand filter, most of the suspended solids will be removed at the surface of the medium (Baumann and Huang, 1974). This will cause the headloss to increase very rapidly across the medium and the whole depth of the bed will not be utilized for suspended solids removal (Baumann and Huang, 1974). Efficient filtration can be achieved if a large portion of the filter bed depth is utilized for suspended solids removal (Baumann and Huang, 1974; Shell and Burns, 1973). This is known as "in depth" filtration. "In depth" filtration could be achieved by filtering first through coarse medium and then through progressively finer medium (Holding, 1972). This will permit long filter cycles because the headloss increase is relatively uniform due to the uniform distribution of suspended solids throughout the bed depth (Holding, 1972). Thus, a dual media bed (coarse anthracite over fine sand) would increase the capacity of the filter due to "in depth" filtration, allowing longer filter cycles and making more efficient use of the filter bed (Baumann and Huang, 1974). Laboratory, pilot-plant scale and full plant scale results have shown that dual media filters are more efficient than single medium filters made with either of those media (Conley, 1961). Other literature (Cleasby and Sejkora, 1975; Phillips and Shell, 1969) also reported dual media filters to be more effective than conventional single medium sand filters.

## Backwashing, An Important Aspect of Filtration

An important consideration in the design of granular media filter is the periodic backwashing of the filter which is necessary to remove the solids entrained within the filter bed during the filter cycle. As filtration progresses, suspended solids in the iinfluent are removed within the bed which progressively clog the pores within the filter medium and cause increases in headloss. Hence, the frequency of backwash will depend on the hydraulic loading rate and the quality of influent. "In depth" filtration is very desirable for efficient operation, but the subsequent formation of slimes on the media is a problem (Shell and Burns, 1973). These biological slimes would tend to accumulate throughout most of the bed depth because of the pattern of suspended solids removal and also because the media acts as a flooded biological filter (Shell and Burns, 1973). Mixing of chlorine or other disinfectants with the filter influent may control the biological growth. Secondly, there is a tendency for suspended solids to stick to the filter media due to the nature of the particles in wastewater effluents (Holding, 1972; Kreissl, 1974). These two conditions require a very efficient method of backwashing. Filters treating wastewater cannot be backwashed using the same methods as those applied for the filters treating potable water.

Efficient backwashing of the filter is very important because inadequate removal of the filtered suspended solids will cause progressive increase of clean bed headloss, consequently reducing the time of filter cycle, perhaps to a point where complete removal of media will be necessary for thorough cleaning (Holding, 1972). Vigorous agitation of filter media by air scour along with water backwash has proven to be the most successful method of backwashing the filter which treats water containing biodegradable organics (Holding, 1972; Shell and Burns, 1973). Compressed air is first introduced through symmetrically placed nozzles at the bottom of the media at a rate which is sufficient to achieve complete upset and scour the media. This is followed by water backwash to carry away loosened particles. Water backwash rate is quite high, normally 5 to 6 times the normal filtration rate, expanding the bed considerably, thus producing greater voids for efficient removal of the solids. Efficiency of backwash could be greatly increased by using a pulsed system of air scour (Holding, 1972). This is due to the fact that initially introduced air will follow the least resistant path, i.e. least clogged areas. A pulsed system uses intermittent air scour which allows the disturbed bed to settle before another air scour cycle occurs. Thus alternation of the air scour/rest cycle would result in an effective separation of entrained particles from the filter media.

The volume of wash water required is also an important consideration in the filter design. Normally, the filter effluent itself is used for backwashing the filter media. The percentage of filtered water required for backwashing is directly related to the required efficiency of the filter (Holding, 1972). Backwash water requirements are usually less than three to four percent of the filtered water.

## Previous Filtration Investigations

Tebbutt (1971) conducted a filtration study on bacteria bed effluent. Three 89 mm (0.29 ft) diameter x 600 mm (1.97 ft) depth filter columns were used for this pilot-plant study at the University of Birmingham, Birmingham, England. Several different media were tried with different hydraulic loading rates. The main object of the study was to reduce $BOD_5$ and suspended solids. Influent suspended solids concentrations throughout the test were 30 to 40 mg/l. For all the media examined, the filtration rate did not affect suspended solids removal over the range of 100-600 $m^3 m^{-2} d^{-1}$ (1.71 - 10.23 gpm/ft$^2$) hydraulic loading rates. Suspended solids removal ranged from 38 to 70 percent during the tests. A fine sand (0.5 - 1.0 mm effective size) bed nor a dual media (anthracite coal over sand) bed offered any significant improvement in suspended solids removal over an anthracite bed of 1.0 -2.5 mm effective size. Anthracite medium did not prove to be any better than similar grain size sand for suspended solids removal. The author recommended at least a 12-month study on any laboratory or pilot-scale tertiary treatment installation -because of the variable quality of wastewater influent.

Tchobanoglous (1970) conducted a study on tertiary filtration on activated sludge effluent at the Palo Alto wastewater treatment plant. A single medium sand filter, a dual media (anthracite coal and sand) filter, and a multimedia (anthracite coal, sand, and garnet) filter were used in this pilot-plant study. The main purpose was to study the effects of different sizes and different depths of filter media and filtration rate on suspended solids removal. Based on this study, the author concluded that in the design of single medium filters, the two most important variables appear to be the size of the filter medium and whether or not chemicals are added to the filter influent. He inferred that most dual media and multimedia filter beds as presently designed, do not utilize the full bed depth effectively. The study indicated that the suspended solids removal efficiency without chemical addition is primarily a function of grain size and bed depth. Turbidity breakthroughs were not observed within the headloss range of 8 to 10 ft (2.44 to 3.05 m). Polyelectrolytes could be added to filter influent to achieve different degrees of suspended solids removal (Tchobanoglous, 1970).

Baumann and Huang (1974) conducted one of the most complete studies on tertiary filtration of wastewater. The study was undertaken at the Ames, Iowa, Pollution Control Plant, on standard rate trickling filter plant effluent. The pilot-plant consisted of three sets of filters, each containing four filter cells. The filters were 4 inches (10.16 cm) in diameter and in each set the four filters were provided for different depths of media (1 inch, 5 inches, 14 inches, and 24 inches) (2.54, 12.70, 35.56, and 60.96 cm). Anthracite coal and silica sand were used as filter media. The object of the study was to compare the operating characteristics of single medium filters with dual media filters and to examine the effects of media size and hydraulic loading rate on filtration. The study demonstrated selection procedures for the size and depth of filter media. High hydraulic loading rate did not significantly affect filtrate quality, and the suspended solids accumulations within the filter bed had a direct bearing on headloss development. The authors suggested a pilot-plant operation before designing a tertiary filter plant because of variations in the quality of filter influent. A granular media filter plant could be designed as an efficient method of tertiary treatment on the basis of data collected from pilot plant operation (Baumann and Huang, 1974).

# EQUIPMENT AND EXPERIMENTAL PROCEDURES

Central Weber Wastewater Treatment Plant influent consists of a mixture of domestic wastewater, storm water, feedlot waste, irrigation runoff, railroad yard waste, dairy waste and an assortment of industries. The raw wastewater receives preliminary treatment (three mechanically cleaned bar screens and three mechanically cleaned grit chambers) before it is pumped by raw wastewater pumps to the primary treatment units. The primary treatment process consists of three circular tanks 140 ft (42.67 m) in diameter having 8.5 ft (2.59 m) side wall depth. The primary sedimentation tanks provide 1 to 1½ hours of detention time depending on the quantity of wastewater being treated. Effluent from the primary sedimentation tanks flows to the secondary treatment units. Twelve standard rate trickling filters with rotary distributors serve as the biological treatment process. These are circular tanks 230 ft (70.10 m) in diameter with 5.25 ft (1.60 m) side wall depth which are filled with standard sized rocks. Trickling filter effluent then flows to the secondary sedimentation tanks after being chlorinated. Three circular tanks 140 ft (42.67 m) in diameter with 8.5 ft (2.59 m) side wall depth are used to settle out humus from the secondary biological treatment and to allow sufficient chlorine contact time. The secondary sedimentation tanks also provide 1 to 1½ hour detention time and the effluent is discharged into the Weber River. Settled humus from the secondary sedimentation tanks is recirculated into the plant. Primary settled sludge is dewatered by vacuum filtration. Part of the Weber River water downstream of the outfall point is used for irrigation purposes.

## Experimental Apparatus

The pilot SVG filter was installed at the treatment plant to evaluate its performance as a means of polishing the secondary effluent (Figures 1 and 2). The filter is a dual media, gravity flow unit with a self contained automatic backwashing system. On automatic operation, the filter backwashes when the headloss across the filter media reaches a preset maximum headloss level. When the filtered water in the backwash storage compartment reaches a level set by a low level probe, the backwash cycle terminates and the filter cycle starts again through a clean filter bed.

The pilot filter is composed of a 4 feet (1.22 m) diameter tank containing a filter compartment, a collection chamber and a storage compartment which are interconnected through pipe and valve arrangements. The filter compartment contained filter media consisting of 1 foot (0.30 m) of anthracite coal (effective size 1.10 mm, uniformity coefficient 1.60, and specific gravity 1.5) overlying 1 foot (0.30 m) of sand (effective size 0.40 mm, uniformity coefficient 1.50, and specific gravity 2.5) (Figure 3). Filtered water from the filter compartment passes into the collection chamber through a drainage assembly. Eimco FlexKleen nozzles are used as underdrain distributors which are designed to eliminate the necessity of a gravel supporting bed by preventing filter media carry-over into the filtered effluent. The backwash storage compartment stores enough filtered water for backwash purposes, letting the excess out through the effluent pipe.

To help determine the total headloss through the filter media and at various depths of the filter media, seven headloss gages are attached at different points in the filter. Headloss gage number 1 is located 3½ inches (8.89 cm) below the bottom of the filter bed. Headloss gage number 2 is located 7 inches (17.78 cm) above the bottom of the filter bed. The others are tapped at 7 inches (17.78 cm) center-to-center, above gage number 2, and two of the gages are above the filter media.

## Filter Cycle (Refer to Figures 1 and 2)

At the start of a filter cycle, the three way valve is positioned with the influent open and backwash closed. The isolation valve is open, the air inlet valve and the drain valve are closed. A small fraction of the plant effluent is pumped from the secondary sedimentation tank to the top of the filter tank into a splitter box. Adjustment of a weir inside the splitter box admits a predetermined amount of wastewater into the influent pipe and the remaining flows into the overflow pipe to waste. Wastewater entering the influent pipe is conveyed into the filtering compartment through the influent portion of the three way valve. Wastewater is filtered through the filter media and then passes through the drainage assembly into the collection chamber. The transfer pipe, which is equipped with an automatic isolation valve, connects the collection chamber and the storage compartment. The automatic isolation valve allows isolation of the filter media during the air backwash, thus preventing the loss of media. Filtered water from the collection chamber goes to the storage compartment through the transfer pipe. The storage compartment lets the excess filtered water out through an outlet box into the effluent pipe. When wastewater is being filtered, the headloss through the filter bed increases due to clogging of pores in the media by solids in the wastewater. Headloss gages mounted on the outside of the filter tank show the headloss at different depths of the filter media. The rate of increase of headloss will be governed by the influent suspended solids concentration and the loading rate. When the headloss reaches a preset maximum, a pressure switch is actuated which sends an electrical signal to the filter control panel starting the air-water backwash cycle.

## Backwash Cycle (Refer to Figures 1 and 2)

When the backwash cycle is initiated, the three way valve is positioned with the influent closed and backwash open, which is just the opposite of the filter cycle. The isolation valve is closed isolating the storage compartment from the rest of the system. The drain valve is opened and drains part of the water left in the filter compartment and the collection

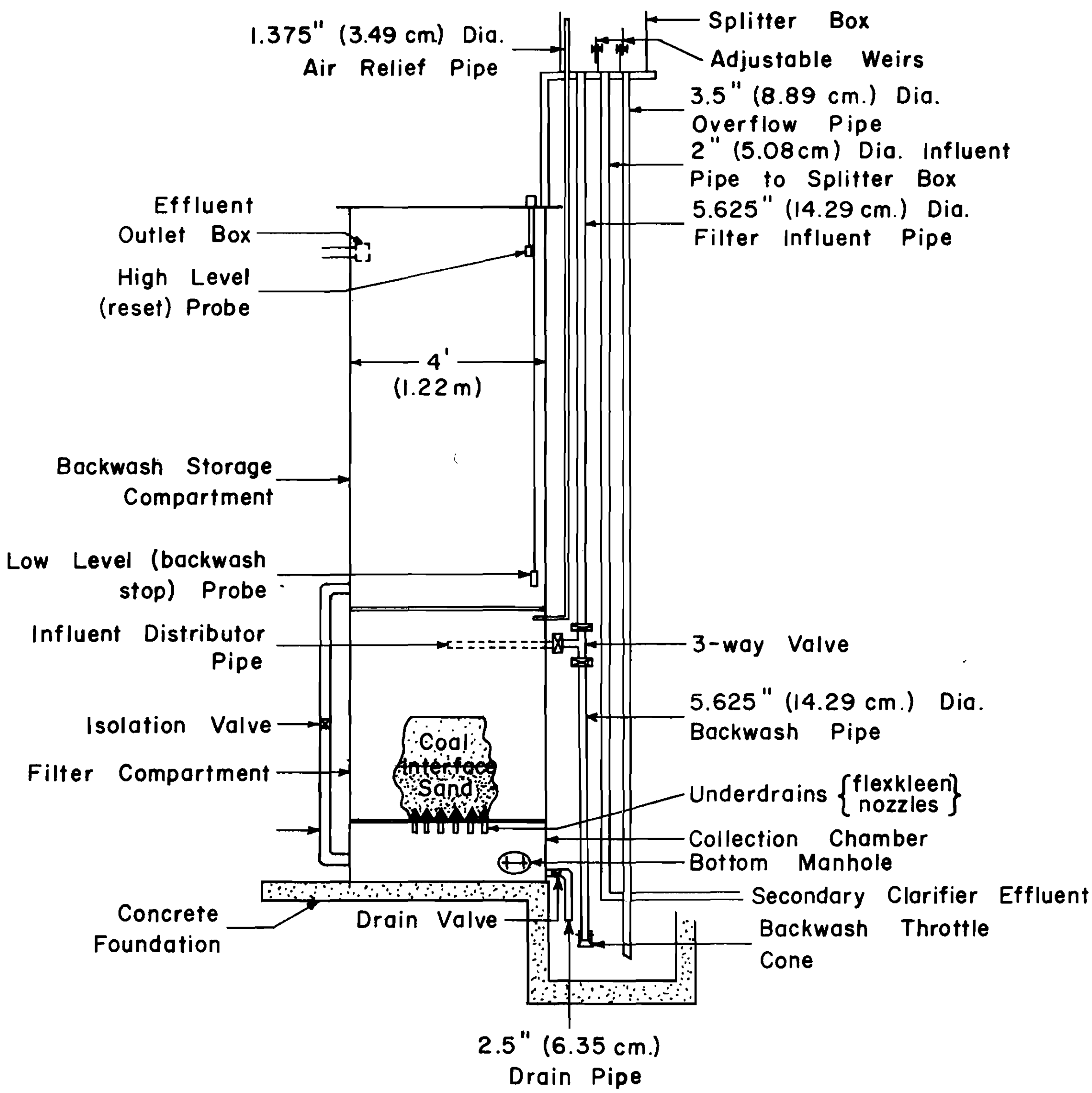


**Figure 1. SVG filter: Front view.**

chamber through a drain pipe. The drain valve is closed after a time that has been preadjusted on the control panel. The air inlet valve is opened after a short delay following closing of the drain valve. A blower[2] starts at the same time blowing the air through a pipe to the collection chamber and then up through the FlexKleen nozzles into the filter compartment, thus agitating the media and loosening the particles trapped within the media. The air backwash time is preadjusted on the control panel after which the air inlet valve is closed. The isolation valve then opens, starting the water backwash cycle. Backwash water held in the storage compartment flows through the transfer pipe into the collection chamber and then up through the FlexKleen nozzles into the filter compartment, agitating and expanding the filter media. The wash water flows into the backwash pipe through the backwash valve, carrying along with it, the loosened particles. The backwash pipe carries the dirty backwash water to a recirculation manhole. The rate of water backwash is controlled by adjustment of a rate control cone at the end of the backwash outlet pipe. Water backwash stops when the water level in the storage compartment reaches a low level probe, terminating the backwash cycle. This actuates the pilot valve causing the three way valve to be re-positioned with the influent open and backwash closed, starting the filter cycle.

[2] Roots Connersville, Connersville, Indiana, Serial No. 6308-687.

## Experimental Procedures

The pilot filter was operated at hydraulic loading rates of 3, 4, 5, and 6 gpm/ft$^2$ (122.10, 162.80, 203.50, 244.20 l/min/m$^2$) for this study. It was backwashed manually before collection of each composite sample. The experiment was carried out in two different stages with two different sizes of media as discussed later. The experimental procedures for both these stages are summarized in Table 1. At each

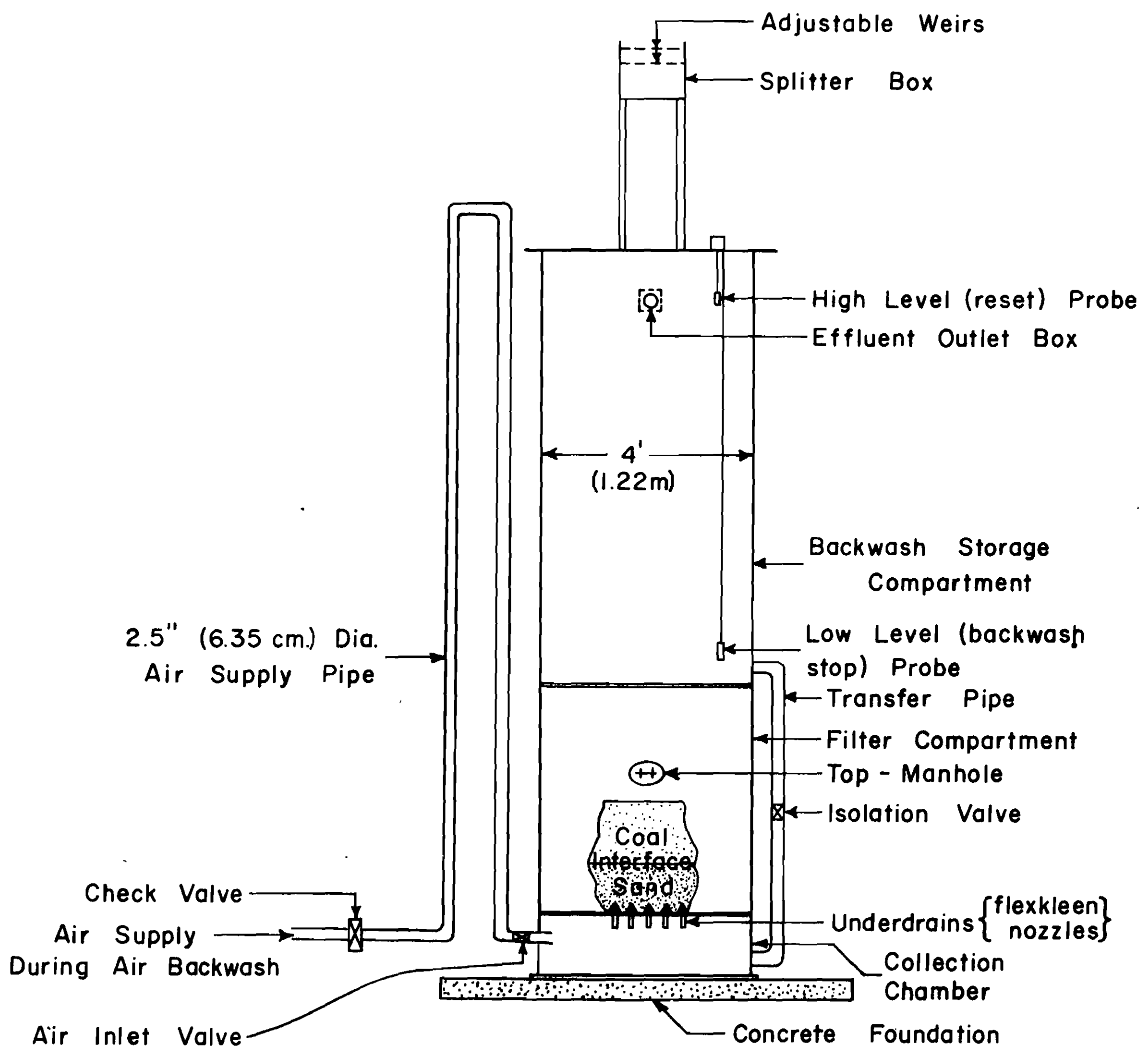


**Figure 2. SVG filter: Side view.**

**Table 1. Details of the experimental procedures.**

| | Filter Media Used | Flow Rate (gpm/ft$^2$)[a] | Sample Collection | Laboratory Analysis Performed | Sample Preservation Technique Used[b] |
|---|---|---|---|---|---|
| 1 | Coal: 1.00 mm e.s., 1.70 u.c., 1 foot depth (0.30 m); Sand: 0.45 mm e.s., 1.70 u.c., 1 foot depth (0.30 m) | 3 | 24 hr. composite | $BOD_5$, SS | Refrigeration at 4°C (39.2°F) |
| | | | | $NO_3$-N, $NO_2$-N, $NH_3$-N, total-P, ortho-P | 40 mg $HgCl_2$/l of sample and refrigeration at 4°C (39.2°F) |
| | | 4 | 24 hr. composite | $BOD_5$, SS | Refrigeration at 4°C (39.2°F) |
| | | | | $NO_3$-N, $NO_2$-N, $NH_3$-N, total-P, ortho-P | 40 mg $HgCl_2$/l of sample and refrigeration at 4°C (39.2°F) |
| | | 3 | 1 filter cycle composite | $BOD_5$, SS | Refrigeration at 4°C (39.2°F) |
| | | | | $NO_3$-N, $NO_2$-N, $NH_3$-N, total-P, ortho-P | 40 mg $HgCl_2$/l of sample and refrigeration at 4°C (39.2°F) |
| 2 | Coal: 1.10 mm e.s., 1.60 u.c., 1 foot depth (0.30 m); Sand: 0.45 mm e.s., 1.50 u.c., 1 foot depth (0.30 m) | 3 | 1 filter cycle composite | $BOD_5$, SS, VSS, soluble $BOD_5$ | Refrigeration at 4°C (39.2°F) |
| | | 4 | 1 filter cycle composite | $BOD_5$, SS, VSS | Refrigeration at 4°C (39.2°F) |
| | | 5 | 1 filter cycle composite | $BOD_5$, SS, VSS | Refrigeration at 4°C (39.2°F) |
| | | 6 | 1 filter cycle composite | $BOD_5$, SS, VSS | Refrigeration at 4°C (39.2°F) |

[a] gpm/ft$^2$ x 40.7 = l/min/m$^2$.

[b] No preservation technique required for SS and VSS tests.

hydraulic loading rate, grab samples of filter influent and effluent were collected and composited over a certain length of time as shown in Table 1. The quantity of a grab sample and the frequency of sample collection were varied according to the length of the filter cycle and the quantity of composite sample needed for the laboratory analyses. The sample preservation techniques (Cox, 1974) and the laboratory analyses (APHA, 1971; Cowan and Porcella, 1971) for these composite samples are summarized in Table 1.

Headloss gage readings were recorded at the time of each grab sample collection. The pressure switch was set such that the filter would backwash at 6.8 feet (2.07 m) (of water) total bed headloss. This was the driving head available for filtration. Filter backwashing was carried out in four steps as shown in Table 2.

To determine the variation of suspended solids concentration in the backwash water, samples of the backwash water were collected at 45 second time intervals during a water-wash cycle and were analyzed for suspended solids. Water level in the storage compartment, at 30 second intervals during a water-wash cycle, was recorded for the backwash rating curve. During filter runs, periodic measurements were made of filtration rate (constant) and filter cycle time was recorded for each run.

## Analytical Techniques

$BOD_5$ concentrations were determined by the modified Winkler method as outlined in Standard Methods (APHA, 1971). Soluble $BOD_5$ tests were carried out after filtering the samples through a glass fiber filter.[3]

Suspended solids and volatile suspended solids concentrations were determined according to Standard Methods (APHA, 1971), using a glass fiber filter[4] to collect suspended solids.

Concentrations of nitrate-nitrogen ($NO_3$-N), nitrite-nitrogen ($NO_2$-N), ammonia-nitrogen ($NH_3$-N), total phosphorus (total-P) and ortho-phosphate (ortho-P) were determined by methods outlined in Standard Methods (APHA, 1971). Procedures followed for these tests are listed in Table 3.

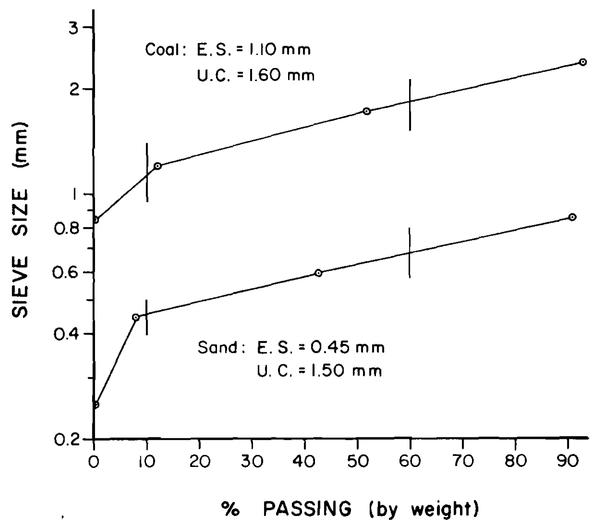


**Figure 3. Sieve analyses of the SVG filter media.**

**Table 2. Filter backwashing procedure.**

| Cycle | Cycle Time (min.) | Average Rate |
|---|---|---|
| Drain cycle | 3.50 min. | |
| Rest | 0.50 min. | |
| Air backwash cycle | 5.00 min. | (1.22 $m^3/min/m^2$) 4 $scfm/ft^2$ |
| Water backwash cycle | 3.75 min. | 15 $gpm/ft^2$ (610.50 $l/min/m^2$) |

**Table 3. Test procedures for nutrient analyses.**

| Parameter and Unit | Analytical Method |
|---|---|
| $NO_3$-N, mg/l | Cadmium-reduction method |
| $NO_2$-N, mg/l | Diazotization method |
| $NH_3$-N, mg/l | Indophenol method |
| Total-P, mg/l | Ascorbic acid method |
| Ortho-P, mg/l | Ascorbic acid method |

[3] W & R Balston Ltd., Whatman No. 1 filters.

[4] Reeve Angel, Clifton, New Jersey, Grade 934 AH, size 4.7 cm.

# DISCUSSION OF RESULTS

Data were collected using two different sizes of media. First, a filter bed consisting of 1 foot (0.30 m) of anthracite coal with an effective size of 1.0 mm and a uniformity coefficient of 1.7 overlying 1 foot (0.30 m) of sand with an effective size of 0.45 mm and a uniformity coefficient of 1.7 was used for the experiment. Twenty-four hour composite samples were collected at the beginning of the experiment, but later it was decided to collect 1 filter cycle composite sample which would be more representative of the actual filter performance. The data from the first phase of the experiment are summarized in Tables A-1 through A-3 in the appendix.

Before the experiment could be completed, the clean bed headloss through the filter increased after each backwash cycle. Very low water backwash rates, which were noticed during the water backwash cycles, indicated clogging of either filter media or FlexKleen nozzles. Upon opening the top manhole, the media were found to be completely clogged with a zoogloeal (sphaerotilus) mass. The same slimy microbial growth, also found on the surface of the air extension tubes, developed because a chlorine residual was not maintained in the filter effluent.

During the summer months, the filter influent chlorine residual depletes very rapidly through the filter bed because of higher influent $BOD_5$ and suspended solids, and also because of decreased chlorine solubility in warm water. Chlorine residual in the filter effluent during the summer was close to zero most of the time but was greater than 0.2 mg/l during winter. Hence, during the summer, there is no residual chlorine in the wastewater to prevent growth in the filter bed. The influent wastewater (part of which is domestic waste) contains sufficient nutrients to support microbial growth. Moreover, the filter media provide a surface for the growth of microorganisms and the temperature during summer is optimum for this type of microbial growth. Thus, the filter media act as a flooded biological filter. Chlorine or other disinfectants must be added to the filter influent in sufficient dosage to insure as a minimum a detectable residual in the filter effluent.

After the occurrence of the growth problem, the media in the experimental filter were treated with strong chlorine solution and then later flooded with strong chlorine solution and copper sulfate solution over a weekend. This did not solve the problem because, not only was the media clogged, but the underdrains (FlexKleen nozzles) were clogged too. The filter media and the FlexKleen nozzles were then removed from the filter compartment. The FlexKleen nozzles were soaked in muratic acid[5] overnight and were then thoroughly cleaned (especially steel mesh) with compressed air. The filter media were completely replaced by new media which consisted of 1 foot (0.30 m) of 1.10 mm effective size and 1.60 uniformity coefficient anthracite coal and 1 foot (0.30 m) of 0.45 mm effective size and 1.50 uniformity coefficient sand (Figure 3).

Experimental data which were collected with new filter media are presented in Tables A-4 through A-7 in the appendix. All samples were 1 filter cycle composites. Nutrient analyses were not performed during this phase of the experiment. As shown in Tables A-1 through A-3 (Appendix), no significant change in the concentration of any nutrient was produced by filtration without chemicals.

## Biochemical Oxygen Demand

As shown by the data in the appendix (Tables A-1 through A-7), the experimental filter was not very efficient in removing $BOD_5$. However, after installing the more uniform and, a filter effluent meeting the State of Utah standards of 10 mg/l of $BOD_5$ was produced at a hydraulic loading rate of 3 gpm/ft$^2$, and the standard was exceeded by less than 2 mg/l at the loading rates of 4, 5, and 6 gpm/ft$^2$. Beccause much of the $BOD_5$ is attributable to fine volatile suspended solids passing the filter, it is likely that the selection of other sands (smaller effective size and uniformity coefficient) will produce an acceptable effluent at all loading rates.

Soluble $BOD_5$ in the filter influent was fairly high, ranging from 26 percent to 44 percent of the total $BOD_5$. Another reason for the high effluent $BOD_5$ was high percentage of volatile suspended solids in the effluent. As shown by the data in the appendix (Tables A-4 through A-7), volatile suspended solids in the effluent ranged from 71

[5]Bailey's Kim-Ko Inc., Ogden, Utah.

percent to 90 percent of the total suspended solids. Most of the colloidal solids passing through the filter were evidently organic solids. Thus, high soluble $BOD_5$ and high colloidal volatile suspended solids concentrations in the filter influent contributed to the poor efficiency of the filter in removing $BOD_5$. Large variations in the total $BOD_5$ removed by the filter is due to the variable nature of the soluble $BOD_5$ which passes through the filter unaffected. The variable nature of the soluble $BOD_5$ also contributed to poor correlation between influent $BOD_5$ and effluent $BOD_5$, and $BOD_5$ and volatile suspended solids.

## Suspended Solids

As shown by Figures 4 and 5, filtration rate did not affect the suspended solids removal efficiency which agrees with previous investigations (Baumann and Huang, 1971). Influent suspended solids concentrations ranged from 19 mg/l to 45 mg/l during the study. One of the secondary sedimentation tanks was being repaired during the first part of May which caused high influent suspended solids concentrations during that time. Effluent suspended solids concentrations of less than 10 mg/l were produced during the study with the exception of three experiments out of a total of 44 experiments with the more uniform media. The three concentrations (10.5, 11.2, and 11.7 mg/l) only slightly exceeded the State of Utah standard of 10 mg/l. Percentage suspended solids removal ranged between 70 to 80 percent most of the time. Volatile suspended solids contributed about 70 percent to 80 percent of influent and effluent suspended solids. As Figure 6 shows, the percent volatile suspended solids in the filter influent and effluent were fairly constant throughout the experiment. Total suspended solids accumulation during a filter cycle decreased with increasing hydraulic loading rates because of short filter cycles at high hydraulic loading rates. Accumulation was unaffected by influent suspended solids concentration.

## Headloss Development

Headlosses through the filter bed, as a function of time for different hydraulic loading rates, are shown in Figures 7 through 10. Headloss increased very rapidly with time at high filtration rates. Relationships between clean bed pressure differential and bed depth at various hydraulic loading rates are shown in Figures 11 through 14. The clean bed headloss, as anticipated, increased as the filtration rate was increased. These figures also show typical terminal pressure differential as a function of bed depth at various hydraulic loading rates. These figures demonstrate the fact that the suspended solids in the filter influent were removed in the top few inches of coal at a low hydraulic loading rate of 3 gpm/ft² (122.10 l/min/m²), but were pushed down further into the media at higher hydraulic loading rates. This led to very short filter cycles at hydraulic loading rates higher than 3 gpm/ft² (122.10 l/min/m²) as explained in the following section.

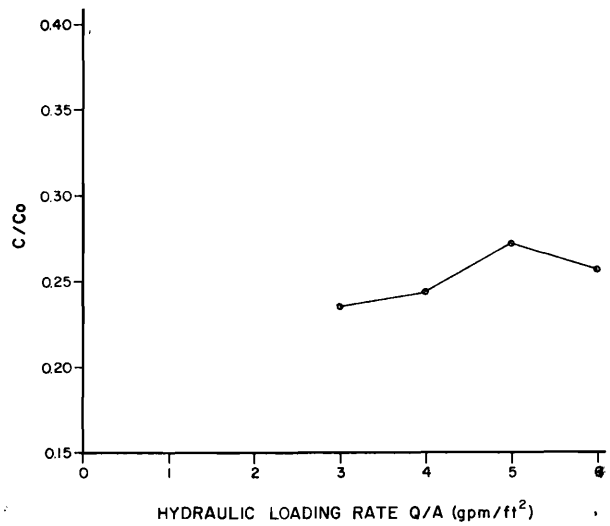


**Figure 4. Suspended solids concentration ratios ($C/C_o$) as a function of hydraulic loading rate.**

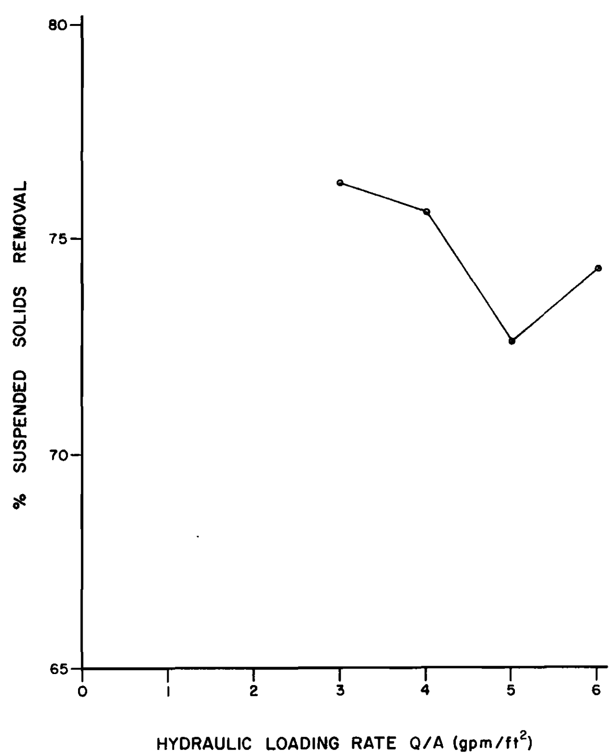


**Figure 5. Percent suspended solids removal as a function of hydraulic loading rate for various influent SS concentrations.**

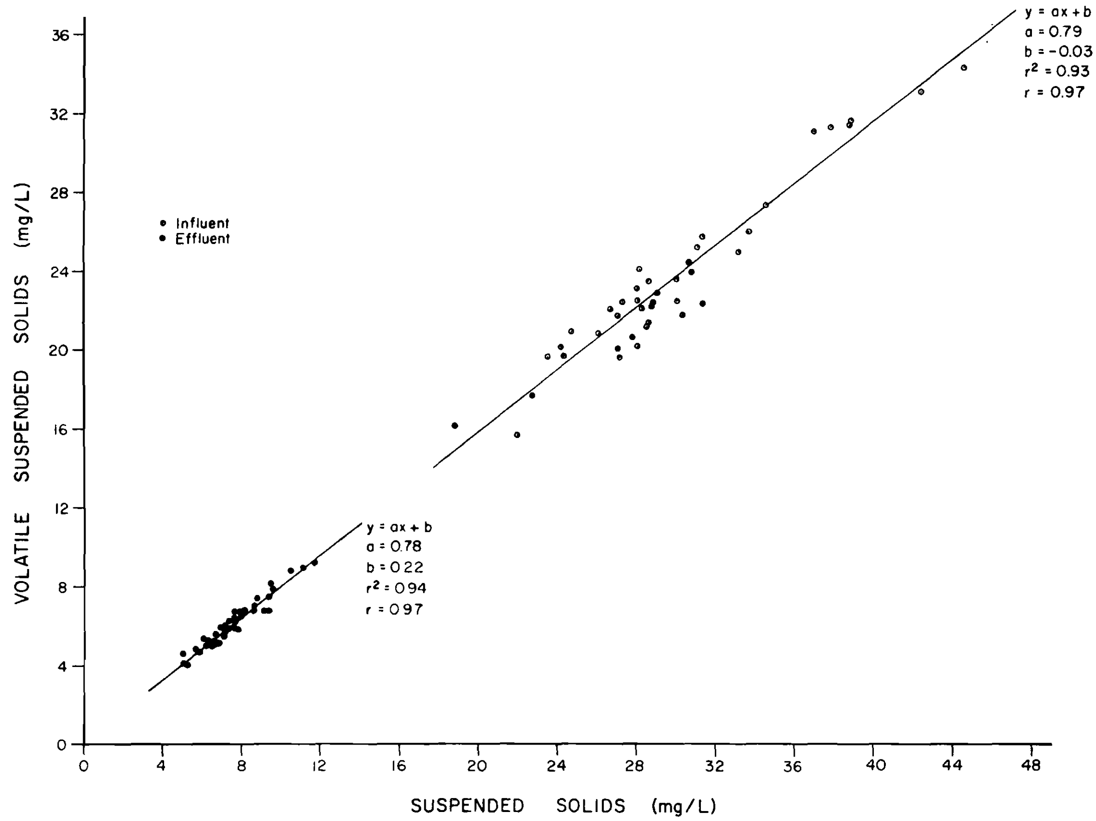


**Figure 6. Volatile suspended solids as a function of suspended solids for influent and effluent.**

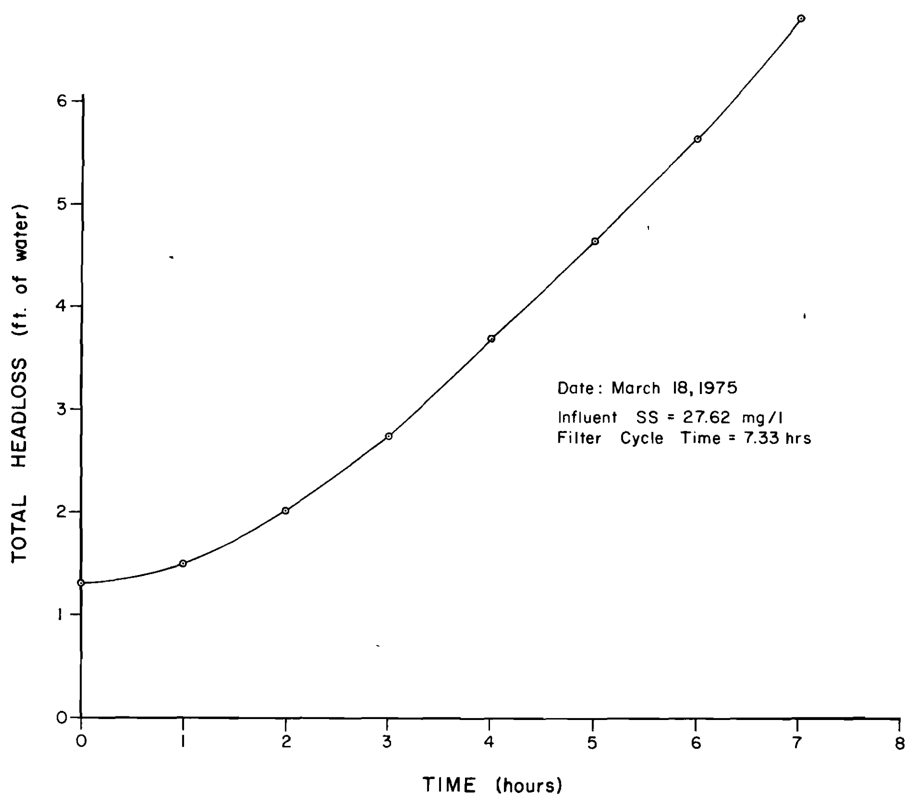


**Figure 7. Total headloss as a function of time for hydraulic loading rate = 3 gpm/ft$^2$.**

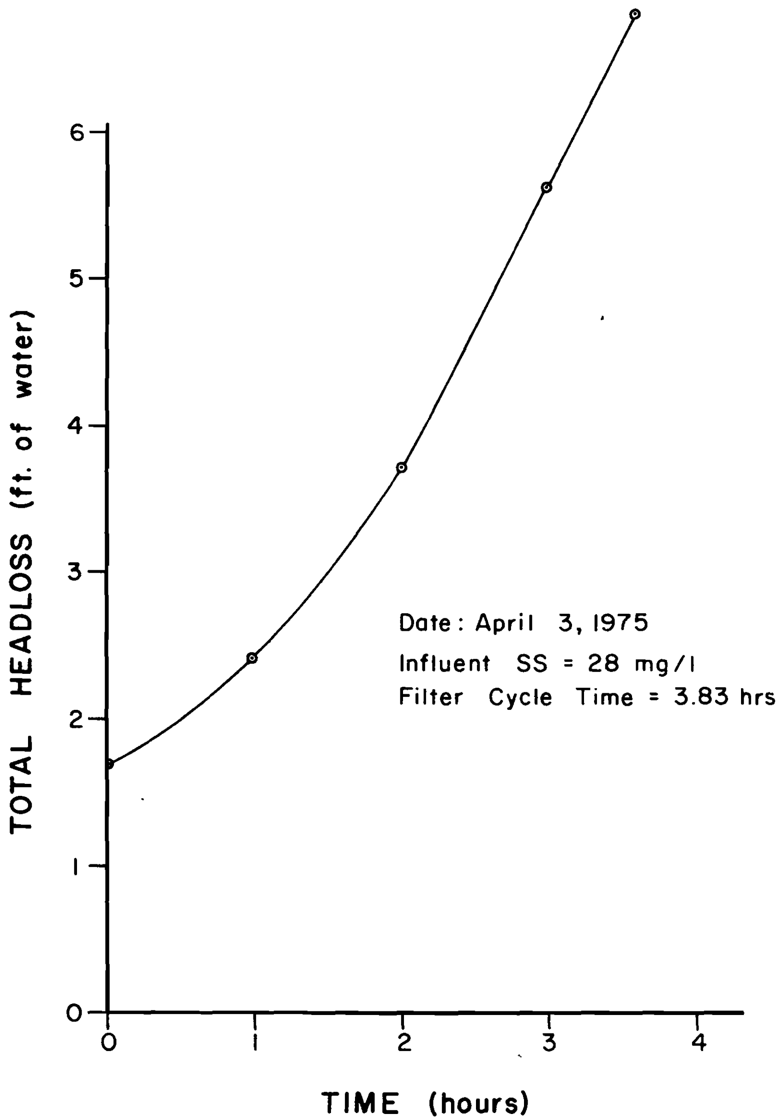


**Figure 8.** Total headloss as a function of time for hydraulic loading rate = 4 gpm/ft$^2$.

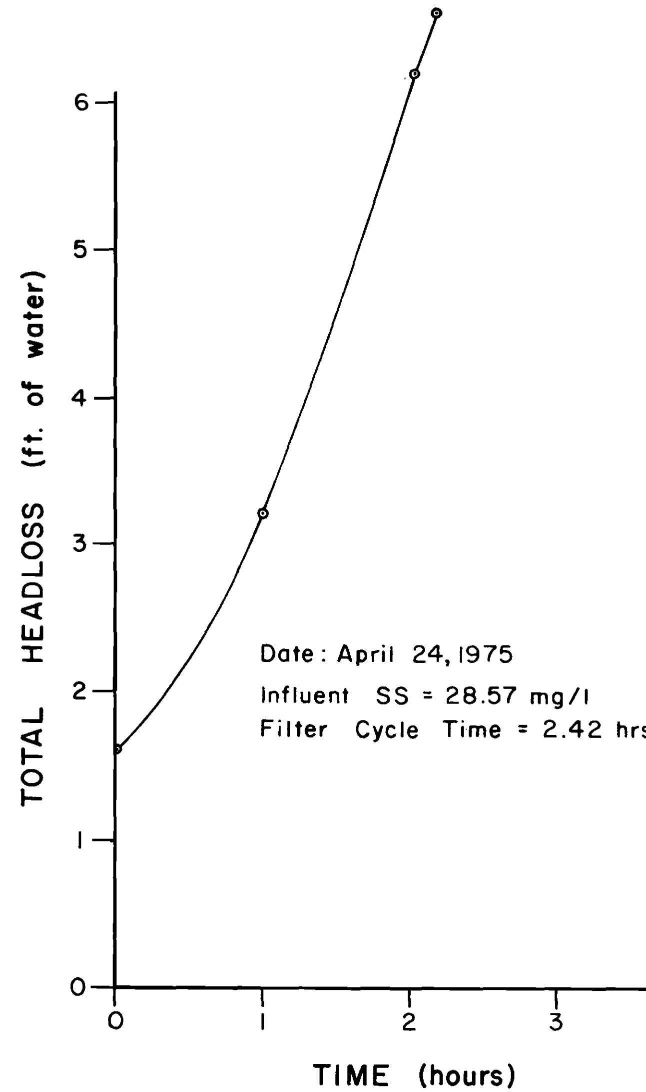


**Figure 9.** Total headloss as a function of time for hydraulic loading rate = 5 gpm/ft$^2$.

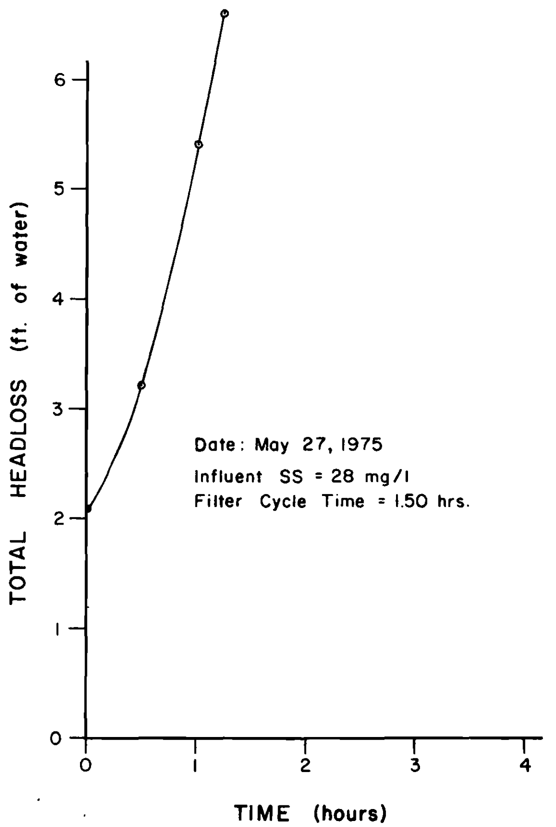


Figure 10. Total headloss as a function of time for hydraulic loading rate = 6 gpm/ft$^2$.

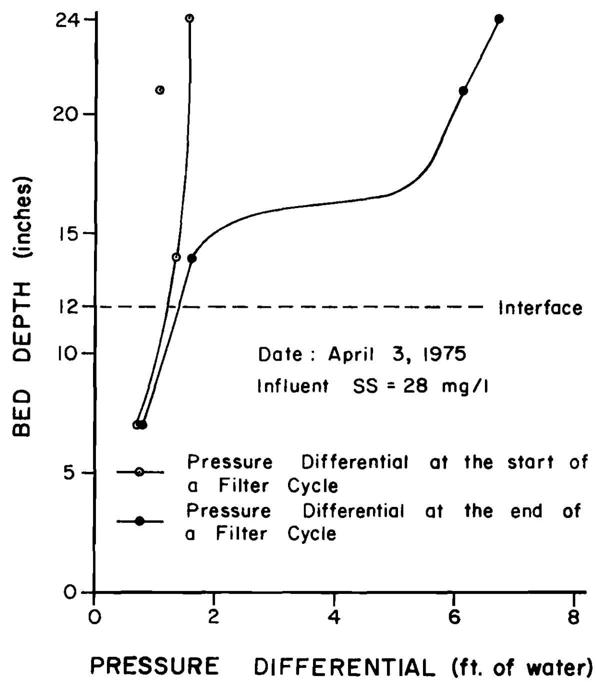


Figure 12. Pressure differential as a function of bed depth at hydraulic loading rate = 4 gpm/ft$^2$.

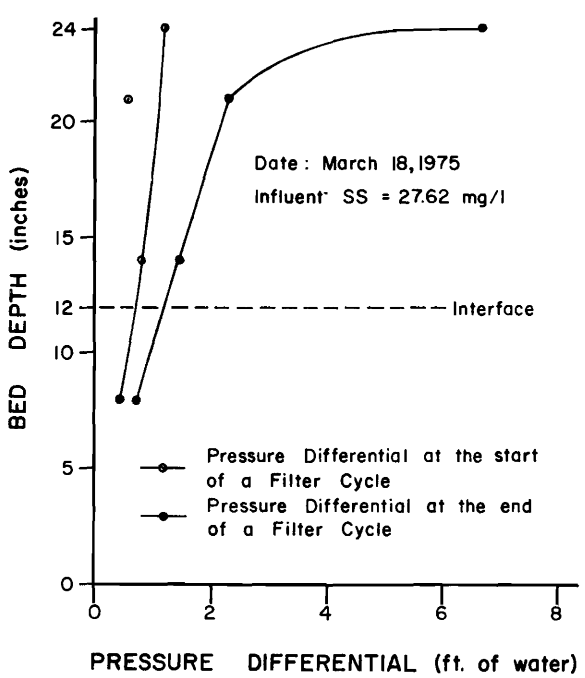


Figure 11. Pressure differential as a function of bed depth at hydraulic loading rate = 3 gpm/ft$^2$.

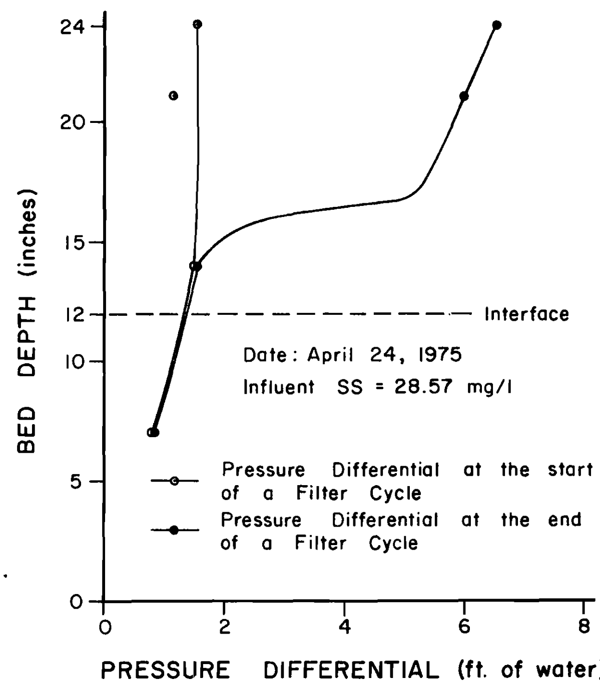


Figure 13. Pressure differential as a function of bed depth at hydraulic loading rate = 5 gpm/ft$^2$.

### Filter Cycle Length

Run length was affected by hydraulic loading rate as shown in Figure 15. Filter cycle time was reduced greatly when the hydraulic loading rate was increased from 3 gpm/ft² (122.10 l/min/m²) to 4 gpm/ft² (162.80 l/min/m²), but the influence of hydraulic loading rate on cycle time was not as great from 4 gpm/ft² (162.80 l/min/m²) through 6 gpm/ft² (244.20 l/min/m²). The possible reason for these short filter cycle runs at hydraulic loading rates higher than 3 gpm/ft² (122.10 l/min/m²) can be explained by referring to the pressure differential curves in Figures 11 through 14. As shown by Figure 11, at 3 gpm/ft² (122.10 l/min/m²), most of the solids removed by the filter media were trapped in the top few inches of coal. Figures 12 through 14 suggest that at high hydraulic loading rates (higher than 3 gpm/ft²), due to the higher velocity of the influent through the filter bed, most of the solids were driven down further into the filter bed. Core samples of the filter media, as shown in Figure 16, taken immediately after a backwash cycle, showed that the intermixing of filter media took place up to about 6 inches (15.24 cm) from the interface. This caused the top fine sand to fill up the void spaces present in the bottom coarse coal which in turn reduced the solids storage capacity of the bottom coarse coal. Apparently, at hydraulic loading rates greater than 3 gpm/ft² (122.10 l/min/m²) the influent solids were driven down to this intermixed

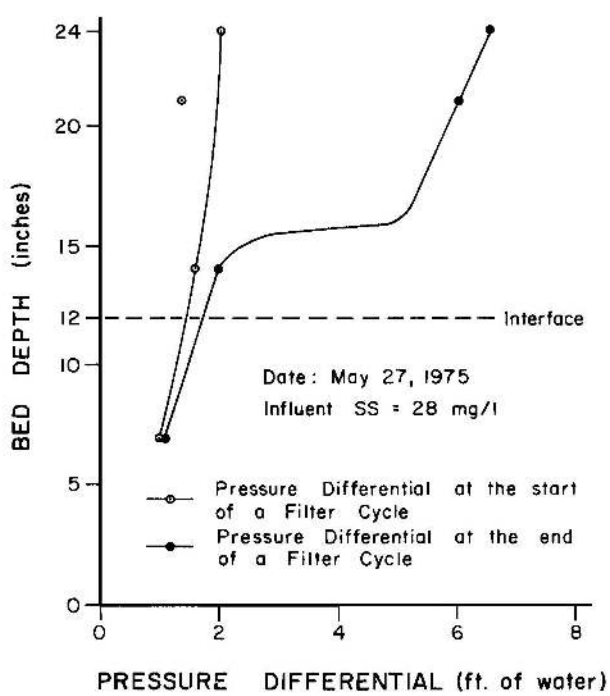


Figure 14. Pressure differential as a function of bed depth at hydraulic loading rate = 6 gpm/ft².

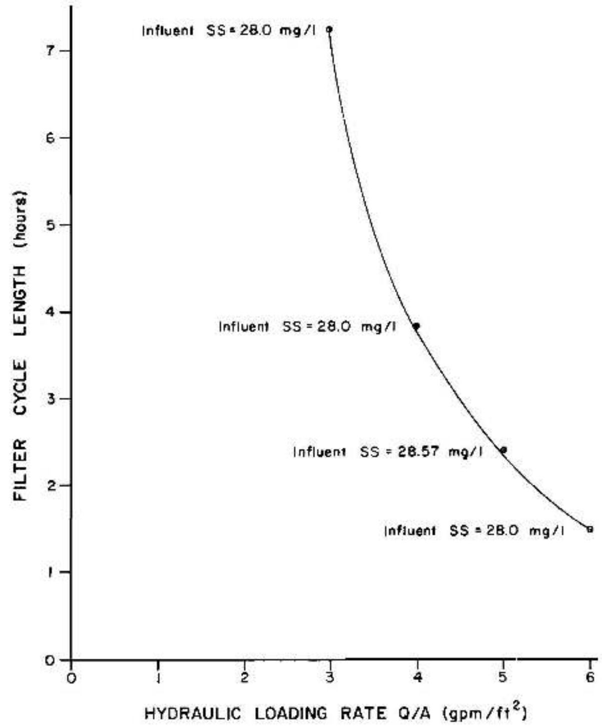


Figure 15. Filter cycle length as a function of hydraulic loading rate Q/A.

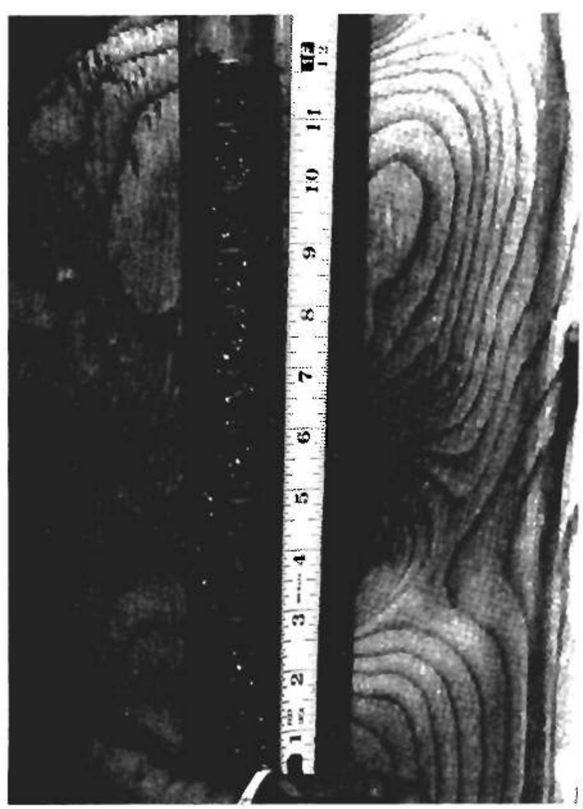

Figure 16. Core sample of the filter media after a backwash cycle.

portion of the filter media and due to less storage capacity in this portion of the bed, headloss increased very rapidly. This caused short duration filter cycles at high hydraulic loading rates.

By increasing the depth of coal in the filter or by using different size media to achieve less intermixing, higher filter cycle lengths could be accomplished at higher hydraulic loading rates. If the total depth of coal is increased to such an extent that most of the influent solids are trapped within the non-mixed portion of the filter media, longer duration filter cycles could be achieved at a hydraulic loading rate of 4 gpm/ft$^2$ (162.80 l/min/m$^2$). Again, if the filter media sizes are selected in such a way that intermixing is minimized, most of the influent solids at 4 gpm/ft$^2$ (162.80 l/min/m$^2$) could be removed within the non-mixed portion of the media. This would result in longer duration filter cycles.

A poor relationship exists between influent suspended solids concentration and the duration of filter cycles. The relationship between net filtered water production and filtration rate is shown in Figure 17. Net filtered water production is directly proportional to the duration of the filter cycle. The abrupt change at hydraulic loading rates greater than 3 gpm/ft$^2$ (122.10 l/min/m$^2$) can be explained by the intermixing of media as discussed above.

## Filter Backwash

The experimental filter was backwashed when the headloss through the filter bed reached 6.8 feet (2.07 m) of water, the head available for filtration. Filter backwashing was carried out in three different steps as mentioned earlier. Clean bed headloss as shown in Figure 18, for a specific hydraulic loading rate, remained almost constant after each backwash cycle. A constant initial headloss indicated that the backwashing procedure followed in this experiment was adequate to clean the filter bed, but at 6 gpm/ft$^2$ (244.20 l/min/m$^2$) a higher initial headloss occurred. This is probably attributable to the higher hydraulic loading rate.

The percentage of filtered water used for backwashing is a very important criterion in the filter design. Figure 19 shows the relationship of this parameter to the filtration rate during the study. Percentage lost in backwash is inversely related to the duration of the filter cycle. Again, the elimination of intermixing of the media would probably tend to lessen the slope of the line shown in Figure 19. Generally, a well designed filter uses 3 to 4 percent or less of filtered water for backwash purposes. Utilizing this criterion, only the 3 gpm/ft$^2$ (122.10 l/min/m$^2$) hydraulic loading rate is satisfactory (Figure 19). The SVG filter used less than 5 percent of the filtered water for backwash at 3 gpm/ft$^2$ (122.10 l/min/m$^2$).

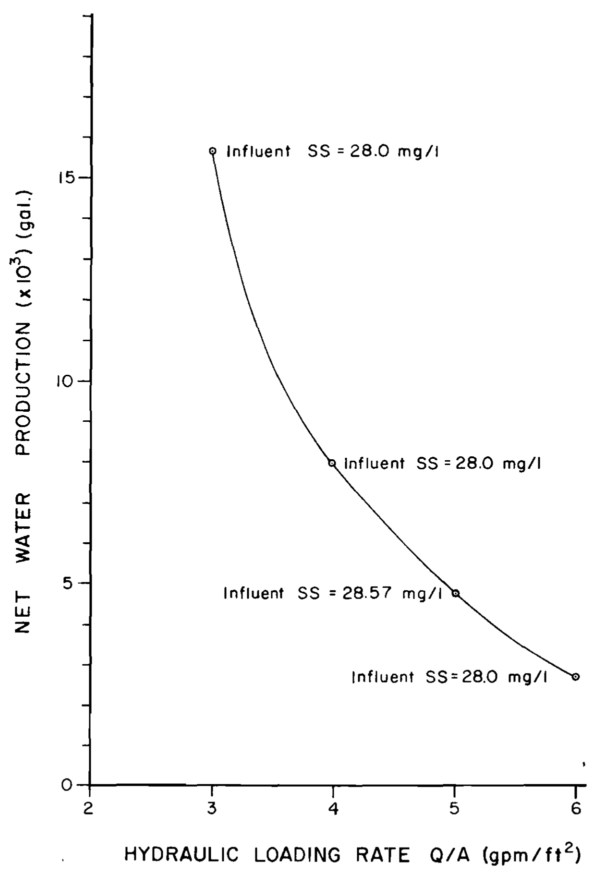


**Figure 17. Net water production as a function of hydraulic loading rate Q/A.**

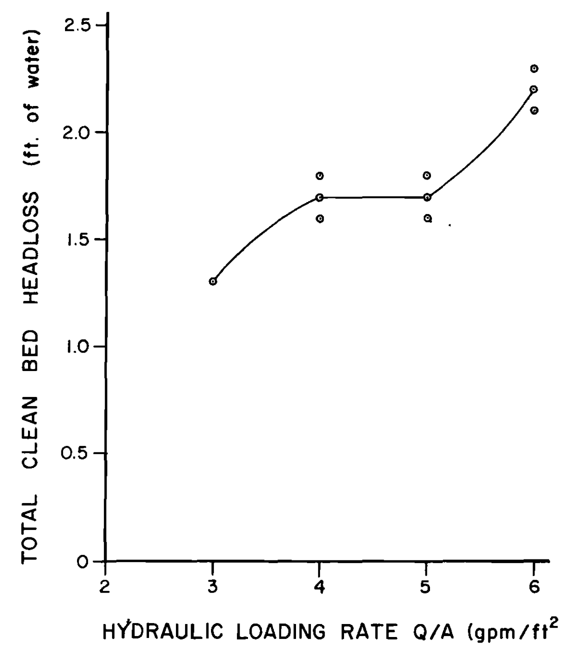


**Figure 18. Total clean bed headloss as a function of hydraulic loading rate.**

Hydraulic loading rates of 5 gpm/ft² (203.50 l/min/m²) and 6 gpm/ft² (244.20 l/min/m²) are undesirable because the filtered water lost in backwash exceeded 10 percent of production most of the time. At 4 gpm/ft² (162.80 l/min/m²), the volume of washwater ranged between 5 and 10 percent of production with one exception. As explained in the previous section, if the total depth of coal in the filter is increased or if the finer coal is used to promote less intermixing of the filter media, the filtration rate of 4 gpm/ft² (162.80 l/min/m²) would probably give more promising results.

Figure 20 shows the variation of suspended solids concentrations in the backwash water as a function of backwash time. The lower portion of the curves converge indicating that the filter bed was cleaned to approximately the same extent at the end of each backwash cycle regardless of the amount of solids trapped in the filter media.

The rate of water backwash as a function of backwash time is shown in Figure 21. High water backwash rates at the beginning of the water backwash cycle were able to carry away most of the solids trapped in the filter bed.

## Chemical Addition

The addition of aluminum sulfate [$Al_2(SO_4)_3 \cdot 14H_2O$] alone and aluminum sulfate and polymer to the filter influent was evaluated. Optimum chemical dosages were determined using jar tests. When dosages of coagulant determined with the jar tests were applied in the field evaluations, post-

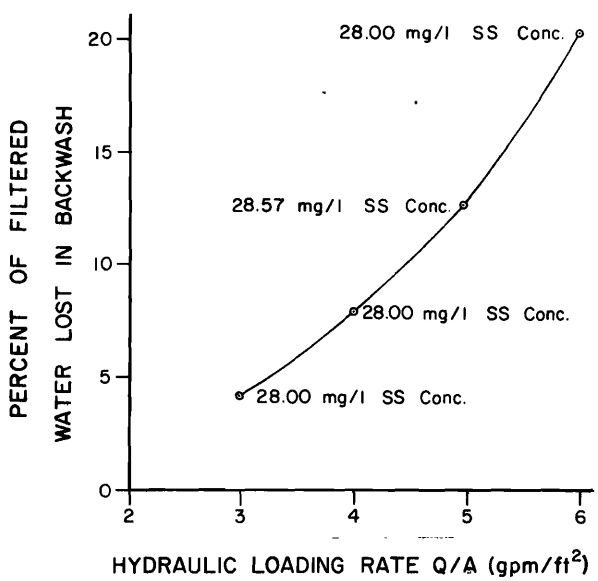


**Figure 19. Percent of filtered water lost in backwash as a function of hydraulic loading rate Q/A.**

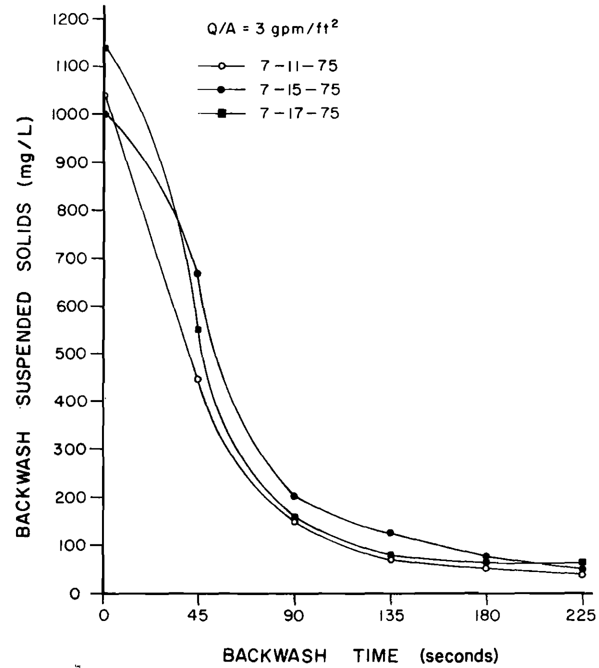


**Figure 20. Backwash suspended solids concentrations as a function of the duration of backwash.**

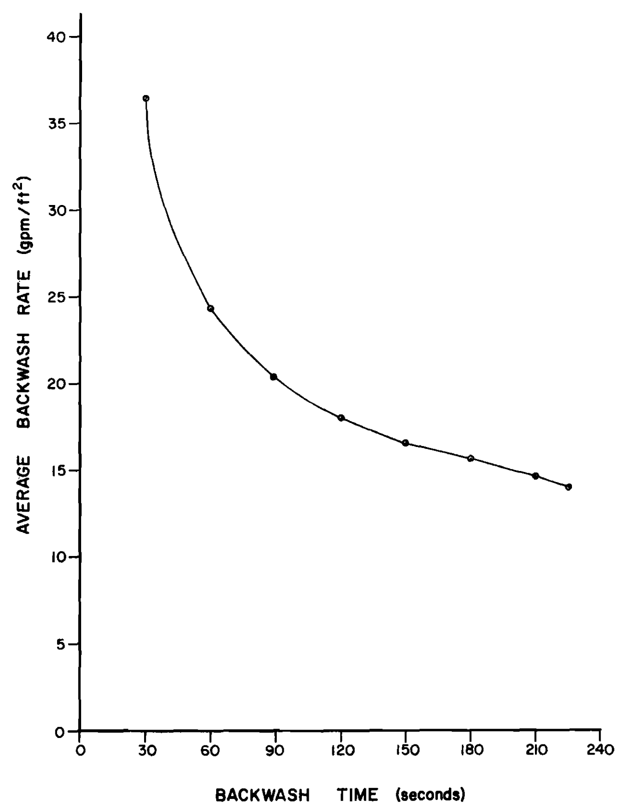


**Figure 21. Average backwash rate as a function of backwash time.**

flocculation occurred. Alum flocs were observed in the effluent in the storage compartment. This could have been caused by overdosing or insufficient mixing time. Overdosing could occur if the conditions in the jar tests differ from field conditions (Habibian and O'Melia, 1975). Chemical addition to the filter influent also led to very short filter cycles. Filter runs with chemical addition, at 3 gpm/ft$^2$ (122.10 l/min/m$^2$) hydraulic loading rate, were reduced by more than 50 percent. Chemical addition was eliminated from the study because of the significant decrease in the duration of a filtration cycle and because nutrient reduction is not currently required. With the solids reduction the major concern, it would be economically infeasible to operate large scale filters with very short filter cycles.

## Calculations Involved in a Filter Design

The principal design criteria for a granular media filter are the quantity and characteristics of the wastewater. The following example is based upon the maximum hydraulic loading rate which occurs during a 24-hour period and the maximum suspended solids concentration expected in the filter influent. The design is based upon the assumption that the filter media to be used in the full scale filters are the same as those used in the second phase of the study and that chemicals will not be added to the filter influent. The design parameters used in this example are summarized in Table 4.

The following example uses 40 feet diameter SVG filters:

Required filter capacity = Design flow + Recycled backwash water

= [60+0.05 (60)] mgd

= 63 mgd

Nominal filtration rate = 3 gpm/ft$^2$

Nominal run length = 6.5 hours [Table A-4, appendix]

Nominal filter area required $= \dfrac{63 \times 10^6}{60 \times 24 \times 3}$

= 14,583 ft$^2$

$\simeq$ 14,600 ft$^2$

Number of filters required $= \dfrac{14600}{\pi r^2}$

$= \dfrac{14{,}600}{3.14 \times (20)^2}$

= 11.62

= 12 units

Actual available filter area with 12 units $= 12 \times \pi \times (20)^2$

= 15,072 ft$^2$

Actual filtration rate $= \dfrac{63 \times 10^6}{1440 \times 15{,}072}$

= 2.90 gmp/ft$^2$

**Table 4. Design parameters used in example.**

| | Parameter | Design Value |
|---|---|---|
| 1 | Maximum 24-hour raw wastewater flow | 60 mgd |
| 2 | Design filtration rate | 3 gpm/ft$^2$ (122.10 l/min/m$^2$) |
| 3 | Maximum suspended solids concentration expected in the filter influent | 30 mg/l |
| 4 | Filter media used | Coal: 1.10 mm e.s.<br>1.60 u.c.<br>1 foot depth (0.30 m)<br>Sand: 0.45 mm e.s.<br>1.50 u.c.<br>1 foot depth (0.30 m) |
| 5 | Backwashing procedure | Drain cycle: 3.50 min<br>Rest: 0.50 min<br>Air backwash cycle: 5.00 min at 4 scfm/ft$^2$ avg. rate<br>Water backwash cycle: 3.75 min at 15 gpm/ft$^2$ avg. rate |
| 6 | Backwash water recycled | 5% of filtered water |

When one of the filters is out of service or backwashing:

$$\text{Available filter area} = 11 \times \pi \times 400$$

$$= 13{,}816 \text{ ft}^2$$

$$\text{Filtration rate} = \frac{63 \times 10^6}{1440 \times 13{,}816}$$

$$= 3.17 \text{ gpm/ft}^2$$

When one of the filters is out of service and one is backwashing:

$$\text{Available filter area} = 10 \times \pi \times 400$$

$$= 12{,}560 \text{ ft}^2$$

$$\text{Filtration rate} = \frac{63 \times 10^6}{1440 \times 12{,}560}$$

$$= 3.48 \text{ gpm/ft}^2$$

Thus, if the hydraulic loading rate of 3 gpm/ft$^2$ (122.10 l/min/m$^2$) is critical, 14 units should be provided so that 12 units will still be in operation when one filter is out of service and one is backwashing, keeping the hydraulic loading rate at 2.90 gpm/ft$^2$. With 14 units, a hydraulic loading rate of 3 gpm/ft$^2$ (122.10 l/min/m$^2$) would be exceeded only under severe operating conditions. If an occasional hydraulic loading rate in excess of 3 gpm/ft$^2$ is acceptable, 12 or 13 filter units should be adequate.

# COST ANALYSIS

Cost estimation of any system involves past experience, data from other similar projects and data from the pilot plant studies. It is very difficult to estimate the actual cost of a large scale system because of the large number of variations involved and ever rising cost of construction. All of the problems involved in a large scale system may not be realized in a pilot plant study.

The details of the cost estimation of a large scale dual media-SVG-filtration process for the Central Weber Wastewater Treatment Plant are shown in estimates 1 through 4 in the appendix. A summary of the cost estimates are presented in Tables 5 and 6. Total cost estimates vary from 2.26 to 2.57 cents per 1000 gallons of wastewater treated.

Costs related to the SVG filters were based on data provided by the Eimco Division of Envirotech, Salt Lake City, Utah. Costs pertaining to backwash storage lagoons and recycle pumps were taken from the EPA manual on cost estimation and manpower requirements for conventional wastewater treatment facilities (U.S. EPA, 1971). A maximum of one day detention time was assumed in determining the capacity of the backwash water storage lagoons. Total pumping capacity of backwash water recycle pumps was based on the assumption that all the backwash water collected in one day would be pumped back at the head of the plant (recirculated into the plant) during the 12 hour period of low flow. Backwash water storage and pumping facilities for 3 gpm/ft$^2$ filtration rate were not included in the cost analyses because of fairly insignificant amounts of backwash water to be handled at this loading rate.

**Table 5. Summary of cost estimation (with federal assistance).**

| Filtration Rate (gpm/ft$^2$) | Capital Cost ($) | Annual Debt Service ($) | Annual O & M Cost ($) | Total Annual Cost ($) | Debt Service (¢/1000 gal) | O & M Cost (¢/1000 gal) | Total Cost (¢/1000 gal) |
|---|---|---|---|---|---|---|---|
| 3 | 4,560,200 | 95,400 | 181,900 | 277,300 | 0.44 | 0.83 | 1.27 |
| 4 | 3,978,100 | 83,300 | 187,200 | 270,500 | 0.38 | 0.85 | 1.23 |
| 5 | 3,687,600 | 77,200 | 185,600 | 262,800 | 0.35 | 0.85 | 1.20 |
| 6 | 3,743,500 | 78,400 | 193,600 | 272,000 | 0.36 | 0.88 | 1.24 |

**Table 6. Summary of cost estimation (without federal assistance).**

| Filtration Rate (gpm/ft$^2$) | Capital Cost ($) | Annual Debt Service ($) | Annual O & M Cost ($) | Total Annual Cost ($) | Debt Service (¢/1000 gal) | O & M Cost (¢/1000 gal) | Total Cost (¢/1000 gal) |
|---|---|---|---|---|---|---|---|
| 3 | 4,560,200 | 381,600 | 181,900 | 563,500 | 1.74 | 0.83 | 2.57 |
| 4 | 3,978,100 | 332,900 | 187,200 | 520,100 | 1.52 | 0.85 | 2.37 |
| 5 | 3,687,600 | 308,600 | 185,600 | 494,200 | 1.41 | 0.85 | 2.26 |
| 6 | 3,743,500 | 313,300 | 193,600 | 506,900 | 1.43 | 0.88 | 2.31 |

# SUMMARY AND CONCLUSIONS

This study was conducted to evaluate the performance of a dual media filtration system serving as a polishing step for a secondary wastewater treatment plant effluent. Filter cycle duration and the percentage of filtered water used in the backwash indicated that 3 gpm/ft$^2$ (122.10 l/min/m$^2$) was the most efficient of all the filtration rates studied. Hydraulic loading rate did not affect suspended solids removal over the range of 3 to 6 gpm/ft$^2$ (122.10 to 244.20 l/min/m$^2$). The filter media evaluated produced excellent suspended solids removal with effluent concentrations less than 10 mg/l the majority of the time. The percentage of $BOD_5$ removed was generally poor because of the high concentrations of soluble $BOD_5$ and colloidal organic solids in the filter influent. However, the effluent quality satisfied the State of Utah standard of 10 mg/l at a hydraulic loading rate of 3 gpm/ft$^2$ and exceeded the standard by less than 2 mg/l at loading rates of 4, 5, and 6 gpm/ft$^2$.

The majority of the influent suspended solids were removed in the top few inches of coal at 3 gpm/ft$^2$ (122.10 l/min/m$^2$) but, at 4, 5, and 6 gpm/ft$^2$ (162.80, 203.50, and 244.20 l/min/m$^2$) solids were forced deeper into the bed where they were removed by the intermixed portion of the media. The nature of the suspended solids accumulations caused very rapid headloss development at hydraulic loading rates higher than 3 gpm/ft$^2$ (122.10 l/min/m$^2$) which resulted in very short filtering cycles at 4, 5, and 6 gpm/ft$^2$ (162.80, 203.50, and 244.20 l/min/m$^2$).

Addition of aluminum sulfate [$Al_2(SO_4)_3 \cdot 14H_2O$] and a cationic polymer in conjunction with alum to the filter influent resulted in very short filter cycles. Filtration without the addition of chemicals had very little effect, if any, on the concentration of nutrients in the filter effluent.

The quality of the final plant effluent varies, because of the variable nature of the influent wastewater and also because of the variations in the treatment efficiencies of the preceding units. Hence, a pilot-plant operation should be carried out for at least 12 months on any tertiary treatment unit to approach optimum economic design.

The following conclusions were derived from the evaluation of the SVG pilot-scale tertiary filtration study:

1. Tertiary filtration of secondary wastewater treatment plant effluent to meet effluent quality standards is economically feasible.
2. Effluents containing less than 10 mg/l of $BOD_5$ and SS can be produced by granular media filtration when a good quality secondary effluent ($<30$ mg/l of $BOD_5$ and SS) is applied to the filters.
3. If large scale filters are designed based upon this study, 3 gpm/ft$^2$ (122.10 l/min/m$^2$) appears to be the optimum design filtration rate based on effluent quality, but economic considerations indicate that 5 gpm/ft$^2$ is the optimum.
4. Filter cycles of greater duration could be achieved, especially at 4 gpm/ft$^2$ (162.80 l/min/m$^2$) by increasing the depth of coal or by using different size media with less intermixing.
5. More uniform and smaller media would improve effluent quality and affect the duration of the filter cycle; thereby forcing another economic analysis.
6. The filter effluent must have a detectable amount of chlorine (or other disinfectant) residual in order to prevent biological growth within the filter media.
7. Very close attention should be paid to the clean bed headloss and the rate of water backwash. Progressive increase of the clean bed headloss and concurrent reduction in the rate of water backwash is the sign of inadequate removal of suspended solids and/or bacterial growth in the filter bed.
8. If the addition of coagulants and coagulant aids to the filter influent is desirable, jar tests should simulate field conditions as closely as possible in order to prevent post-flocculation.

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# APPENDIX

## Table A-1. Experimental data at hydraulic loading rate = 3 gpm/ft$^2$ (122.10 l/min/m$^2$).

Filter media used: Coal - e.s. 1.0 mm and u.c. 1.7
Sand - e.s. 0.45 mm and u.c. 1.7

| Date | Sample Grab/Comp. | Filter Cycle Run (Hours) | Volume of Water Filtered (Gallons)[a] | % Lost in Back-wash | $BOD_5$ | | | SS | | | $NO_3$-N (mg/l) | | $NO_2$-N (mg/l) | | $NH_3$-N (mg/l) | | O-$PO_4$ (mg/l) | | Total P (mg/l) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Infl. (mg/l) | Effl. (mg/l) | % Re-moval | Infl. (mg/l) | Effl. (mg/l) | % Re-moval | Infl. | Effl. | Infl. | Effl. | Infl. | Effl. | Infl. | Effl. | Infl. | Effl. |
| 5-11-74 - 5-12-74 | 16 hr. comp. | 8.50 | 19,220 | 3.6 | 16.0 | | | 22.5 | 5.7 | 74 | 4.2 | 4.3 | 0.09 | 0.09 | 0.76 | 0.75 | 3.0 | 3.1 | 4.4 | 4.0 |
| 5-12-74 5-13-74 | 20 hr. comp. | 8.33 | 18,830 | 3.7 | 15.6 | 9.4 | 40 | | | | 5.3 | 5.2 | 0.08 | 0.07 | 0.77 | 0.72 | 2.4 | 2.3 | 3.2 | 3.0 |
| 5-13-74 - 5-14-74 | 18 hr. comp. | 9.00 | 20,350 | 3.4 | 10.8 | 9.1 | 16 | 22.8 | 3.4 | 85 | 3.5 | 3.6 | 0.06 | 0.07 | 0.59 | 0.74 | 3.0 | 3.1 | 4.5 | 4.2 |
| 5-18-74 - 5-19-74 | 24 hr. comp. | 6.50 | 14,700 | 4.7 | 17.6 | 13.2 | 25 | 35.4 | 15.1 | 57 | 4.5 | 4.3 | 0.10 | 0.11 | 0.50 | 0.25 | 2.5 | 2.4 | 3.6 | 3.2 |
| 5-19-74 - 5-20-74 | 24 hr. comp. | 6.50 | 14,700 | 4.7 | 13.2 | 10.4 | 21 | 44.7 | 18.2 | 59 | 4.5 | 4.8 | 0.07 | 0.08 | 0.29 | 0.23 | 2.1 | 2.2 | 3.0 | 2.8 |
| 5-20-74 - 5-21-74 | 24 hr. comp. | 6.00 | 13,570 | 5.1 | 20.0 | 11.0 | 45 | 41.9 | 19.0 | 55 | 3.7 | 3.8 | 0.07 | 0.07 | 0.18 | 0.22 | 2.0 | 2.0 | 3.1 | 2.8 |

[a]Gallons x 3.785 = l.

## Table A-2. Experimental data at hydraulic loading rate = 4 gpm/ft$^2$ (162.80 l/min/m$^2$).

Filter media used: Coal - e.s. 1.0 mm and u.c. 1.7
Sand - e.s. 0.45 mm and u.c. 1.7

| Date | Sample Grab/Comp. | Filter Cycle Run (Hours) | Volume of Water Filtered (Gallons)[a] | % Lost in Back-wash | $BOD_5$ | | | SS | | | $NO_3$-N (mg/l) | | $NO_2$-N (mg/l) | | $NH_3$-N (mg/l) | | O-$PO_4$ (mg/l) | | Total-P (mg/l) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Infl. (mg/l) | Effl. (mg/l) | % Re-moval | Infl. (mg/l) | Effl. (mg/l) | % Re-moval | Infl. | Effl. | Infl. | Effl. | Infl. | Effl. | Infl. | Effl. | Infl. | Effl. |
| 6-3-74 - 6-4-74 | 16 hr. comp. | | | | | | | 43.1 | 17.6 | 59 | 3.5 | 3.8 | 0.07 | 0.08 | 0.75 | 0.94 | 2.5 | 2.5 | 3.8 | 3.6 |
| 6-15-74 - 6-16-74 | 24 hr. comp. | 5.00 | 11,300 | 6.1 | | | | 47.9 | 28.1 | 41 | 4.3 | 4.2 | 0.04 | 0.06 | | | 2.3 | 2.4 | 3.0 | 2.9 |
| 6-16-74 - 6-17-74 | 24 hr. comp. | 6.25 | 14,130 | 4.9 | 9.3 | 7.5 | | | | | 4.0 | 4.3 | 0.04 | 0.06 | | | 2.2 | 2.2 | 2.7 | 2.6 |

[a]Gallons x 3.785 = l.

## Table A-3. Experimental data at hydraulic loading rate = 3 gpm/ft$^2$ (122.10 l/min/m$^2$).

Filter media used: Coal - e.s. 1.0 mm and u.c. 1.7
Sand - e.s. 0.45 mm and u.c. 1.7

| Date | Sample Grab/Comp. | Filter Cycle Run (Hours) | Volume of Water Filtered (Gallons)[b] | % Lost in Back-wash | $BOD_5$ | | | SS | | | $NO_3$-N (mg/l) | | $NO_2$-N (mg/l) | | $NH_3$-N (mg/l) | | O-$PO_4$ (mg/l) | | Total-P (mg/l) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Infl. (mg/l) | Effl. (mg/l) | % Re-moval | Infl. (mg/l) | Effl. (mg/l) | % Re-moval | Infl. | Effl. | Infl. | Effl. | Infl. | Effl. | Infl. | Effl. | Infl. | Effl. |
| 7-1-74 | 1 F.C.[a] comp. | 10.50 | 23,740 | 2.9 | | | | 20.7 | 6.9 | 67 | 4.0 | 3.8 | 0.16 | 0.17 | 0.77 | 0.72 | 2.4 | 2.3 | 3.2 | 2.9 |
| 7-2-74 | 1 F.C. comp. | 10.33 | 23,350 | 3.0 | | | | 21.7 | 5.8 | 73 | 4.4 | 4.1 | 0.19 | 0.19 | 1.13 | 1.06 | 1.9 | 1.9 | 2.7 | 2.5 |
| 7-7-74 | 1 F.C. comp. | 9.42 | 21,300 | 3.2 | | | | | | | 5.0 | 5.1 | 0.18 | 0.14 | 1.17 | 1.44 | 1.9 | 1.9 | 2.7 | 2.4 |
| 7-8-74 | 1 F.C. comp. | 8.58 | 19,400 | 3.6 | | | | 32.9 | 8.6 | 74 | 3.6 | 3.6 | 0.90 | 0.10 | | | 2.4 | 2.3 | 3.5 | 3.2 |
| 7-9-74 | 1 F.C. comp. | 8.58 | 19,400 | 3.6 | | | | 32.2 | 18.5 | 43 | 3.6 | 3.5 | 0.10 | 0.11 | 2.34 | 1.38 | 2.2 | 2.1 | 3.2 | 2.7 |
| 7-20-74 | 1 F.C. comp. | 9.67 | 21,860 | 3.2 | | | | 23.1 | 6.0 | 74 | 4.9 | 4.9 | 0.07 | 0.08 | 0.28 | 0.30 | 2.4 | 2.3 | 3.3 | 2.9 |
| 7-21-74 | 1 F.C. comp. | 12.33 | 27,880 | 2.5 | | | | 21.0 | 4.5 | 79 | 5.0 | 4.7 | 0.04 | 0.06 | 0.20 | 0.24 | 2.0 | 1.9 | 2.6 | 2.3 |
| 7-22-74 | 1 F.C. comp. | 11.83 | 26,750 | 2.6 | | | | 22.1 | 9.3 | 58 | 3.6 | 3.6 | 0.04 | 0.06 | 0.32 | 0.25 | 2.7 | 2.6 | 3.6 | 3.3 |
| 7-23-74 | 1 F.C. comp. | 13.25 | 29,960 | 2.3 | | | | 24.3 | 13.1 | 46 | 3.2 | 3.1 | 0.03 | 0.05 | 0.40 | 0.39 | 2.5 | 2.4 | 3.2 | 2.9 |

[a]F.C. = filter cycle.
[b]Gallons x 3.785 = l.

## Table A-4. Experimental data at hydraulic loading rate = 3 gpm/ft$^2$ (122.10 l/min/m$^2$).

Filter media used: Coal - e.s. 1.10 mm and u.c. 1.60
Sand - e.s. 0.45 mm and u.c. 1.50
All samples are 1 filter cycle composite

| Date | Filter Cycle Run (Hours) | Volume of Water Filtered (Gallons)[a] | %Lost in Back-wash | BOD$_5$ | | | | | | | Suspended Solids | | | Volatile Suspended Solids | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Influent | | | Effluent | | % Removal | | Infl. | Effl. | | Influent | | Effluent | |
| | | | | Total (mg/l) | Soluble (mg/l) | % Soluble | Total (mg/l) | Soluble (mg/l) | Total | Soluble | (mg/l) | (mg/l) | % Removal | (mg/l) | % of SS | (mg/l) | % of SS |
| 2-5-75 | 8.58 | 19,400 | 3.6 | 17.2 | 7.5 | 44 | 12.2 | 7.8 | 29 | 0 | 22.7 | 6.5 | 71 | 17.6 | 77 | 5.1 | 79 |
| 2-12-75 | 9.42 | 21,300 | 3.2 | 13.4 | 4.7 | 35 | 8.0 | 4.7 | 40 | 0 | 22.0 | 5.3 | 76 | 15.6 | 71 | 4.0 | 76 |
| 2-15-75 | 8.50 | 19,220 | 3.6 | | | | | | | | | | | | | | |
| 2-19-75 | 9.50 | 21,480 | 3.2 | 15.5 | 4.9 | 31 | 8.8 | 4.8 | 43 | 0 | 18.8 | 5.0 | 73 | 16.1 | 85 | 4.5 | 90 |
| 3-11-75 | 6.42 | 14,510 | 4.8 | 20.0 | 5.6 | 28 | 10.0 | 5.7 | 50 | 0 | 30.8 | 7.7 | 75 | 23.8 | 77 | 5.9 | 77 |
| 3-13-75 | 9.00 | 20,350 | 3.4 | 19.0 | 6.3 | 33 | 9.2 | 6.0 | 52 | 0 | 24.3 | 5.1 | 79 | 19.5 | 80 | 4.1 | 81 |
| 3-18-75 | 7.33 | 16,570 | 4.2 | 23.0 | 5.9 | 26 | 10.4 | 6.4 | 55 | 0 | 27.6 | 6.6 | 76 | | | | |
| 3-20-75 | 7.25 | 16,390 | 4.2 | 16.8 | | | 8.0 | | 52 | 0 | 28.0 | 6.6 | 76 | 22.3 | 80 | 5.2 | 79 |

[a]Gallons x 3.785 = 1.

## Table A-5. Experimental data at hydraulic loading rate = 4 gpm/ft$^2$ (162.80 l/min/m$^2$).

Filter media used: Coal - e.s. 1.10 mm and u.c. 1.60
Sand - e.s. 0.45 mm and u.c. 1.50
All samples are 1 filter cycle composite

| Date | Filter Cycle Run (hours) | Volume of Water Filtered (Gallons)[a] | % Lost in Backwash | BOD$_5$ | | | Suspended Solids | | | Volatile Suspended Solids | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | Influent | | Effluent | |
| | | | | Influent (mg/l) | Effluent (mg/l) | % Removal | Influent (mg/l) | Effluent (mg/l) | % Removal | (mg/l) | % of SS | (mg/l) | % of SS |
| 3-25-75 | 3.67 | 8,300 | 8.3 | 26.5 | 12.5 | 53 | 31.1 | 7.9 | 75 | 25.0 | 80 | 6.6 | 83 |
| 3-25-75 | 3.17 | 7,170 | 9.6 | 22.5 | 14.0 | 33 | 33.7 | 9.5 | 72 | 25.8 | 77 | 7.5 | 79 |
| 3-27-75 | 3.58 | 8,090 | 8.5 | 15.5 | 8.4 | 46 | 27.1 | 6.6 | 76 | 19.5 | 72 | 5.1 | 77 |
| 4- 1-75 | 2.75 | 6,220 | 11.1 | 15.2 | 8.5 | 44 | 27.7 | 5.9 | 79 | 20.5 | 74 | 4.6 | 79 |
| 4- 1-75 | 3.33 | 7,530 | 9.2 | 13.9 | 9.0 | 35 | 28.4 | 6.2 | 78 | 21.1 | 74 | 4.9 | 80 |
| 4- 3-75 | 3.50 | 7,910 | 8.7 | 21.5 | 10.4 | 52 | 27.0 | 6.8 | 75 | 20.0 | 74 | 5.1 | 75 |
| 4- 3-75 | 3.83 | 8,660 | 8.0 | 16.8 | 10.3 | 39 | 28.0 | 6.8 | 76 | 20.0 | 71 | 5.1 | 75 |
| 4- 8-75 | 3.67 | 8,300 | 8.3 | 25.0 | 12.8 | 49 | 28.7 | 8.6 | 70 | 22.1 | 77 | 6.8 | 79 |
| 4- 8-75 | 4.92 | 11,120 | 6.2 | 28.5 | 14.0 | 51 | 28.2 | 8.7 | 69 | 22.1 | 78 | 7.0 | 80 |
| 4-10-75 | 4.33 | 9,790 | 7.0 | 22.0 | 10.3 | 53 | 26.1 | 6.4 | 75 | 20.6 | 79 | 5.1 | 79 |
| 4-10-75 | 4.58 | 10,350 | 6.7 | 26.5 | 11.8 | 56 | 29.0 | 8.6 | 70 | 22.8 | 79 | 6.8 | 79 |

[a]Gallons x 3.785 = 1.

## Table A-6. Experimental data at hydraulic loading rate = 5 gpm/ft$^2$ (203.50 l/min/m$^2$).

Filter media used: Coal - e.s. 1.10 mm and u.c. 1.60
Sand - e.s. 0.45 mm and u.c. 1.50
All samples are 1 filter cycle composite

| Date | Filter Cycle Run (Hours) | Volume of Water Filtered (Gallons)[a] | % Lost in Backwash | BOD$_5$ | | | Suspended Solids | | | Volatile Suspended Solids | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | Influent | | Effluent | |
| | | | | Influent (mg/l) | Effluent (mg/l) | % Removal | Influent (mg/l) | Effluent (mg/l) | % Removal | (mg/l) | % of SS | (mg/l) | % of SS |
| 4-15-75 | 3.25 | 7,350 | 9.4 | 21.5 | 12.5 | 42 | 30.0 | 9.6 | 68 | 23.5 | 78 | 8.0 | 82 |
| 4-15-75 | 3.42 | 7,730 | 8.9 | 19.5 | 11.1 | 43 | 31.3 | 10.5 | 66 | 25.6 | 82 | 8.8 | 83 |
| 4-17-75 | 2.67 | 6,040 | 11.4 | 22.0 | 10.8 | 51 | 33.1 | 9.2 | 72 | 24.8 | 75 | 6.8 | 74 |
| 4-17-75 | 3.50 | 7,910 | 8.7 | 17.2 | 10.0 | 42 | 26.7 | 9.4 | 65 | 21.8 | 82 | 6.7 | 71 |
| 4-22-75 | 2.83 | 6,400 | 10.8 | 17.2 | 9.8 | 43 | 28.8 | 7.2 | 75 | 22.4 | 78 | 5.7 | 79 |
| 4-22-75 | 2.67 | 6,040 | 11.4 | 17.8 | 8.8 | 51 | 30.0 | 7.8 | 74 | 22.4 | 75 | 6.1 | 79 |
| 4-24-75 | 2.58 | 5,830 | 11.8 | 17.8 | 10.3 | 42 | 29.3 | 7.1 | 76 | 21.7 | 74 | 5.4 | 76 |
| 4-24-75 | 2.42 | 5,470 | 12.6 | 22.5 | 11.3 | 50 | 28.6 | 7.8 | 73 | 21.1 | 74 | 5.9 | 75 |
| 4-29-75 | 2.58 | 5,830 | 11.8 | 18.8 | 9.5 | 50 | 27.0 | 6.7 | 75 | 21.6 | 80 | 5.5 | 82 |
| 4-29-75 | 2.75 | 6,220 | 11.1 | 23.5 | 11.0 | 53 | 31.4 | 7.6 | 76 | 22.2 | 71 | 6.3 | 83 |
| 4-30-75 | 2.58 | 5,830 | 11.8 | 25.5 | 10.3 | 60 | 30.6 | 8.1 | 73 | 24.4 | 80 | 6.7 | 83 |
| 4-30-75 | 2.33 | 5,270 | 13.1 | 27.5 | 13.0 | 53 | 34.4 | 8.8 | 74 | 27.2 | 79 | 7.3 | 82 |

[a]Gallons x 3.785 = 1.

**Table A-7. Experimental data at hydraulic loading rate = 6 gpm/ft$^2$ (244.20 l/min/m$^2$).**

Filter media used: Coal: - e.s. 1.10 mm and u.c. 1.60
Sand - e.s. 0.45 mm

Filter media used: Coal - e.s. 1.10 mm and u.c. 1.60
Sand - e.s. 0.45 mm and u.c. 1.50

All samples are 1 filter cycle composite

| Date | Filter Cycle Run (Hours) | Volume of Water Filtered (Gallons)[a] | % Lost in Backwash | BOD$_5$ Influent (mg/l) | BOD$_5$ Effluent (mg/l) | BOD$_5$ % Removal | Suspended Solids Influent (mg/l) | Suspended Solids Effluent (mg/l) | Suspended Solids % Removal | Volatile Suspended Solids Influent (mg/l) | Volatile Suspended Solids Influent % of SS | Volatile Suspended Solids Effluent (mg/l) | Volatile Suspended Solids Effluent % of SS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5- 9-75 | 1.00 | 2,260 | 30.5 | 21.0 | 11.3 | 46 | 44.5 | 11.7 | 74 | 34.2 | 77 | 9.1 | 78 |
| 5- 9-75 | 0.92 | 2,080 | 33.1 | 22.0 | 11.7 | 47 | 42.4 | 11.2 | 74 | 32.9 | 78 | 8.9 | 80 |
| 5-13-75 | 0.92 | 2,080 | 33.1 | 28.0 | 16.5 | 41 | 37.0 | 7.7 | 79 | 30.9 | 84 | 6.6 | 86 |
| 5-13-75 | 0.92 | 2,080 | 33.1 | 26.5 | 16.8 | 36 | 37.8 | 9.5 | 75 | 31.1 | 82 | 8.2 | 86 |
| 5-15-75 | 1.00 | 2,260 | 30.5 | | | | 38.8 | 7.4 | 81 | 31.3 | 81 | 6.2 | 84 |
| 5-15-75 | 0.92 | 2,080 | 33.1 | | | | 38.8 | 7.4 | 81 | 31.5 | 81 | 6.0 | 81 |
| 5-27-75 | 2.25 | 5,090 | 13.5 | 18.0 | 11.3 | 38 | 23.6 | 6.1 | 74 | 19.6 | 83 | 5.2 | 86 |
| 5-27-75 | 1.50 | 3,390 | 20.3 | 22.5 | 13.8 | 39 | 28.0 | 7.2 | 74 | 23.0 | 82 | 5.9 | 82 |
| 5-29-75 | 2.00 | 4,520 | 15.2 | 15.2 | 9.5 | 37 | 26.8 | 7.3 | 73 | 22.0 | 82 | 6.0 | 82 |
| 5-29-75 | 1.83 | 4,140 | 16.7 | 16.5 | 10.5 | 36 | 28.6 | 7.9 | 72 | 23.3 | 82 | 6.6 | 84 |
| 6- 3-75 | 1.67 | 3,780 | 18.3 | 16.8 | 8.0 | 52 | 27.3 | 6.2 | 77 | 22.3 | 82 | 5.2 | 84 |
| 6- 3-75 | 2.00 | 4,520 | 15.2 | 17.5 | 10.0 | 43 | 24.1 | 5.7 | 76 | 20.0 | 83 | 4.8 | 84 |
| 6- 5-75 | 2.33 | 5,270 | 13.1 | 17.5 | 9.4 | 46 | 24.7 | 7.0 | 72 | 20.8 | 84 | 5.9 | 85 |
| 6- 5-75 | 1.83 | 4,140 | 16.7 | 26.5 | 10.6 | 60 | 28.1 | 7.7 | 73 | 23.8 | 85 | 6.5 | 85 |

[a]Gallons x 3.785 = 1.

Estimate 1:

| | |
|---|---|
| Design flow | 60 mgd |
| Design filtration rate | 3 gpm/ft$^2$ |
| Recycled backwash water | 5% = 3 mgd |
| Interest rate | 5½% |
| Time period | 20 years |
| Annuity factor | 0.08368 |
| Cost funded by EPA | 75% of the capital cost |

Capital cost:

| Item | Quantity | Unit Cost | Total Cost |
|---|---|---|---|
| SVG filters (40' diameter) | 12 | $175,000 | $2,100,000 |
| Screw pumps | 4 | $ 76,500 | $ 306,000 |
| Building (300' x 130' x 30' height) | 1 | $15/ft$^2$ | $ 585,000 |
| Laboratory | | | $ 20,000 |
| Installation of filters and pumps 55%, engineer fees, etc. 7½% | | | $1,549,200 |
| | Total | | $4,560,200 |

Cost to the plant per year:

With federal assistance: $4,560,200 x 0.25 x 0.08368 = $95,400

Without federal assistance: $4,560,200 x 0.08368 = $381,600

Annual operation and maintenance cost:

Power cost:

Screw pumps: $4 \text{ units} \times 125 \frac{\text{HP}}{\text{unit}} \times 0.746 \frac{\text{kw}}{\text{HP}} \times 0.01 \frac{\$}{\text{kwh}} \times 24 \frac{\text{hrs}}{\text{day}} \times 365 \frac{\text{days}}{\text{yr}}$ — $ 32,700

SVG: $12 \text{ units} \times 25 \frac{\text{HP}}{\text{unit}} \times 0.746 \frac{\text{kw}}{\text{HP}} \times 0.01 \frac{\$}{\text{kwh}} \times 1 \frac{\text{hr}}{\text{day}} \times 365 \frac{\text{days}}{\text{yr}}$ — $ 800

Material and supply cost: — $ 42,000

Operation cost: $2{,}000 \frac{\text{man hrs}}{\text{yr}} \times 7 \frac{\$}{\text{man-hr}}$ — $ 14,000

Maintenance cost: $2{,}000 \frac{\text{man-hrs}}{\text{yr}} \times 10 \frac{\$}{\text{man-hr}}$ — $ 20,000

Cost of replacement of media: — $ 17,600

Cost of treating additional suspended solids: $\frac{\$30}{\text{dry ton}} \times 1{,}825 \frac{\text{dry ton}}{\text{yr}}$ — $ 54,800

Total — $ 181,900

Estimate 2:

| | |
|---|---|
| Design flow | 60 mgd |
| Design filtration rate | 4 gpm/ft$^2$ |
| Recycled backwash water | 7.5% = 4.5 mgd |
| Interest rate | 5½% |
| Time period | 20 years |
| Annuity factor | 0.08368 |
| Cost funded by EPA | 75% of the capital cost |

Capital cost:

| Item | Quantity | Unit Cost | Total Cost |
|---|---|---|---|
| SVG filters (40' diameter) | 9 | \$175,000 | \$1,575,000 |
| Screw pumps | 4 | \$ 76,500 | \$ 306,000 |
| Building (250' x 130' x 30' height) | 1 | \$15/ft$^2$ | \$ 487,500 |
| Laboratory | | | \$ 20,000 |
| Backwash water storage lagoons | 2 (2.25 million gallons each) | | \$ 73,200 |
| Backwash water recycle pumps | Total 9 mgd capacity | | \$ 182,900 |
| Installation of filters and pumps 55%, engineer fees, etc. 7½% | | | \$1,333,500 |
| | Total | | \$3,978,100 |

Cost to the plant per year:
With federal assistance: \$3,978,100 x 0.25 x 0.08368 = \$83,300
Without federal assistance: \$3,978,100 x 0.08368 = \$332,900
Annual operation and maintenance cost:
Power cost:

| | |
|---|---|
| Screw pumps: 4 units x 125 $\frac{\text{HP}}{\text{unit}}$ x 0.746 $\frac{\text{kw}}{\text{HP}}$ x 0.01 $\frac{\$}{\text{kwh}}$ x 24 $\frac{\text{hrs}}{\text{day}}$ x 365 $\frac{\text{days}}{\text{yr}}$ | \$ 32,700 |
| SVG: 9 units x 25 $\frac{\text{HP}}{\text{unit}}$ x 0.746 $\frac{\text{kw}}{\text{HP}}$ x 0.01 $\frac{\$}{\text{kwh}}$ x 1.5 $\frac{\text{hrs}}{\text{day}}$ x 365 $\frac{\text{days}}{\text{yr}}$ | \$ 900 |
| Backwash water recycle pumps: Total 9 mgd capacity | \$ 2,700 |
| Material and supply cost: | \$ 33,900 |
| Operation cost: 3,000 $\frac{\text{man-hrs}}{\text{yr}}$ x 7 $\frac{\$}{\text{man-hr}}$ | \$ 21,000 |
| Maintenance cost: 2,800 $\frac{\text{man-hrs}}{\text{yr}}$ x 10 $\frac{\$}{\text{man-hr}}$ | \$ 28,000 |
| Cost of replacement of media: | \$ 13,200 |
| Cost of treating additional suspended solids: $\frac{\$30}{\text{dry ton}}$ x 1,825 $\frac{\text{dry ton}}{\text{yr}}$ | \$ 54,800 |
| Total | \$ 187,200 |

Estimate 3:

| | |
|---|---|
| Design flow | 60 mgd |
| Design filtration rate | 5 gpm/ft$^2$ |
| Recycled backwash water | 10.5% = 6.3 mgd |
| Interest rate | 5½% |
| Time period | 20 years |
| Annuity factor | 0.08368 |
| Cost funded by EPA | 75% of the capital cost |

Capital cost:

| Item | Quantity | Unit Cost | Total Cost |
|---|---|---|---|
| SVG filters (40' diameter) | 8 | \$175,000 | \$1,400,000 |
| Screw pumps | 4 | \$ 76,500 | \$ 306,000 |
| Building (200' x 130' x 30' height) | 1 | \$15/ft$^2$ | \$ 390,000 |

| | | | |
|---|---|---|---|
| Laboratory | | | \$ 20,000 |
| Backwash water storage lagoons | 3 (2.1 million gallons each) | | \$ 105,000 |
| Backwash water recycle pumps | Total 12.6 mgd capacity | | \$ 222,600 |
| Installation of filters and pumps 55%, engineer fees, etc., 7½% | | | \$1,244,000 |
| | Total | | \$3,687,600 |

Cost to the plant per year:
With federal assistance: \$3,687,600 x 0.25 x 0.08368 = \$77,200
Without federal assistance: \$3,687,600 x 0.08368 = \$308,600
Annual operation and maintenance cost:
Power cost:

Screw pumps: $4 \text{ units} \times 125 \frac{\text{HP}}{\text{unit}} \times 0.746 \frac{\text{kw}}{\text{HP}} \times 0.01 \frac{\$}{\text{kwh}} \times 24 \frac{\text{hrs}}{\text{day}} \times 365 \frac{\text{days}}{\text{yr}}$ — \$ 32,700

SVG: $8 \text{ units} \times 25 \frac{\text{HP}}{\text{unit}} \times 0.746 \frac{\text{kw}}{\text{HP}} \times 0.01 \frac{\$}{\text{kwh}} \times 2 \frac{\text{hrs}}{\text{day}} \times 365 \frac{\text{days}}{\text{yr}}$ — \$ 1,100

Backwash water recycle pumps: Total 12.6 mgd capacity — \$ 3,400

Material and supply cost: — \$ 31,100

Operation cost: $3{,}100 \frac{\text{man-hrs}}{\text{yr}} \times 7 \frac{\$}{\text{man-hr}}$ — \$ 21,700

Maintenance cost: $2{,}900 \frac{\text{man-hrs}}{\text{yr}} \times 10 \frac{\$}{\text{man-hr}}$ — \$ 29,000

Cost of replacement of media: — \$ 11,800

Cost of treating additional suspended solids: $\frac{\$30}{\text{dry ton}} \times 1{,}825 \frac{\text{dry ton}}{\text{yr}}$ — \$ 54,800

Total — \$ 185,600

Estimate 4:

| | |
|---|---|
| Design flow | 60 mgd |
| Design filtration rate | 6 gpm/ft$^2$ |
| Recycled backwash water | 20% = 12 mgd |
| Interest rate | 5½% |
| Time period | 20 years |
| Annuity factor | 0.08368 |
| Cost funded by EPA | 75% of the capital cost |

Capital cost:

| Item | Quantity | Unit Cost | Total Cost |
|---|---|---|---|
| SVG filters (40' diameter) | 7 | \$175,000 | \$1,225,000 |
| Screw pumps | 4 | \$ 76,500 | \$ 306,000 |
| Building (200' x 130' x 30' height) | 1 | \$15/ft$^2$ | \$ 390,000 |
| Laboratory | | | \$ 20,000 |
| Backwash water storage lagoons | 6 (2 million gallons each) | | \$ 205,200 |
| Backwash water recycle pumps | Total 24 mgd capacity | | \$ 365,700 |
| Installation of filters and pumps 55%, engineer fees, etc., 7½% | | | \$1,231,600 |
| | Total | | \$3,743,500 |

Cost to the plant per year:
With federal assistance: \$3,743,500 x 0.25 x 0.08368 = \$78,400
Without federal assistance: \$3,743,500 x 0.08368 = \$313,300
Annual operation and maintenance cost:
Power cost:

Screw pumps: $4 \text{ units} \times 125 \frac{\text{HP}}{\text{units}} \times 0.746 \frac{\text{kw}}{\text{HP}} \times 0.01 \frac{\$}{\text{kwh}} \times 24 \frac{\text{hrs}}{\text{day}} \times 365 \frac{\text{days}}{\text{yr}}$ — \$ 32,700

| | |
|---|---|
| SVG: 7 units x $25 \frac{HP}{unit}$ x $0.746 \frac{kw}{HP}$ x $0.01 \frac{\$}{kwh}$ x $4 \frac{hrs}{day}$ x $365 \frac{days}{yr}$ | \$ 1,900 |
| Backwash water recycle pumps: Total 24 mgd capacity | \$ 5,800 |
| Material and supply cost: | \$ 30,600 |
| Operation cost: $3{,}500 \frac{\text{man-hrs}}{yr}$ x $7 \frac{\$}{\text{man-hr}}$ | \$ 24,500 |
| Maintenance cost: $3{,}300 \frac{\text{man-hrs}}{yr}$ x $10 \frac{\$}{\text{man-hr}}$ | \$ 33,000 |
| Cost of replacement of media: | \$ 10,300 |
| Cost of treating additional suspended solids: $\frac{\$30}{\text{dry ton}}$ x $1{,}825 \frac{\text{dry ton}}{yr}$ | \$ 54,800 |
| Total | \$ 193,600 |