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SLOW SAND FILTRATION

TECHNOLOGY

FOR

KOSRAE STATE

THE FEDERATED STATES OF MICRONESIA:

A PILOT PROJECT

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The activities on which this report is based were financed in part by the Department of the Interior, U.S. Geological Survey, through the University of Guam Water and Environmental Research Institute of the Western Pacific.

ACKNOWLEDGMENTS

There were many wonderful individuals who helped us during the course of this project. Without them this project could not have taken place. First, we would like to acknowledge Mr. Likiak P. Wesley, Administrator, Kosrae's Division of Planning and Statistics, for his persistent encouragement and consistent funding support for this project. We are grateful to Mr. Bruce Howell, Administrator Kosrae Road Division, Department of Transportation and Utility, for his technical support and daily supervision of construction. We thank Mr. Soloman Talley and Kosrae Department of Transportation and Utility employees for construction and operation of the facility. Special thanks to Mr. Katchuo William, Chief Sanitarian, Kosrae Department of Health Services, who graciously undertook the task of daily data collection and analysis. We extend our special thanks to our Laboratory Manager, Mr. Rick Wood for his valuable advice and expertise in water quality, and the previous Director of WERI, Dr. Galt Siegrist, for his support and interest in this project. We would also like to thank Dr. Gary Denton, for his valuable advice and interest.

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ABSTRACT

This report presents the results of a collaborative pilot study between University of Guam Water and Environmental Research Study and the Kosrae State. The objective of this study was to determine the feasibility of slow sand filtration technology in treating surface water resources on Kosrae, the easternmost state of the Federated States of Micronesia. The pilot facility constructed on Kosrae consisted of four filters; two filters with locally manufactured filtration media and two packed with imported sand specially formulated for slow sand filtration. The goals of the study were to determine: 1) if locally crushed basalt media exhibited the same bacteria and turbidity removal efficiency as imported media, 2) the run lengths associated with both types of media when an hydraulic loading rate (HLR) of 0.2 m/hr was used and, 3) to determine design criteria for the construction of full-scale facility. The study revealed that in mature filters, 1) fecal Coliform removal exceeded 2-log cycle (over 99%) and, 2) turbidity removal appeared similar. Significant differences in tail water pH (8.3 for local media; 7.6 for imported media), and filter run lengths were observed.

INTRODUCTION

The lack of clean drinking water is a significant problem for residents of the high, volcanic islands of the Federated States of Micronesia (FSM) (US EPA, 1986). Residents of Kosrae State, the easternmost member of the FSM, are highly reliant on surface water resources. These waters, however, are not treated, due to a lack of funding for conventional treatment methods (Dekel, 1981). Consequently, untreated water is piped directly into people's homes bearing with it significant sediment and contamination.

1.1 Problem of Surface Water Quality on Kosrae

Abundant rainfall occurs throughout the islands of the FSM, which are situated in the tropical latitudes of the Western Pacific (Figure 1).

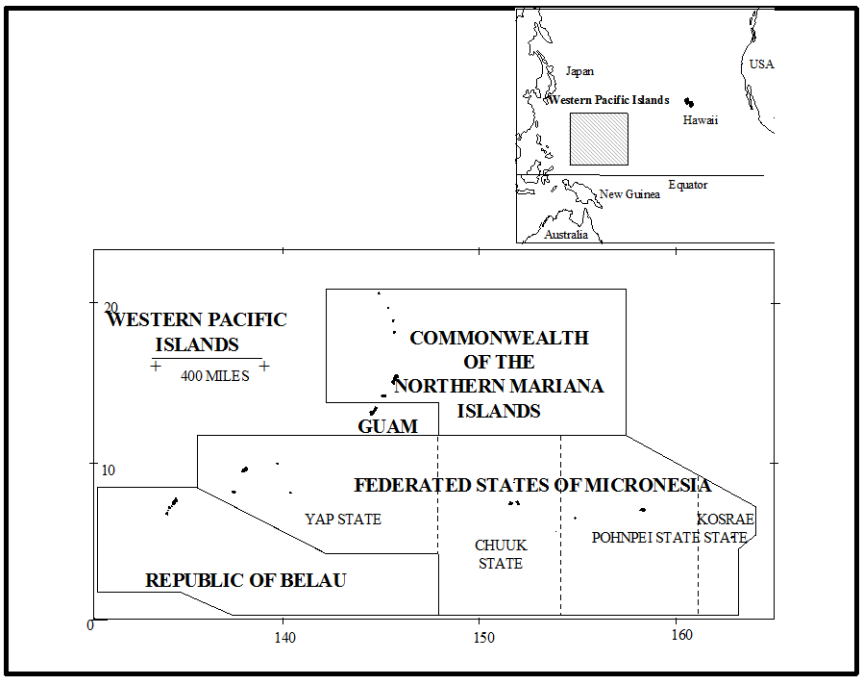


Figure 1. Map of the Federated States of Micronesia (FSM).

Approximately 200 inches of rainfall is received annually along the coast of Kosrae, and an estimated 225 inches falls in the mountainous interior (USGS, 1984). The rainforests of the interior significantly increase rainfall interception, while the absorptive capacity of the terrain allows for capture of precipitation and a gradual release of water, creating perennial flow streams (U.S. Army Corps, 1985). These streams presently serve as a major source of water on the island (US EPA, 1986). Water is piped to all communities from water intakes connected to dams on the larger streams (Dekel, 1981) (Figure 2).

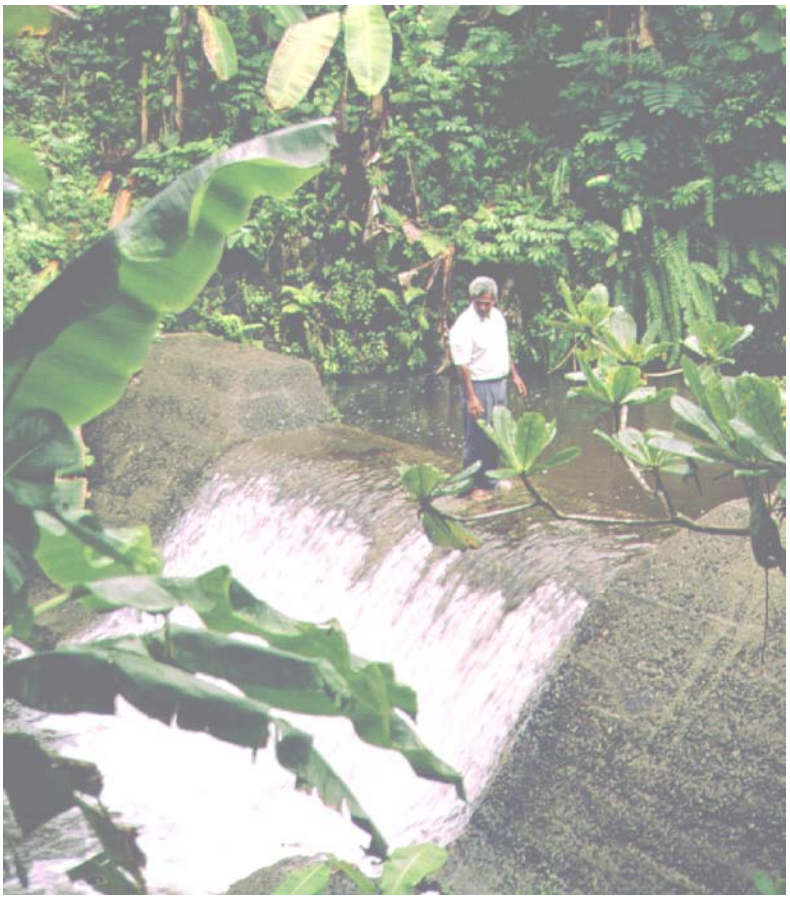


Figure 2. A Kosraen dam on the Tafuyat River.

Dekel reported in 1981 that funding limitations imposed by Kosrae's subsistence level economy prevented development of highly technical water treatment facilities. Presently, similar economic barriers continue to hinder development of water treatment facilities throughout the FSM.

Prior to establishment of the Compacts of Free Association with the United States in 1986, the Federated States of Micronesia was part of the Trust Territory of the Pacific Islands, established by the United Nations in 1947 (US EPA, 1986). The United States was assigned to administer the Trusteeship by providing for political self-determination and improvement in the overall quality of life for islanders (US EPA, 1986). Under this agreement, some funding for water treatment systems was provided, enabling development of treatment units including slow and rapid sand filtration (US EPA, 1986). A rapid sand facility was built on Kosrae, however, lack of training, plant maintenance costs, problems stemming from the treatment of highly turbid stream water, coupled with high water demand, resulted in closure of the facility.

Until recently, local support for water treatment has been minimal. Resistance to using Compact funds for water treatment facilities is strong, as residents fear that costs associated with operating and maintaining facilities built with such funds will ultimately be their responsibility and they do not want to pay for water, which has always been free (personal communication with business owners and Kosrae government personnel). Consequently, those residing on Kosrae presently consume stream water that is untreated.

Additional sources of drinking water on Kosrae are groundwater (Okat area primarily) and rainwater. Bacterial testing of catchment sources, however, reveals contamination most likely attributed to fecal matter left by rats and other small animals (personal communication with Chief Sanitation Officer for Kosrae, Katchuo William in June 2000; US EPA, 1986). Given

the condition of surface water resources, residents of Kosrae suffer from various water-borne illnesses.

Annual cases of amoebiasis (ameobic dysentery), gastroenteritis and diarrheal diseases, hepatitis, giardiasis and leptospirosis have been reported on Kosrae (US EPA, 1986). In 1983, three cases of giardiasis were reported, but it is believed that due to symptoms similar to other diarrheal diseases, the incidence rate may be considerably higher than reported (US EPA, 1986). In 1986, the third leading cause of death on Kosrae was attributed to diarrheal and intestinal diseases (US EPA, 1986).

In 1990, David Sasaki, the State of Hawaii's Veterinary Medical Officer, published a travel report following his November 8-15 1990 visit to Kosrae. He estimated an annual incidence rate of leptospirosis on Kosrae of 400 cases per 100,000 individuals. According to Sasaki, this estimate for incidences of leptospirosis was 61 times higher than Hawaii's highest annual incidence rate estimate of 6.5 cases per 100,000 individuals and 8000 times greater than the United States estimated incidence rate of 0.05 cases per 100,000 nationwide (Sasaki, 1990). Between January 1990 and October 1990, eight patients were airlifted to Hawaii at a cost of $25,000 per person (Sasaki, 1990). In his report, Sasaki recommended that Kosrae chlorinate the stream fed fresh water systems, as most cases of leptospirosis involved exposure to these waters.

In order to reduce the incidence rate of leptospirosis and other water borne illnesses, water treatment is necessary. Recommended treatment includes both filtration and chlorination (US EPA, 1986). Furthermore, the treatment technology must be economical to build, and simple to operate and maintain given the adverse economic and environmental conditions of this remote island. For these reasons, slow sand filtration has been selected as a potential water treatment technology.

1.2 The Selection of Slow Sand Filtration Technology for Kosrae State

Slow sand filtration technology has been employed in developing countries using design requirements that emphasize simplicity of construction, operation and a reliance on locally available materials (Ellis, 1985; Shenkut, 1996). For these reasons, and the fact that local residents can be trained to operate and maintain slow sand filters, the World Health Organization (WHO) has been a strong advocate of slow sand filtration technology in developing countries. The design and operation of such systems avoids the difficulty and expense of hiring and retaining technically skilled personnel in remote, developing regions; a significant aspect to consider in the selection of water treatment technology for such locations (Ellis, 1985).

In 1981, the potential benefit of slow sand filtration technology for Kosrae State was identified by E. Dekel, a sanitary engineer working for WHO on a water resources review of the island. Dekel reported

> Although piped water supply is available to most areas, water quality is poor and unsafe for drinking. To make water supplied meet acceptable drinking water standards, water treatment is required. The most suitable treatment under local conditions is the slow sand filter process. This process is very efficient in terms of improving bacteriological quality of water, is very simple to operate and maintain and does not require any chemicals for water treatment (beside chlorination which may be added for protection).
>
> It is recommended that the consideration would be given for the construction of slow sand filters for treatment of raw water at the different sources of supply.
>
> Technical advice on the design of these filters could be provided by WHO at government's request.

Given the potential benefit that slow sand filtration technology holds for its residents, the State of Kosrae requested regional assistance in assessing the feasibility of applying such technology in treating local surface water resources.

1.3 Purpose of the Study

To accomplish this assessment, a collaborative pilot project between the University of Guam Water and Environmental Research Institute (WERI) and the Engineering Staff of the Kosrae State Department of Transportation and Utility was undertaken. The scope of the pilot study included: 1) planning and design of the pilot plant, 2) construction of the plant, 3) plant start-up and operation, 4) continuous monitoring and testing of the pilot plant and, 5) a performance evaluation of the plant.

The two primary goals of this study involved 1) evaluating the relative effectiveness of locally manufactured filtration media in removing bacteria and reducing turbidity and, 2) determining appropriate design criteria for a full-scale slow sand filtration facility, detailing optimum filter loading rate, time to filter bed maturation, and the length of filter run length prior to scraping.

To achieve these goals, the filtration effectiveness of locally manufactured basalt sand media was compared to that of imported sand media specifically formulated to meet slow sand filtration applications. The overall performance of both types of media, together with the effectiveness of the plant design, was then evaluated.

REVIEW OF LITERATURE

2.1 History of Slow Sand Filtration

Originating in Europe, slow sand filtration is classified as the first, modern water-treatment technology (Ellis, 1985). This filtration process removes particles and microorganisms by the slow percolation of water through a porous sand media. Unlike other water treatment technology (i.e. rapid sand filtration), conventional slow sand technology does not involve chemical or physical pre-treatment applications (Collins, Eighmy, Fenstermacher and Spanos, 1992).

The origin of slow sand filtration technology dates back to 1790, in Lancashire England (Weber-Shirk and Dick, 1997a). It was there that rudimentary sand filters were first constructed to purify water used in the bleaching process. In 1804, John Gibb of Paisley Scotland constructed a sand filter used primarily for his bleachery, however, he also sold excess filtered water to the public (Ellis, 1985). Gibb's design was improved upon by Robert Thom in 1827 (Ellis, 1985). Two years later, this modified design was used by James Simpson in his plans for a one-acre sand filter for the Chelsea Water Company of London (Ellis, 1985). The health benefits attributed to London's first sand filter led to the construction of additional filters. By 1852, the city of London required filtration of all drinking water sold to the public. To ensure fulfillment of this requirement, the Thames Conservancy Board was established to regulate drinking water quality (Hendricks, 1991).

Adoption of slow sand filter technology spread throughout Europe in the mid- to late 1800's and by 1872, the technology had reached the United States. Poughkeepsie, New York was the first American town to build a slow sand filter (Hendricks, 1991). Additional

installations followed, and by 1899, twenty such filters were in use in the United States (Hendricks, 1991).

America's preference for this technology, however, was not forthcoming. By 1940, the United States had approximately 100 slow sand filters with an aggregate capacity of 52.6 million gallons per day (mgd), in contrast to roughly 2, 275 rapid sand filters with a production capacity of 237 mgd (Hendricks, 1991). Problems associated with highly turbid waters made conventional slow sand treatment impractical for communities plagued with such source water. Conventional slow sand filters clogged under such conditions, and the technology of choice became rapid sand filtration, due to its ability to produce large quantities of acceptable finished water from highly turbid source water (Ellis, 1985). An additional factor influencing the move to rapid sand filtration was public support for the newest technology available, regardless of community size (Logsdon, 1991).

Recently, however, slow sand filtration technology has received a resurgence of interest in the United States (Logsdon, 1991). Increased concerns regarding the persistence of *Giardia* cysts in many municipal water systems has led to a greater interest in slow sand technology (Lange, Bellamy, Hendricks and Logsdon, 1986; Fogel, Isaac-Renton, Guasparini, Moorehead and Ongerth, 1993). With the 1989 passage of the Surface Water Treatment Rule (SWTR) in the United States, many previously unfiltered surface water sources now require filtration (Logsdon, 1991; Brink and Parks 1996). The United States Environmental Protection Agency (EPA) has set a turbidity standard $\leq$ 1 nephelometric turbidity unit (NTU) 95 percent of the time, never to exceed 5 NTU's. Furthermore, the removal or inactivation of *Giardia* cysts is to be $\geq$ 3-logarithmic (log) and virus removals are to be $\geq$ 4-log removal. Removals of microorganisms in slow sand filters have proven to be 2 – log to 4 – log in effluent of slow sand filters (Hendricks

and Bellamy, 1991). The effectiveness of slow sand filtration in removing *Giardia* cysts is well documented (Fogel et al., 1993; Bellamy, Hendricks and Logsdon, 1985; Ellis, 1985). Research in the United States and Great Britain has shown the effectiveness of slow sand filtration in removing viruses and bacteria (Wheeler and Lloyd, 1988; Poynter and Slade 1977 as cited by Hendricks and Bellamy, 1991).

The effectiveness, affordability and ease of operation available with slow sand filtration systems is appealing to small communities (those under 10,000 people) that lack significant capital for constructing, operating and maintaining rapid sand filtration facilities (Riesenberg, Walters, Steele, and Ryder, 1995; Li, Ma and Du, 1996). As of 1984, a survey by Simms and Slezak identified 71 slow sand filtration facilities in operation in the United States. Brink and Parks (1996) stated that a preliminary report compiled for the American Slow Sand Association indicated that 225 such facilities were in use in the United States. It is anticipated that additional facilities will be built by small communities needing affordable, effective water treatment technology to comply with the surface water requirements established in 1989 (Logsdon, 1991; Brink and Parks, 1996).

2.2 General Description of Slow Sand Filtration Technology

2.2.1 *Mechanisms of Filtration*

Particulate (microbial, viral and sediment) removal in slow sand filtration is considered a passive process, differing from rapid sand filtration in that chemical pre-treatment of inflow is generally not performed and backflushing (pressurized flow reversal) is not used for cleaning the filter media (Haarhoff and Cleasby, 1991). In rapid sand systems, filtration requires flocculation to coagulate particles contained in the inflow, coupled with backflushing every 1-2 days to dislodge coagulated particles trapped in the media (Haarrhoff and Cleasby, 1991). In contrast,

slow sand water purification depends upon two passive removal mechanisms: 1) biological and 2) physical-chemical; neither of which is well understood (Weber-Shirk and Dick, 1997a; Weber-Shirk, 1997b). Removals attributed to biological activity within the filter media are absent in rapid sand filters, due to the aforementioned processes that prevent establishment of biological communities within the filtration media (Haarhoff and Cleasby, 1991).

In slow sand filters, biological processes are considered to dominate the uppermost region of the filter bed (Haarhoff and Cleasby, 1991; Ellis 1995). A layer termed the *schmutzdecke*, literally translated as "dirty skin" (as cited in Hendricks, 1991), forms on the surface of the sand bed and is believed to contribute to the removal of water impurities. Considerable disagreement exists in the literature, however, as to how and to what extent this is accomplished (Weber-Shirk and Dick, 1997a).

It has been hypothesized that within the *schmutzdecke*, algae, plankton, diatoms, and bacteria break down introduced organic matter through biological activity (Weber-Shirk and Dick, 1997a).

Collins et al. (1992) showed that bacterial concentrations in the *schmutzdecke* were a function of elapsed time and potential for cell growth, rather than the filtration of free-living bacteria from source water. This suggests that biological communities grow and develop within this layer.

In addition to the *schmutzdecke*, the sand grains of the filter bed provide additional biological and physical mechanisms that contribute to removal efficiency (McMeen and Benjamin, 1997; Ellis 1985). A biofilm develops around the sand grains and it has been hypothesized that such films create sticky surfaces, causing the attachment of organic and inorganic particles (Weber-Shirk, 1997b). This surface is thought to be biologically active (consisting of bacteria, protozoa

and bacteriophages) and a site for the decomposition of organic matter (Weber-Shirk, 1997b). Hendricks (1991) presents a thorough review of the potential pathways that particles (organic and inorganic) follow through the filter media and the theoretical collisions such particles experience within the media.

Physical mechanisms such as straining and adsorption are also considered to contribute to the removal effectiveness of slow sand filters (Weber-Shirk and Dick, 1997a). Adsorption of suspended material is influenced by zeta potentials (Hendricks, 1991). According to O'Brien (1996), a zeta potential may be described as follows

> A charged particle suspended in an electrolytic solution attracts ions of the opposite charge to those at its surface, where they form the Stern layer. To maintain the electrical balance of the suspending fluid, ions of opposite charge are attracted to the Stern layer. The potential at the surface of that part of this diffuse double-layer of ions that can move with the particle when subjected to a voltage gradient is the zeta potential. This potential is very dependent upon the ionic concentration, pH, viscosity, and dielectric constant of the solution being analyzed.

The biological and physical factors associated with slow sand filtration make factors affecting filter biogeochemistry (pH, dissolved oxygen, and temperature) useful variables to measure in pilot studies designed to determine: 1) the suitability of a particular water source considered for filtration and, 2) the performance of a particular filtration media for slow sand filtration (Ellis, 1985).

Temperature measurements are used in determining physical characteristics of the media such as the intrinsic hydraulic conductivity, k' which is a function of the viscosity of the water moving through a filter and the filter media itself (sand size, distribution and the aggregation of the sand grains) (Hendricks, 1991). Temperature adjustment for viscosity allows for determination of the porous characteristics of the media (Hendricks, 1991), which is useful for determining: 1) if a particular sand meets the porosity specifications for slow sand filtration

applications, and 2) what amount of headloss can be expected due to this porosity when the filter bed is clean (Hendricks, 1991).

2.2.2 Design Elements

A slow sand filter consists of essentially three components: 1) sand, 2) gravel and 3) an underdrainage (Ellis, 1985). A container (circular, square or rectangular) is used to hold a column of water (the supernatant or headwater) on a bed of sand (filtration media) supported by a gravel medium (Pyper and Logsdon, 1988). The column of water provides a pressure head for driving the flow of raw water through the filter media. The gravel supports the sand bed in addition to the underdrains, a network of perforated pipes that collect filtered water and channel it out of the filter container (Ellis, 1985), which it covers. The gravel is arranged with the finest grade directly beneath the sand bed and successively coarser grades leading to and surrounding the underdrain pipes (Pyper and Logsdon, 1991). Haarhoff and Cleasby (1991) cite recommendations made by Visscher regarding design criteria for slow sand filters. These are presented in Table 1 with a modification on bed depth (*) as shown in Hendricks (1991).

Table 1

Design Criteria and Recommendations for Slow Sand Filters

DESIGN PARAMETER	RECOMMENDATION
Depth of filter bed:	
Initial Bed Depth	0.8 m-0.9 m (2.63 ft-2.95 ft) *modified 1.0-1.3 m (3.28 ft-4.27 ft)
Minimum Bed Depth (requires re-sanding at this depth)	0.5 m-0.6 m (1.64 ft-1.97 ft)
Maximum Bed Area	200 m^2 (2153 ft^2) minimum of 2 beds
Sand size:	
Effective size (d_{10})	0.15 mm-0.30 mm (0.006 in-0.012 in)
Uniformity Coefficient (UC)	< 5 (preferably <3)
Depth of gravel support	0.3 m-0.5 m (0.984 ft-1.64 ft
Depth of supernatant (headwater)	1 m (3.28 ft)
Filtration Rate	0.1 –0.2 m/hr

2.2.3 Regulation of Flow

In addition to the design parameters identified previously, flow-metering devices for regulating either inflow or outflow are basic in designing slow sand filters (Di Bernardo and Carrasco, 1996; Hendricks, 1991). Orifice plate inflow meters are recommended due to their accuracy and ease of operation (Hendricks, 1991). Flow rate and sand bed surface area are used in determining the hydraulic loading rate (HLR) of a slow sand filter. The hydraulic loading rate is calculated by dividing the flow rate by the plan area of the sand bed. These factors determine the production rate for the filter. Knowing the production rate needed to meet community

demand for water and the optimum HLR for a given filtration media and plant design, allows for calculation of proper filter bed size.

Outflow weirs are used for controlling tailwater (filter effluent) flow (Hendricks, 1991). Such weirs maintain a specific headwater level that prevents dewatering (drying of the sand bed due to negative pressure within the filter caused by low inflow or no inflow) of the filter bed and subsequent formation of air bubbles within the sand media, a condition referred to as air-binding (Hendricks, 1991). Air bubbles block pore spaces and contribute to increased headloss (the loss of media permeability) (Hendricks, 1991).

2.2.4 Filter Run Length and Headloss Measurement

Filter run length is the length of time a filter can effectively operate before cleaning is required. The first step in cleaning involves stopping inflow and allowing the water to be drawn down to a depth approximately eight inches (20 cm) below the sand surface (Letterman, 1991). The exposed surface, consisting of the *schmutzdecke* and underlying sand is then scraped either manually (using rakes and shovels) or mechanically (using a small tractor), removing only the top inch (2 – 3 cm) of material from the surface (Ojha and Graham, 1994). Local economic conditions generally determine the most cost-effective cleaning method, but in either case, filters with long run lengths have lower cleaning costs whereas shorter run lengths have higher operational costs (Berg, Tanner, and Shieh, 1991). As such, it is important to know the filter run length associated with using a particular filtration media and a specific plant design in a particular location. Pilot studies are designed to provide information regarding filter run length by measuring 1) the rate of headloss development and 2) the length of time until terminal headloss, the point when the headwater reaches it's maximum height above the filter bed (Hendricks, 1991).

Headloss is a measure of the resistence to flow in media (Hendricks, 1991). This is measured in slow sand filters using a minimum of two piezometers; one tapped into the headwater and the other positioned in the gravel support layer of the filter (Hendricks, 1991). To calculate headloss, the drop in water level between two piezometer locations is measured (Hendricks, 1991). Placement of additional piezometers between the top and bottom of the filter bed provides insight to flow resistance (clogging of the filter media) at depths below the bed surface (Hendricks, 1991).

The rate of headloss and the maximum depth of the headwater determine the length of time a filter may operate before scraping of the upper surface is required to restore production levels. Once the filter has achieved terminal headloss, filter production is greatly reduced and the filter run is terminated so scraping may be done. The HLR, the turbidity of water entering, filter bed size and the uniformity of the filtration media affect the rate of headloss development (Di Bernardo and Rivera, 1996; Ellis, 1985). Since different sand size distributions influence rates of headloss, it is necessary to examine these characteristics when planning a slow sand filter (Hendricks, 1991). It is also important to determine the maximum turbidity levels present in the source water and the turbidity removal efficiency of the filtration process (Li et al., 1996). For this reason, it is desirable that pilot studies run throughout the course of a year to ensure that periods of high turbidity are included. Highly turbid water, that exceeding 50 NTU's, may require pre-filtration and multi-stage filtration to effectively remove large quantities of particulate matter (Ellis, 1985; Li et al., 1996; Shenkut, 1996).

2.2.5 Filtration Efficiency

Sampling ports and flow regulating valves (gate or ball type) are installed in slow sand filtration facilities to assist in maintenance and monitoring of water quality (fecal coliform levels,

turbidity, temperature, pH etc.). One goal of pilot plant studies is to determine the effectiveness of filter media in removing bacteria and reducing turbidity. Bacterial removal efficiency is determined through spiking tests (Hendricks, 1991). During such tests, high levels of fecal coliforms ($10{,}000^{+}$ per 100 ml) are added to the headwater and samples are drawn from both the headwater and tailwater for an extended period of time to determine the maximum number of bacteria entering and exiting the filter (Hendricks, 1991). A filter is considered mature when the maximum bacterial removal levels (calculated as percent bacterial removals and logarithmic bacterial removals) are obtained (Hendricks, 1991). This occurs after a period of ripening, during which time biological developments within the filter media are believed to reduce bacterial counts.

2.3 Summary of the Purpose of Pilot Studies

Prior to constructing the first slow sand filter in London, James Simpson constructed a small-scale pilot filter to test the feasibility of slow sand filtration as a means for treating London's source water, the Thames River (Collins et al., 1992). Since then, the use of pilot studies has become a standard practice for communities considering slow sand filtration technology (Hendricks, 1991). Due to the passive nature of slow sand filtration, few changes can be made to improve filtration capability once a full-scale plant has been constructed (Ellis, 1985). Therefore, the results from localized pilot studies are used to develop recommendations and establish design criteria for the construction of full-scale facilities.

In summary, a pilot study provides site-specific information regarding: 1) the treatability of raw water, 2) bacterial removal efficiency, 3) design criteria for a full-scale plant, 4) expected operational costs and, 5) the need for pretreatment. Temporal and spatial variability of turbidity, microorganisms, temperature, pH, and algae, as influenced by local factors (climate, geology,

precipitation, waste disposal etc), make pilot studies essential prior to constructing a full-scale plant. Furthermore, if a community intends to use local sand media, pilot studies are strongly recommended for evaluating the filtration effectiveness (bacterial removal) of such media (Pyper and Logsdon, 1988).

METHODOLOGY

3.1 Project Overview

The project objectives were accomplished on Kosrae using a pilot plant comprised of four test filters running simultaneously for six months. The island's remote location required significant coordination between WERI personnel, who were responsible for designing the pilot plant, analyzing field data, and monitoring performance, and personnel of the Kosrae Department of Transportation and Utility, who constructed the plant, provided local logistical support and assisted the Chief Sanitation Officer of Kosrae State in gathering field data following the first month of operation.

3.1.1 Overview of Plant Testing and Monitoring

The data gathered during the study consisted of two general types 1) that pertaining to bacterial spiking tests, and 2) field data obtained through daily monitoring of the filters (Figure 3).

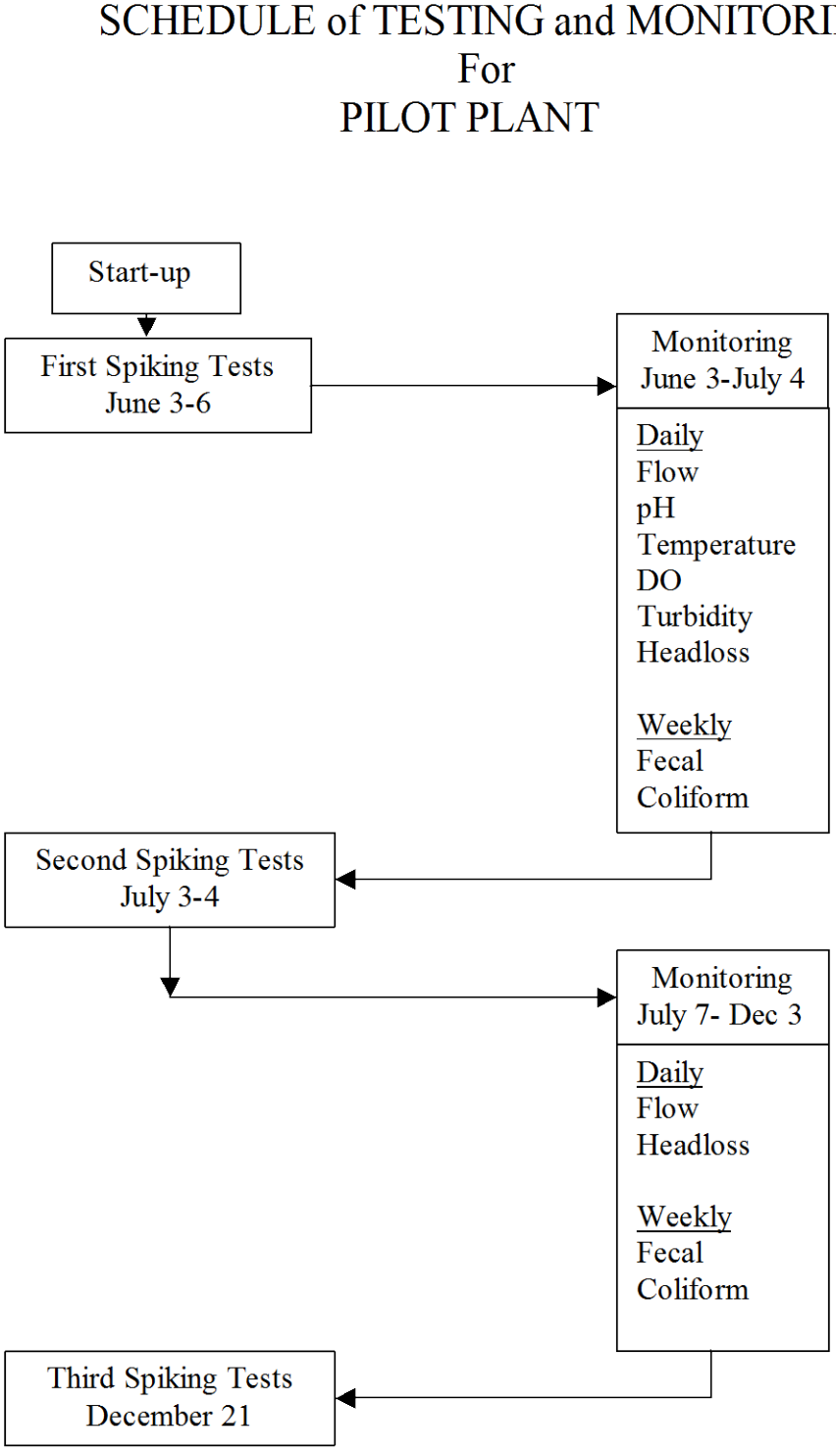


Figure 3. Schedule of testing and monitoring for pilot plant.

3.2 Experimental Site

Kosrae's geographical location and geological characteristics present factors that affect the quality of the island's surface water both temporally and spatially. Such factors include: 1) climate, 2) topography, 3) geology, 4) soils, 5) vegetation, and 6) animals. The amount of runoff and the material transported with runoff into streams is greatly affected by these factors. Furthermore, the turbidity of the island's streams is significantly affected by the frequency, intensity, and duration of rainfall events.

3.2.1 Geographical Location

Kosrae is situated in the tropical latitudes at 5 degrees 20 minutes N with a longitude of 163 degrees and 00 minutes E making it the easternmost island of the Caroline Islands (U.S. Army Corps, 1985). The island has an area of approximately 42 square miles (109 square kilometers) (U.S. Army Corps, 1985).

3.2.2 Climate, Topography, Geology and Soils

Kosrae's tropical climate contributes significantly to the weathering of parent rock material, which in turn affects the chemistry of the island's streams. According to the 1984 United States Geological Survey (USGS) water resources report, Kosrae's average annual temperature is 27.4° C (81.3° F). Annual mean evaporation is estimated to be 66 inches (168 cm), while relative humidity varies from 80-90 percent. These extreme tropical conditions accelerate weathering of the steep, volcanic mountains covering approximately 70 percent of the island. Fifteen percent of the land area consists of gentle foot slopes, alluvial fans and bottomlands. The remaining 15 percent of the islands' surface area consists of sandy beach strands and mangrove swamps (U.S. Army Corps, 1985).

Seventeen distinct soil types have been identified on Kosrae (U.S. Dept. of Agriculture, 1983), with the majority of soils being derived from igneous parent rock (basalt, andesite and trachyte lava flows and dikes) (U.S. Army Corps, 1985). The soil survey conducted in 1983 by the United States Department of Agriculture (USDA) identified the following soil characteristics associated with Kosrae's varied topography: 1) mountainous areas have shallow to moderately deep, well-drained gravelly material, 2) upland soils tend to be well-drained; very shallow to very deep; and steep to extremely steep, 3) bottomlands are very deep; somewhat poorly drained to very poorly drained; and level to nearly level.

The 1983 survey also classified the soils as being slightly to strongly acidic as a result of leaching and the associated loss of soluble bases and nutrients.

3.2.3 Physical Location of Pilot Plant on Kosrae

The pilot plant was constructed in the municipality of Tofol as shown in Figure 4. The plant was supplied by gravity feed with surface water taken from the Tofol diversion site on the Tofol River. The concrete dam is situated approximately 120 feet (36 m) above sea level (WHO, 1981) and is roughly 10 feet (3 m) wide and four feet (1.2 m) high situated 2000 feet (610 m) upstream from the pilot plant.

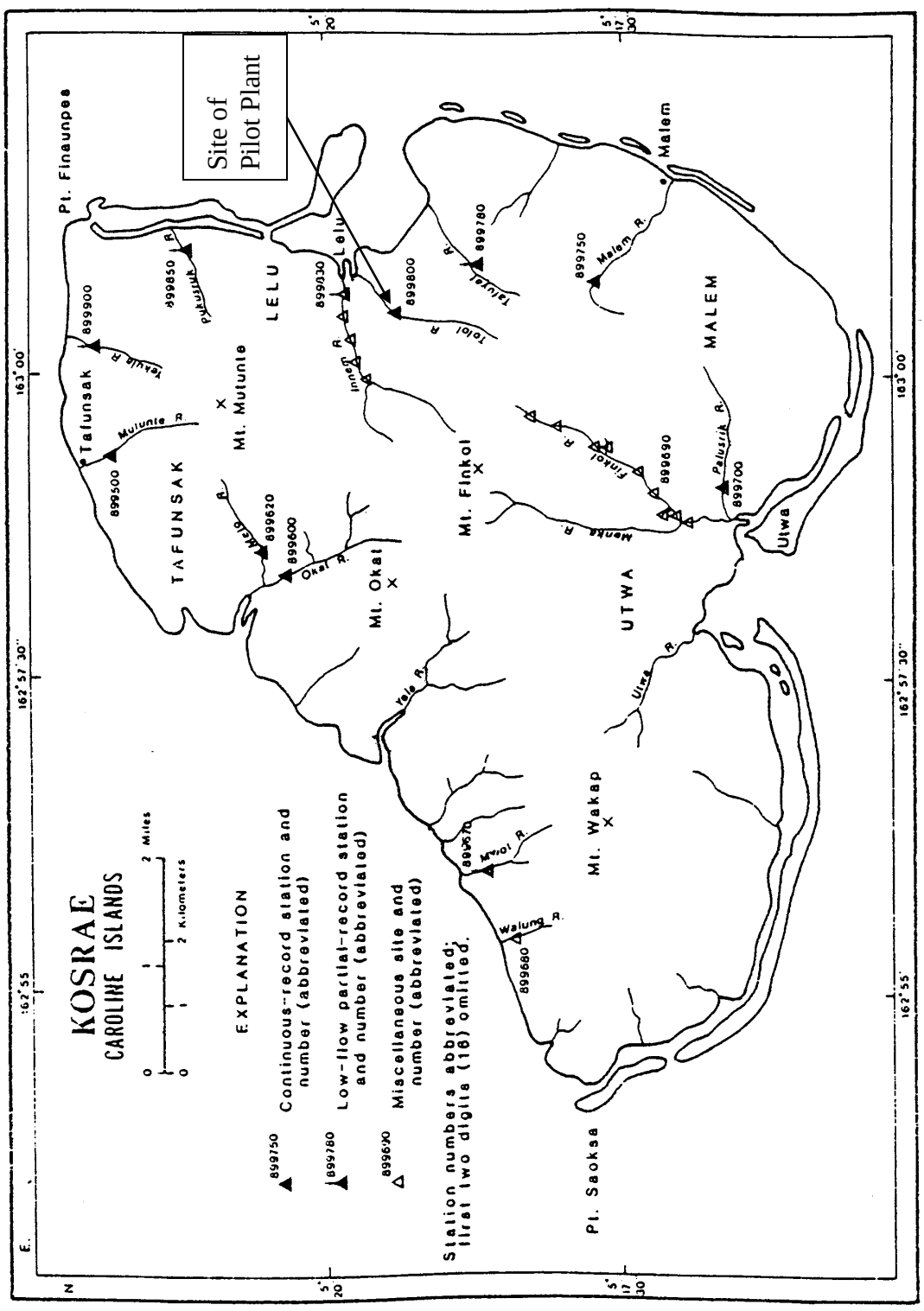


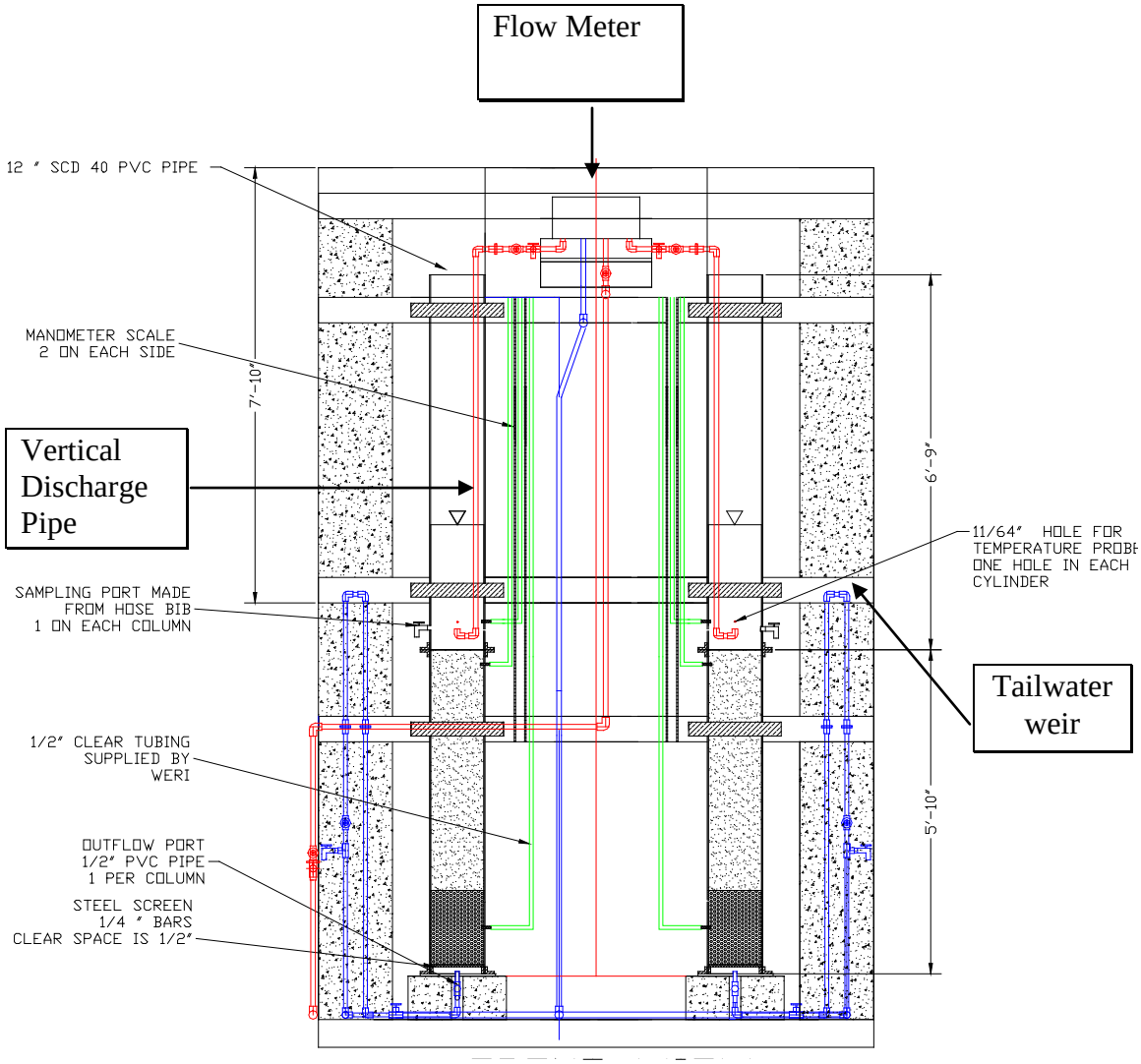
Figure 4. Map of Kosrae showing location of pilot plant.

(Source: US Geological Survey)

3.3 Pilot Plant Design

WERI project engineers, Dr. Shahram Khosrowpanah and Dr. Leroy Heitz, modeled the pilot plant design according to recommendations presented in The Manual of Design for Slow Sand Filtration (Hendricks, 1991). The project's remote location and budgetary concerns were primary factors in the selection of this design. Four 12 .5 feet (3.8 m) tall test filters were used in the study. Front, top and side views of the pilot plant are shown in Figures 5 through 7. Figure 8 shows the actual plant. Each test filter was constructed by joining two pieces of 11.5-inch (29.2 cm) interior diameter poly-vinyl chloride (PVC) pipe with a flange coupling. The filters were bolted to a cinder-block pedestal anchored to a concrete slab.

The filters were divided into two pairs, with one pair containing sand manufactured on Kosrae from local crushed basalt and the other pair containing imported, quartz-based sand formulated to meet specifications for slow sand filtration applications.



* Inflow shown in blue; metered and filtered water shown in red

Figure 5. Front view of pilot plant.

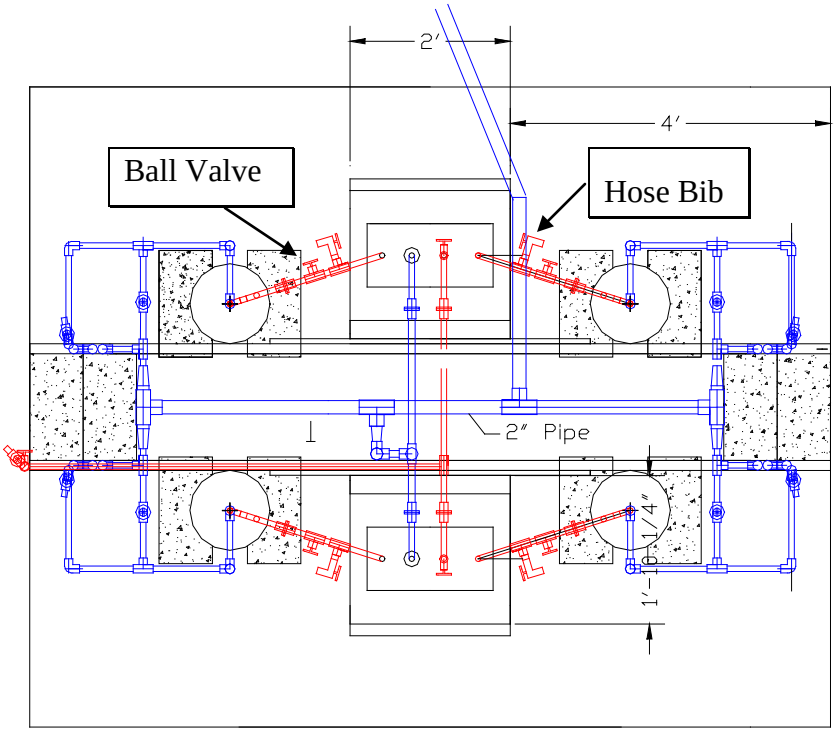


Figure 6. Plan view of pilot plant.

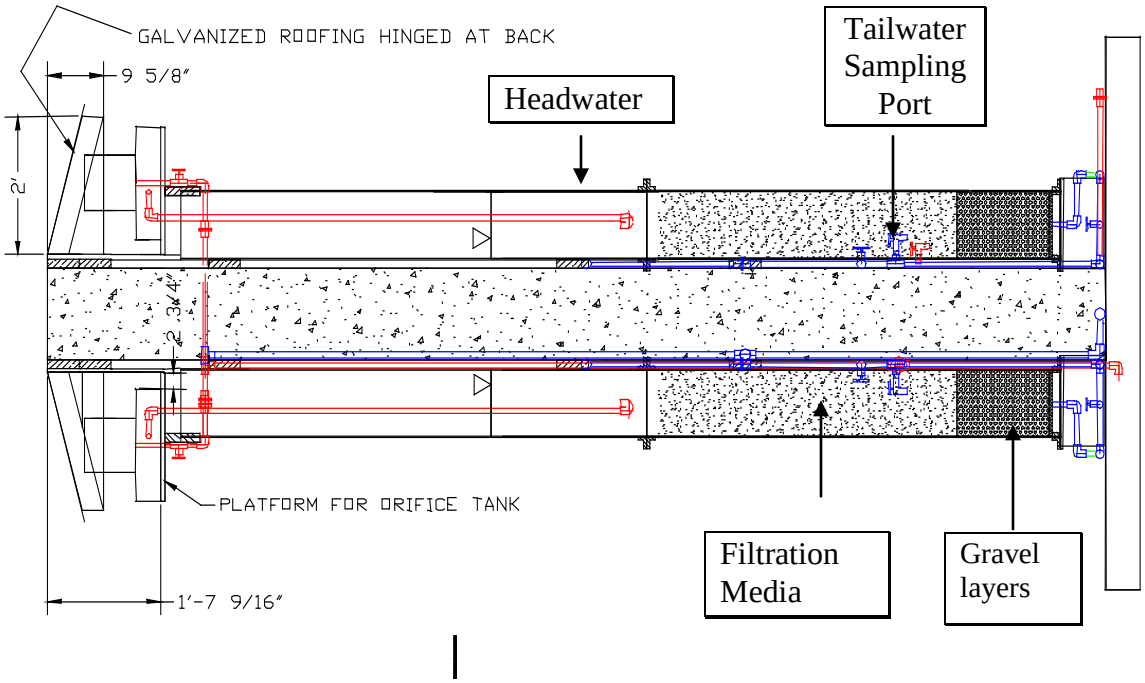


Figure 7. Side view of pilot plant.

Figure 8. Photo of actual pilot plant.

3.3.1 Filtration Media

The filtration media was tested using sieve analysis (ASTM, 1966) to determine the size distributions for both local and imported sand (Table 2). This analysis technique allows for determination of the d_{10} and d_{60} of the sand (Fetter, 1994). The d represents the diameter of particles passing through a given sieve size (estimated from curve shown in Figure 9). The d_{10} (effective grain size) is determined by the sieve size that 10 percent of a sample (by weight) passes through during a sieve test. The d_{60} represents the particle size that 60 percent of a sample (by weight) passes through in a sieve test. Combining these two values as a ratio of d_{60}/d_{10} allows for determination of the uniformity coefficient (UC) of the material (Fetter, 1994).

Table 2

Results of Sieve Analyses for Imported and Local Media

Sieve Analysis				
Sieve Size	Opening passed through (in)	Opening passed through (mm)	Imported % passing by weight	Basalt % passing by weight
3"/2"				
1-1/2"	1.5	37.5		
1"	1	25		
3/4"	0.75	19		
1/2"	0.5	12.5		
3/8"	0.375	9.5		
No. 4		4.75		
No. 8		2.36		
No. 10	0.0787	2		97.7
No. 16		1.18		63.1
No. 20		0.85		46.3
No.30	0.0236	0.6	93.3	35.1
No.40		0.425	22.8	22.9
No.50		0.3	8.5	15.0
No.80/100		0.15	1.5	1.8
No. 200		0.075	0.2	0.8
PAN				

The UC is a reflection of the degree of variation in particles sizes (Fetter, 1994). A lower UC indicates more uniformity in particle size, which generally results in a higher porosity, assuming the particles are uniform in shape (Fetter, 1994). A higher UC indicates greater variation in particle sizes and usually indicates reduced porosity, as the voids created by larger particles fill with smaller sizes. These characteristics of uniformity coefficients serve as guidelines for determining porosity, however, the geometry of the sand particles has a considerable impact on the degree of sorting and hence, porosity of the media (Fetter 1994). The graphical representation of the size distribution of the sieve analyses is shown in Figure 9. The graph provides a means for obtaining the diameters of various percentages (10%, 60% etc.) of particles passing through a particular sieve. The UC's obtained for both local and imported media are presented in Table 3.

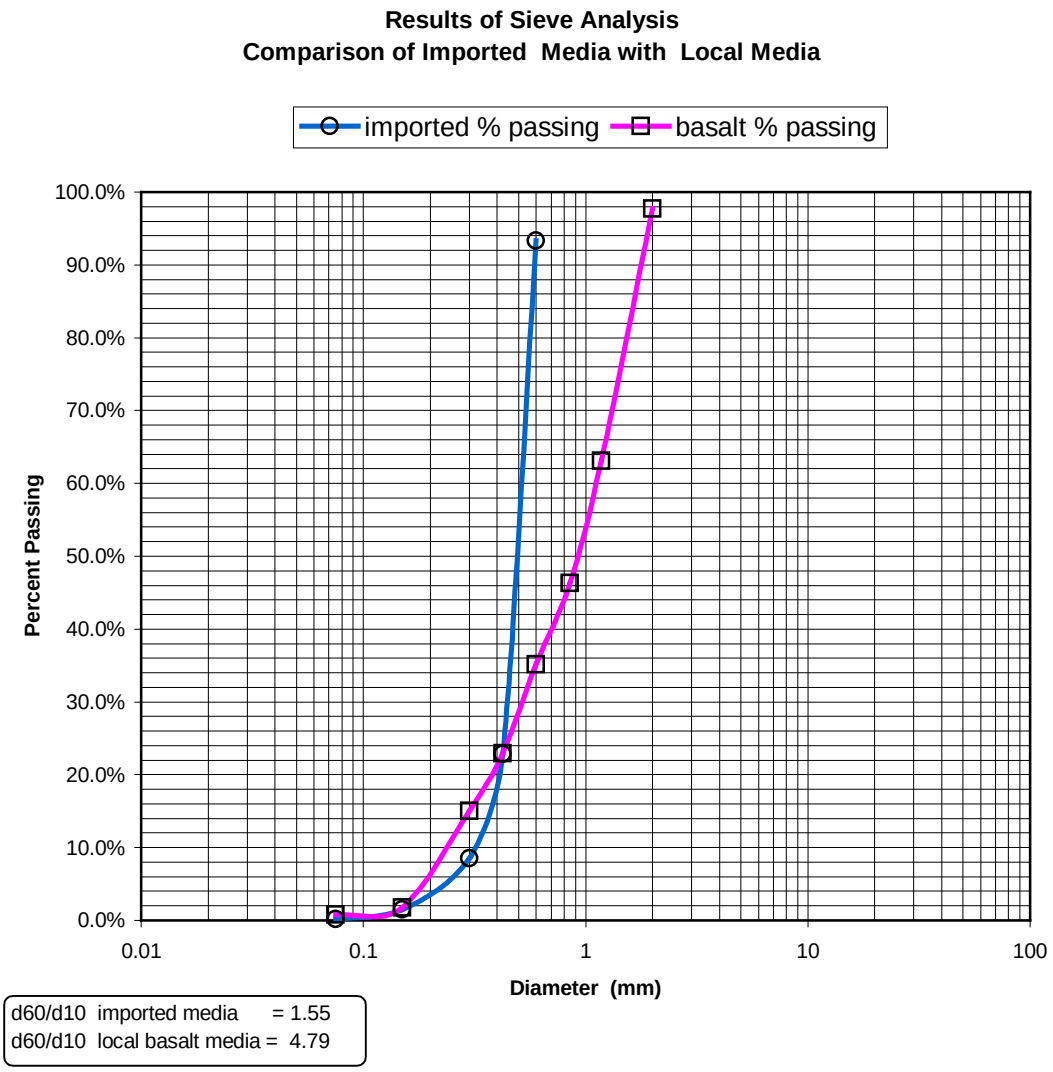


Figure 9. Size distribution curves from sieve analysis of local and imported media.

Table 3

Comparison of d10, d60, and UC for Both Types of Media

Filter Media	Size of Percent Passing (mm)		Uniformity Coefficient (UC)
	d10	d60	d60/d10
Local Sand	0.24	1.15	4.79
Imported Sand	0.33	0.51	1.55

The uniformity coefficient of 1.55 for the imported sand was less than 3, and therefore considered an acceptable value for filtration (see Table 1). The uniformity coefficient of 4.79 for the local media, however, exceeded the preferred upper value of 3 and was near the maximum acceptable value of 5. Such a value generally indicates reduced porosity (Fetter, 1994). However, the sharp geometry of the sand manufactured on Kosrae appeared to have substantially increased the initial porosity, as demonstrated by extremely long initial filter run lengths (as discussed in Chapter 5).

The locally manufactured sand media's UC was the lowest obtainable using the equipment configuration available in Kosrae at the time of the study. The island's rock crusher was set-up for preparing road-paving aggregate, making it unfeasible to set the crusher and screens specifically for the sizes needed in the pilot filters. Therefore, the basalt sand media was obtained by sieving the by-products of the road project aggregate for sand sizes suitable for use in the filters. If a full-scale plant is constructed using local basalt sand, the crusher and screens can be adjusted to obtain a lower UC for the filtration media.

3.3.2 Gravel Sizing

The gravel used to support the filtration media in all filters was local basalt that was washed prior to being poured into the filters. Determination of the correct sizes of gravel and hence, the number of gravel layers required, was accomplished using the rules presented in the Manual of Design for Slow Sand Filtration (Hendricks, 1991). These rules are presented in Appendix A.

The overall goal in gravel sizing for a slow sand filter is to obtain an aggregate size for the top gravel layer that will prevent the sand media from passing through it, while selecting an aggregate size for the bottom layer that is large enough to be retained by the bottom screen or

perforated underdrain pipes. Once the aggregate sizes for the top and bottom gravel layers have been calculated, the need for additional gravel sizes and layers is determined.

If the aggregate size of the top layer allows it to pass into the bottom layer, then an intervening aggregate layer is needed. The sizing for this additional layer is tested against the aggregate sizes of the top and bottom layers to ensure that gravel does not pass from an upper layer into the layer beneath it. The process of adding intervening layers of aggregate is repeated until the gravel sizes no longer pass through to the next layer.

In the pilot study, the sand size distributions and the size of the bottom screen openings required that three layers of gravel be used to meet these conditions (Table 4). The screens used are pictured in Figure 10.

Table 4

Aggregate Sizes Used in Filters

	LOCAL MEDIA (inch)	IMPORTED MEDIA (inch)
BOTTOM LAYER	3/4 minus	3/4 minus
MIDDLE LAYER	3/8*	3/8*
TOP LAYER	1.18-2.25 (mm)	1.18-2.25 (mm)

*note the middle layer used 3/8 inch aggregate and aggregate specifically retained by the next two smaller diameter screens (# 4 and # 8)

Figure 10. Photo of gravel retaining screen.

3.3.3 Packing of Filters with Gravel and Sand Media

The first procedures for filter start-up involved packing the filters with gravel followed by sand media. All filters were packed dry. For ease in packing, the upper section of pipe for each filter column was released from the flange coupling that joined the upper and lower pipes. The upper sections were lifted from the coupling using a crane (later done by hand). Once the lower section of PVC pipe was exposed, the gravel layers were added.

The gravel had been screened previously, however, a second screening was done on-site to ensure proper sizing. To guarantee that each size would provide a depth a six inches (15.2 cm), a section of PVC pipe identical to that of the filters was cut to a length of six inches (15.2

cm), placed in an upright position, and closed temporarily at one end with a piece of plywood. The gravel of a specified size was then poured into this 6-inch (15.2cm) tall section of pipe. Once the correct amount of gravel was measured, it was poured into a filter cylinder. The gravel was then spread evenly in the filter using a 10-foot (3m) length of 1/2 inch (1.3cm) PVC pipe. This process was repeated for all three layers of gravel in each filter. After the gravel was in place, the sand media was poured on top.

All four filters were to be packed with pre-washed sand to minimize the amount of fines contributing to initial tailwater turbidity. However, one of the filters (Filter 2) containing imported sand was accidentally filled with dry, unwashed sand. Given the time and difficulty involved in removing the sand, it was decided to let it remain. In all four filters, the sand was poured to a depth of 48 inches (121cm). This brought the top of the sand layer, to a level just slightly below (approximately 1 inch or 2.54 cm) the top of the lower section of the filter cylinder.

After the sand was in place, the upper piece of PVC pipe was repositioned and re-attached to the lower portion of the filter cylinder using the connecting flange.

3.3.4 Flow Regulation

Flow to the filters was regulated using two constant-head orifice flow meters with each meter regulating flow to two filters of the same type media. The design for the constant-head orifice flow meters was selected by the project engineers based one provided in the Manual of Design for Slow Sand Filtration (1991). Figure 11 contains a three-view drawing of the flow meter. The meters were constructed from quarter-inch plexi-glass and PVC pipe. They were calibrated at WERI to determine discharge at a given head. This was accomplished by adjusting the standpipe inside the meter reservoir box and recording the amount of discharge (Q) in units

of mass per unit time (g/s) for a given head. These discharge values were then converted to volume per unit time (ml/s). Calibration results showed consistent flow between the orifices of each meter and between the two meters. Each meter supplied flow to two filters containing identical filtration media.

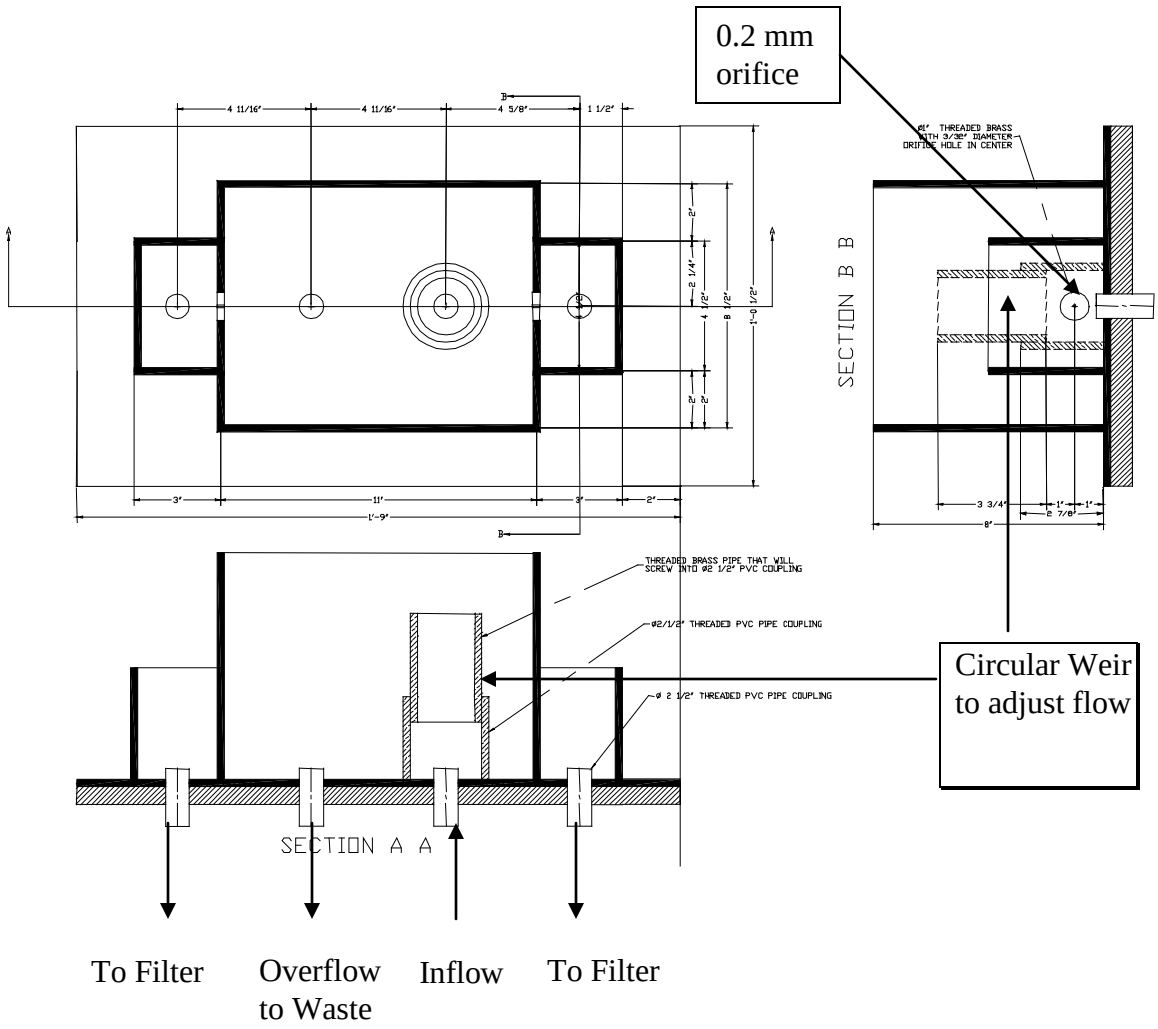


Figure 11. Flow meter diagrams.

3.3.5 Inflow and Outflow Regulation

Raw (untreated) stream water was diverted to the plant from the main water line supplying the municipalities of Tofol and Lelu. The connection of the diversion line to the main was made approximately 20 feet (6 m) in front of the pilot plant using 3/4 inch (1.9 cm) PVC pipe. This line to the plant was then reduced to 1/2 inch (1.3 cm) PVC pipe at the base of the filter system (see Figure 5). Water then flowed through this pipe up to the two flow meters situated on 3/4inch (1.9 cm) plywood platforms 15 feet (4.6 m) above the concrete slab (Figure 12).

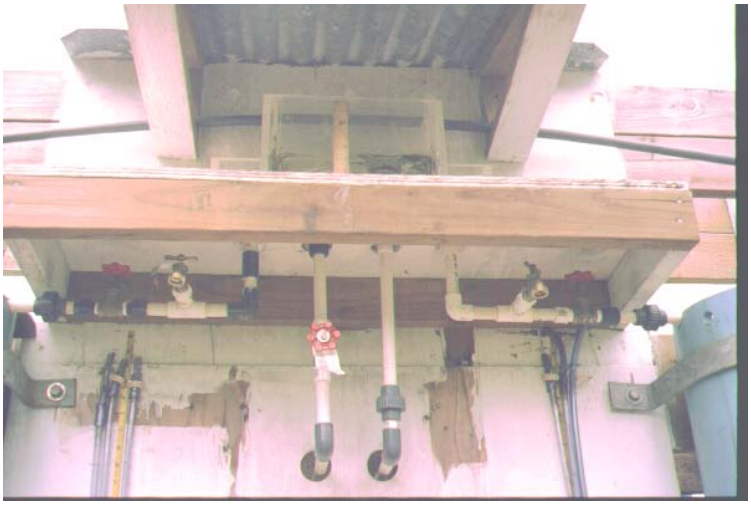


Figure 12. Photo of flow meter showing flow to two filters.

Water discharging from the flow meters passed through a horizontal 1/2 inch (1.3 cm) PVC pipe leading to the top of each filter (Figure 5). The horizontal pipe was fitted with a ball valve and a hose bib (Figure 6). These features were necessary for controlling flow during flow measurements. These measurements were accomplished by closing the ball valve to stop flow into a filter, followed by opening of the hose bib to divert flow from the line for measurement.

The terminal end of the horizontal pipe was supported by the top of the filter column, making a 90-degree elbow connection to a vertical ½ inch PVC pipe that directed water downward into the filter (Figure 7). The discharge end of the vertical pipe was positioned approximately 16 inches (40.6cm) above the sand bed in the center of the filter column to prevent development of preferential flow paths along filter walls (Figures 5 and 7). Submergence of the discharge end at all times, combined with a U-shaped discharge terminus, allowed for dissipation of energy from inflow water thereby preventing sand bed erosion. Complete submergence of the discharge line was accomplished by backfilling each filter at start-up and using tailwater weirs on the outflow lines to maintain a headwater above the discharge level.

The tailwater weirs consisted of inverted, U-shaped, PVC pipe with holes drilled at the top of the inverted "U" (Figure 13). A weir was installed in the 1/2 inch (1.3 cm) outflow line exiting the base of each filter (Figure 5 and Figure 13). After the water moved through the filter media, it traveled from the base of each filter upwards to the weir, across the weir, and then downward and out of the 1/2 inch (1.3 cm) line into a line collecting outflow from all filters. This line discharged water into the Tofol River directly behind the plant. The weirs were attached to the concrete support columns, at a height 16 inches above the level of the sand bed (Figure 13). This maintained a 16-inch depth of water above the sand bed during periods of low inflow or no inflow, thereby preventing bed erosion and de-watering of the filters.

Figure 13. Tailwater weir.

3.3.6 Sampling Ports for Headwater and Tailwater

Sampling of the headwater was accomplished using a hose bib situated 8 inches (20.3 cm) above the sand bed (Figure 5). Tailwater was sampled using a hose bib installed on the downward section of pipe exiting the weirs (Figure 7).

3.3.7 Headloss Measurement

Piezometers were installed in three locations on each filter to provide headloss information across three sections of the filter bed. The piezometer locations were classified as top, middle and gravel based on the positions of the piezometers relative to each other and the filter column itself. The middle piezometer was situated between the top and gravel piezometers, but it was not in the middle of the sand bed. The top piezometer was situated 12 inches (30.5 cm) above the sand bed; the mid-level piezometer was set 8 inches (20.3 cm) below the upper surface of the sand bed; and the third piezometer was positioned in the supporting gravel beneath 8 inches (20.3 cm) of aggregate. Clear and opaque blue plastic ½ inch (1.7 cm) tubing was used for the piezometers. The scale used for piezometer readings was devised from a fiberglass reinforced engineering-scale tape measure cut into lengths and stapled to the plywood backing.. Daily headloss was calculated using the difference in peizometer readings between 1) the headwater and the mid-level peizometer, 2) the mid-level piezometer and the gravel peizometer and 3) the headwater and the gravel peizometer. These differences were recorded and plotted as a time series for each filter.

Terminal headloss for a given filter was achieved when the headwater level reached the top of the filter's upper PVC cylinder and overflowed. This was at a level of approximately 5 feet (1.5 m) above the top surface of the filter bed. Filters were scraped upon reaching terminal headloss and then brought back on-line.

3.4 Pilot Plant Start-Up Procedures

3.4.1 Back Filling

Filter start-up involved back filling until a headwater deep enough to submerge the U-shaped discharge pipe (16 inches/40.6 cm) was established. This process is the preferred method for starting the filter, as it forces air trapped in the pore spaces of the gravel and sand media, up and out of the filter, reducing the likelihood of preferential flow paths developing within the sand media (Hendricks, 1991). Additionally, the process prevents erosion of the sand bed as may occur when filling is from the top down and water discharges onto the exposed sand surface (Hendricks, 1991).

Back filling was accomplished using raw water piped to the pilot plant via the diversion line connecting the pilot facility to the municipal supply line. Water from the diversion line was delivered to the effluent pipe of each filter via a garden hose connecting the filter effluent pipe to a hose bib situated on the diversion line. The rate of back filling was measured using piezometer levels and a stopwatch. This provided a back-filling rate in units of feet/minute. The back-filling procedure ended when the headwater was level with the height of the tailwater weir (16 inches or 40.6cm of depth). Rates of backfilling varied from 1 ft/hour to 2.5 ft/hour (0.3 m/hour – 0.76 m/hour) well below the recommended rate of < 6.5 ft/ hour (< 2 m/hour) cited in Hendricks (1991).

3.4.2 Metered Flow

Following the back-filling procedure, each filter was ready to be brought on line by receiving metered flow of raw water. The flow meters were set to a rate of 4 ml/s based upon the hydraulic loading rate (HLR) chosen for the study. A hydraulic loading rate of 0.2 m/hr was selected based on: 1) previous pilot studies with hydraulic loading rates ranging from 0.1 m/hr-

0.4 m/hr and 2) limited knowledge of the Tofol River turbidity levels. The higher the turbidity, the lower the HLR should be to extend filter run length. Since the rate of 0.2 m/hr falls in the middle of the range used in numerous pilot studies and knowledge of local turbidity levels was minimal, the 0.2 m/hr HLR was selected.

Daily flow rate was measured using a 100 ml graduated cylinder and a stopwatch (Figure 14). This procedure involved turning off the flow of water to the vertical, U-shaped discharge line and opening a hose bib situated on the horizontal section of the discharge line between the flow meter and U-shaped section of the line (Figure 6).

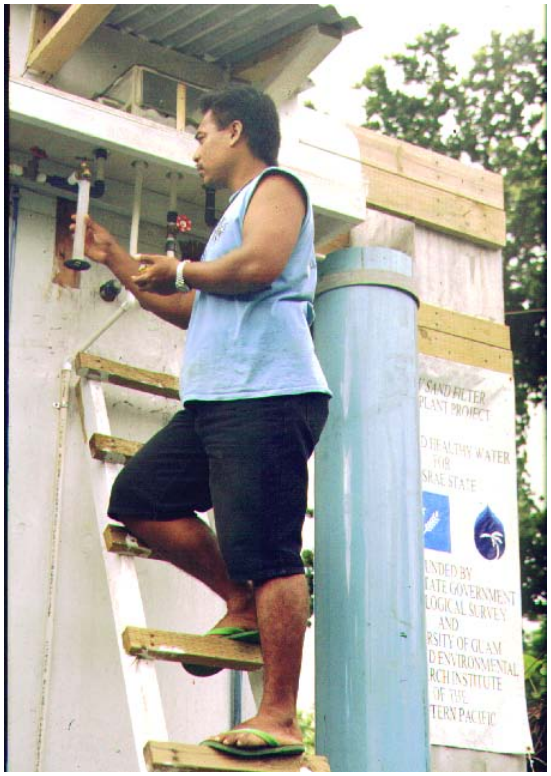


Figure 14. Flow measurement.

3.5 Determination of Bacterial Removal Efficiency and Filter Maturation

3.5.1 First Bacterial Spiking Tests

Bacterial spiking tests were used to determine the length of time for filter ripening and maturation. Ripening refers primarily to the development of a biofilm surrounding the particles of filtration media but also includes the *schmutzdecke* (Ellis, 1985). A filter has matured and is ready to operate in production mode when such features are fully developed and bacterial removals are maximized (Hendricks, 1991).

The filters were brought on-line individually due to labor limitations and incubation space constraints imposed by the bacterial spiking tests conducted immediately following start-up of each filter. The filters were numbered according to the order in which they were brought on-line beginning June 03, 2000. Filters 1 and 2 were those containing commercial sand media; Filters 3 and 4 contained local sand media (see Figure 6).

The bacterial spiking procedure involved adding a concentrated stock solution of fecal coliform bacteria to the filter headwater. The stock solution was created by dissolving a freeze-dried pellet (Microbiologics stock #0335P) containing fecal coliforms in 100 ml of distilled water. The solution was mixed in a 100 ml sterile plastic bottle. The contents were shaken until the pellet was dissolved, then the solution was poured into the top of the filter. Immediately thereafter, the headwater was stirred to ensure sufficient mixing. Stirring was done carefully (to avoid disturbing the top of the sand bed) with a 10-foot (3 m) section of 1/2-inch (1.3 cm) PVC pipe. Following the mixing procedure, samples from the headwater and tailwater were taken and a start-up time was recorded. Elapsed time from start was recorded in hours and minutes for each sample drawn. A sampling time series of eight hours was selected for the first spiking tests on each of the four filters (Table 5). The eight-hour duration was selected using a theoretical

estimate of 6.1 hours for the water to migrate from the headwater through the filter. This value was obtained by dividing the HLR (0.2 m/hr) by bed depth (1.22 meters).

Table 5

Sampling Time Series for the First Spiking Tests

ET (hr:min)	0:00	0:30	1:00	2:00	4:00	6:00	8:00

One water sample was taken from the headwater and tailwater at each of the times shown in Table 5. The headwater samples were taken after letting the water flow through the headwater hose bib for 5 seconds. The short length of the hose bib and the need to prevent significant reduction of bacteria by overdrawing the headwater dictated that minimal water be used to flush the hose bib when drawing headwater samples. Tailwater samples were taken after 15 seconds of flow because of the greater length of the effluent pipe.

The IDEXX© 18-hour Collilert© Quanti-tray© method of bacterial enumeration was used for determining the Most Probable Number (MPN) of fecal coliforms per 100 ml water sample. As stated in Standard Methods for the Examination of Water and Wastewater 20th Edition 1998:

> This is an index of the number of coliform bacteria that, more probably than any other number, would give the results shown by the laboratory examination; it is not an actual enumeration.

All water samples were collected in sterile 100 ml IDEXX© bottles, then placed immediately in a small cooler (containing ice packs) and transported to the lab facility approximately 400 m away. The samples were placed in the lab refrigerator at a temperature of 12° C until time for dilutions and incubation.

Samples were stored for a maximum of eight hours, prior to being diluted with distilled water, mixed with 18-hour culture media and placed in the incubator. The number of sample

dilutions was established using the potentially highest number of bacteria present in the samples. Tremendous uncertainty existed regarding the maximum values, since no baseline data on ambient (river) fecal coliform concentration was available and two preliminary tests with the bacterial pellets at WERI showed variability (one order of magnitude) in numbers of live fecal coliforms. Given these uncertainties, five dilutions for both headwater and tailwater were performed on samples taken during the first spiking event (Table 6).

Table 6

Dilution Factors Selected for the First Spiking Tests

Source	Dilutions				
Headwater	x 25	x 50	x 100	x 1 000	x 10 000
Tailwater	x 1	x 10	x 25	x 50	x 100

Following the dilutions and addition of media, the samples were sealed in Quanti-trays© and placed in an incubator. The incubator was fashioned from a Coleman cooler warmed by a water bath. The water bath consisted of a 1 gallon Clorox bottle containing an aquarium-heating element immersed in water. This apparatus was situated in the cooler with the cord from the heating element being passed through a rubber stopper placed in the drainage hole of the cooler. Electronic thermometer probes were positioned at three levels (bottom, middle and top) in the cooler yielding an average temperature of 37° C following a twenty-four hour equilibration period. The individual readings varied from 34° C at the bottom to 39° C at the top of the cooler. As sample trays were placed in the incubator, a hairdryer was used to force warm air between the sample trays to maintain the incubator temperature.

Following the 18-hour incubation period, results were recorded in data tables arranged as shown below (Table 7).

Table 7

Data Table for Bacterial Spiking Tests

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7
ET (hr:min)	Sample Source	Dilution Factor	Large Cell (+)	Small Cell (+)	Table Value	MPN

Column (1) indicates time elapsed (ET) since introduction of the bacteria (spiking) into the headwater. Column (2) identifies the source of the sample (headwater or tailwater). Column (3) identifies the dilution factor for the sample. Columns (4) and (5) show the actual number of cells that were positive for fecal coliforms in the 97-cell Quanti-tray. Column (6) identifies the probability value associated with the cell counts listed in columns (4) and (5). This value was obtained from the probability table published by IDEXX for the Quanti-tray system. Column (7) represents the most probable number (MPN) of fecal coliforms as calculated using the following equation MPN:

MPN = (Dilution Factor {Column 3}) x (Table Value {Column 6}) (2)

The estimated MPN's of headwater and tailwater for each sample were determined by averaging the MPN's of the dilutions. The average values were then plotted for the corresponding ET and maximum concentrations of headwater and tailwater fecal coliforms were determined.

To determine the bacterial removal efficiency of the filters, percent removals of fecal coliform bacteria were calculated using the following equation (Hendricks and Bellamy as presented in Logsdon, 1991):

$$\% R = \frac{C_i - C_e}{C_i} \times 100 \quad (3)$$

% R = percent removal
C_i = Influent concentration (Average of Maximum Head Sample MPN's)
C_e = Effluent concentration (Average of Maximum Tail Sample MPN's)

The logarithmic (LOG) removal of bacteria was calculated using the following equation:

$$LOG\ R = LOG\ (C_i) - LOG\ (C_e) \quad (4)$$

LOG R = logarithmic bacterial removal
C_i = Influent concentration (Average of Maximum Head Sample MPN's)
C_e = Effluent concentration (Average of Maximum Tail Sample MPN's)

3.5.2 Second Bacterial Spiking Tests

A second spike event was conducted approximately 30 days after the initial spike tests to determine if the filter beds had ripened, and therefore increased their respective bacterial removal efficiency.

The procedures and analysis techniques outlined for the first spiking test were repeated with three modifications. The first modification was an increase in elapsed time for sampling from eight hours to sixteen hours (Table 8). This adjustment was made to provide a more accurate estimate of the maximum levels of fecal coliforms exiting the filters as results from the first spike suggested that the length of time needed for the water to pass through the filters was greater than initially estimated (> 6.1 hours).

Table 8

Sampling Time Series for Second Spiking Tests

ET (hr)	0:00	2:00	4:00	6:00	9:00	12:00	16:00

The second modification was a reduction in the number of dilutions (Table 9). This adjustment was made using results obtained in the first spike tests, that indicated sufficient estimates of the MPN's could be made using fewer dilutions.

Table 9

Dilution Factors Selected for Second Spiking Tests

Source	Dilutions		
Headwater	x 25	x 50	x 100
Tailwater	x 1	x 10	

The third modification was spiking two filters simultaneously, completing the spiking event in two days instead of four. This was possible due to the overall reduction in the number of samples being incubated. Further discussion regarding these adjustments is presented in section 4.

3.5.3 Third Bacterial Spiking Tests

A final spiking test was conducted in December of 2000 following six months of plant operation. The testing protocol was identical to that of the second spike test, however, only two of the four filters were spiked due to budgetary and time constraints. Given these limitations, the goal of the third spike was to test one filter containing each type of media and to spike filters that were considered mature based on headloss and cleaning records.

3.5.4 Measurement of Dissolved Oxygen

Headwater and tailwater dissolved oxygen (DO) concentrations were measured during the first month to assist in determining bed maturation due to aerobic consumption. A difference of 2 mg/L to 4 mg/L between the headwater DO concentration and tailwater DO concentration has been identified as a potential indicator of bed maturity (Hendricks, 1991). Standard Methods

(1998) were followed in gathering samples and in using the dissolved oxygen meter. Differences in headwater and tailwater DO concentration were calculated.

3.6 Daily and Weekly Monitoring

During the first month of operation, all four filters were monitored daily for flow rate, headloss, turbidity, pH, dissolved oxygen (DO), and temperature. The MPN of fecal coliforms was obtained from weekly headwater and tailwater samples tested using the IDEXX© 18-hour Collilert© Quanti-tray© method of bacterial enumeration. During this time, personnel from both the Kosrae Office of Sanitation and the Kosrae State Department of Transportation and Utilities were trained in plant operations, bacterial analysis, flow measurement and piezometer reading. Proper recording techniques were taught using data sheets modified according to observations during training sessions and input received from Kosraen personnel. After the first month, all field data was input into an Excel spreadsheet by personnel at the Kosrae State Department of Transportation, and then e-mailed to WERI on a bi-monthly basis for analysis.

Twenty weeks of data were gathered following the first month of operation. Single samples of each headwater and tailwater were tested for fecal coliforms. The dilution factor for weekly testing was set at $\times 1$ for headwater and tailwater samples given ambient levels of bacteria in the Tofol River (<5,000 fecal coliforms/100 ml) which did not warrant multiple dilutions. Procedures for calculating the MPN's of fecal coliforms in weekly samples were identical to those described previously for spiking tests.

A Model II Anova (single factor, random effects model) (Zar, 1984) was selected as the statistical test for determining differences in performance of filtration media. A randomized design statistic was selected since the filtration media was not specifically chosen from all

possible materials, but rather represented what was both locally available and commercially available within the parameters for filtration media.

The single factor for the tests was filtration media with two levels (local basalt sand and imported quartz-based sand). The null hypothesis for a given variable (fecal coliform concentration, pH, turbidity) was that no differences existed among filters.

Weekly fecal coliform data was grouped for analysis on two time scales. The first time scale was established for making monthly ANOVA comparisons of bacterial levels in the respective filters. This was constructed by taking four consecutive weeks of data and grouping them to obtain a comparison of headwater and tailwater bacterial counts between filters, on a monthly scale. The second time scale was an overall grouping of the twenty weeks of data to obtain a five-month ANOVA.

Problems with the turbidimeter and pH meter provided by Kosrae State prevented consistent collection of turbidity and pH data after the first month of the study. Therefore, turbidity analysis was only applied to data obtained during the first month of plant operations and a three-week interval during September. The analysis of pH was also applied to only the first month's data. Tukey's honestly significant difference (HSD) test was used to specifically identify filters showing significant differences in these values.

RESULTS AND DISCUSSION

4.1 Filter Run Length

4.1.1 Analysis of Run Length for all Filters

Filtration run times varied for all filters, as shown in Table 10 and Figures 15 through 18, due to different rates of headloss development. It should be noted that several days elapsed between the time a filter reached terminal headloss and the time it was scraped, therefore the total days of run time do not add to 180-183 (length of the study; days varied due to staggered start of filters) for any filters except Filter 4. The + indicates that a run was still in progress at the end of the study. Filter 1 required the most frequent scraping and it experienced the shortest succeeding run times of 24 days, 21 days and 33 days respectively. Filters 2 and 3 were scraped only once and had succeeding run times of 68 days and 42 days respectively. Filter 4 was not scraped, as it did not reach terminal headloss during the 180-day study (see Appendix B for complete headloss data).

Table 10

Summary of Filter Run Length for all Filters

Filter	Filter Run Lengths			
	First Run (days)	Second Run (days)	Third Run (days)	Fourth Run (days)
1	58	24	21	33
2	78	68 +	----	----
3	114	42 +	----	----
4	180 +	----	----	----

(+) run in progress when study ended; (---) no run

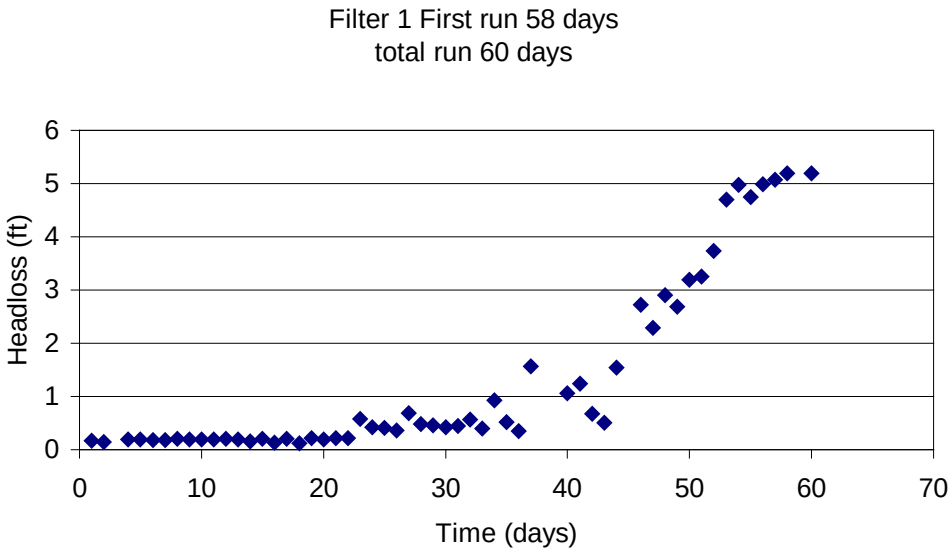


Figure 15. Filter 1 first run length.

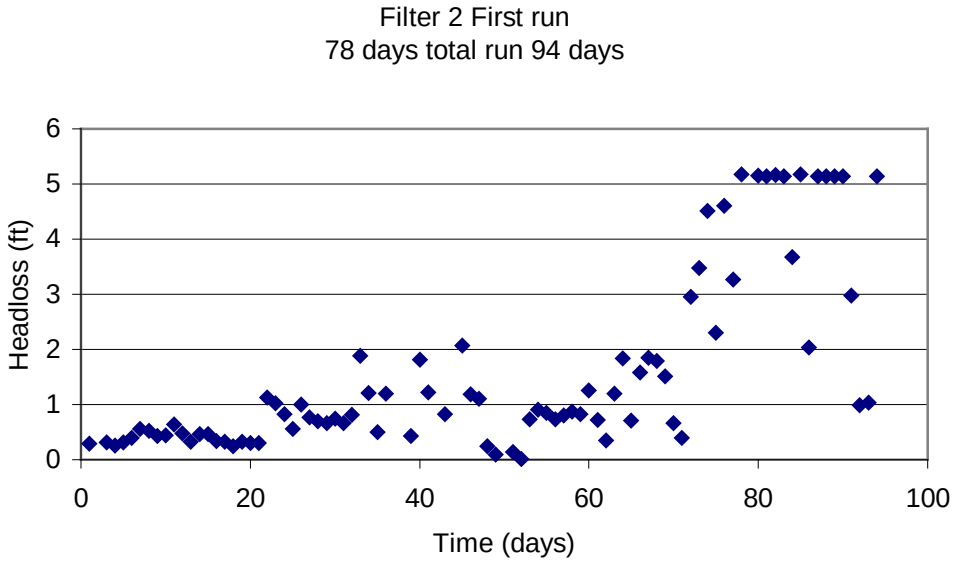


Figure 16. Filter 2 first run length.

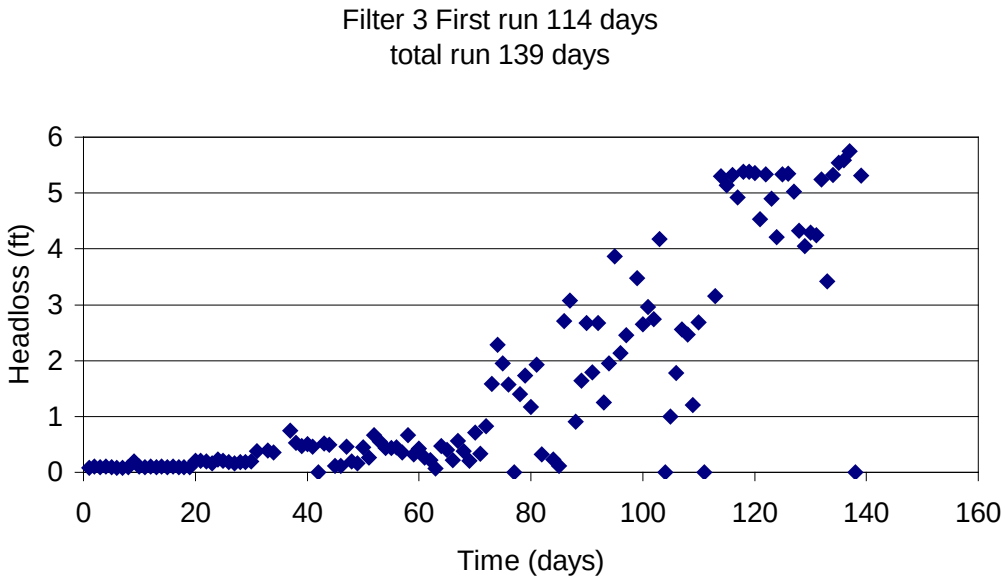


Figure 17. Filter 3 first run length.

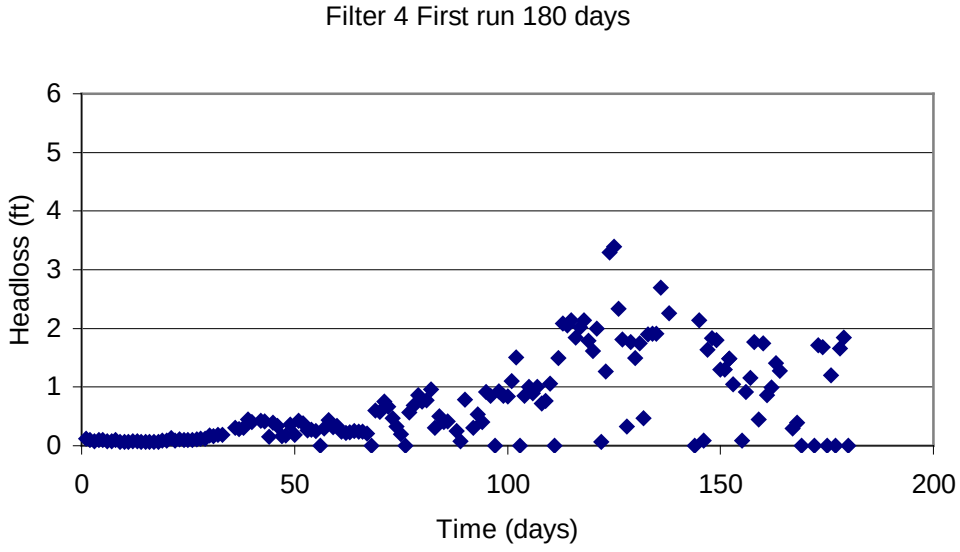


Figure 18. Filter 4 first run length.

4.1.2 The Potential Effects of Uniformity Coefficients on Headloss

The potential significances of the disparity in UC values between the imported sand media and the locally manufactured sand media is evident in: 1) the length of filter run time exhibited by the filters, and 2) differences in where headloss developed. The longer initial run times for Filters 3 and 4 are primarily attributed to the large UC of the local media. These findings are consistent with other studies in which filtration media with large uniformity coefficients (4.29) had run lengths significantly greater than filters with media having UC's of 2.24 and 2.85 (Di Bernardo and Rivera, 1996). The manufactured basalt sand has sharp edges and angles and it appears that the geometry created voids not readily filled by smaller basalt particles. This would allow more sediment carried by the inflow to be deposited throughout the pore spaces deeper in the filter, thereby slowing sediment build-up on the top of the sand bed, similar to what Di Bernardo and Rivera (1996) reported.

This hypothesis is supported by the fact that, excluding the first run for Filter 1, the majority of headloss in Filters 1 and 2 occurred in a different region of the filter bed than was observed in Filters 3 and 4. Filters 1 and 2 exhibited the greatest headloss across the upper section of the filter bed (identified as middle to top), with relatively little headloss occurring across the body of the filter bed (identified as gravel to middle) (Figures 19a-h, Figures 20a-d).

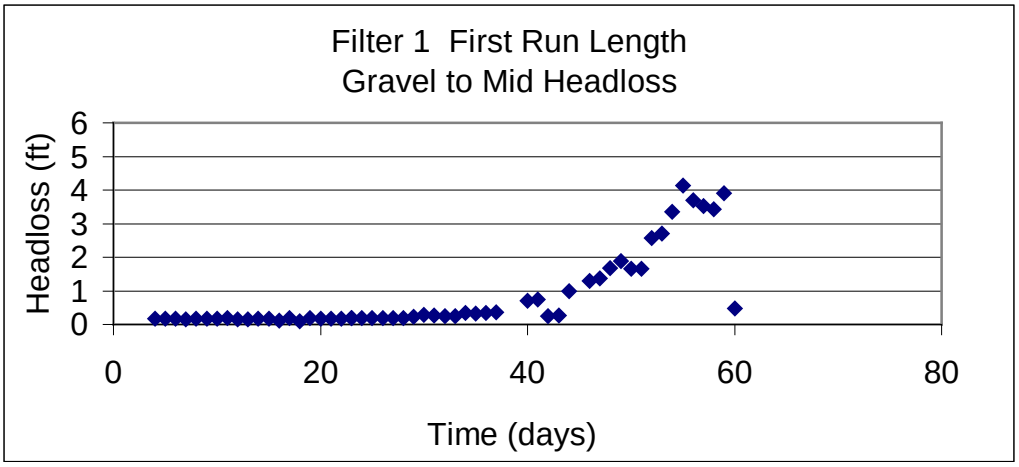


Figure 19a. Filter 1 headloss between gravel and middle piezometer first run.

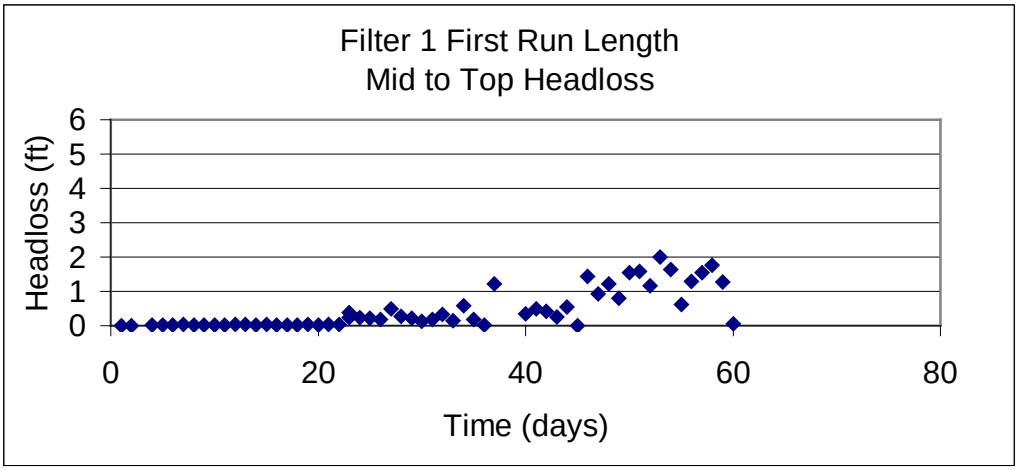


Figure 19b. Filter 1 headloss between middle and top piezometer first run.

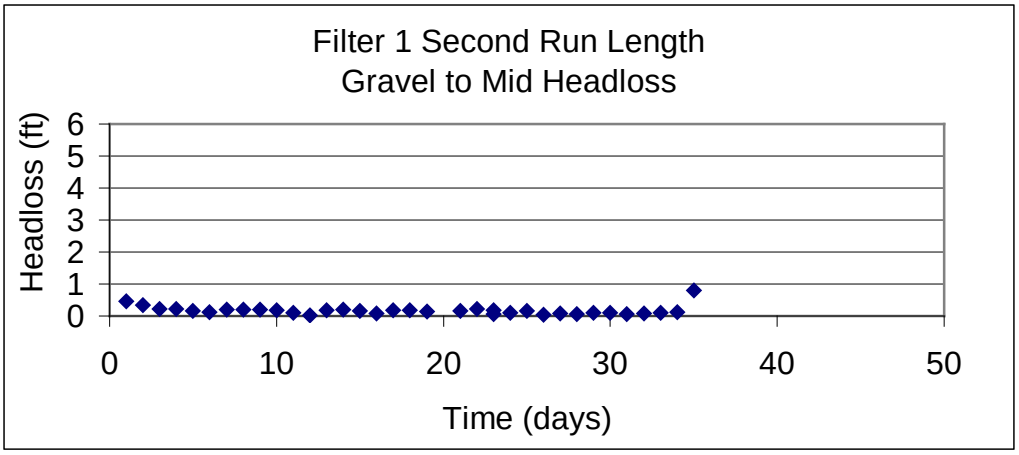


Figure 19c. Filter 1 headloss between gravel and mid piezometer second run.

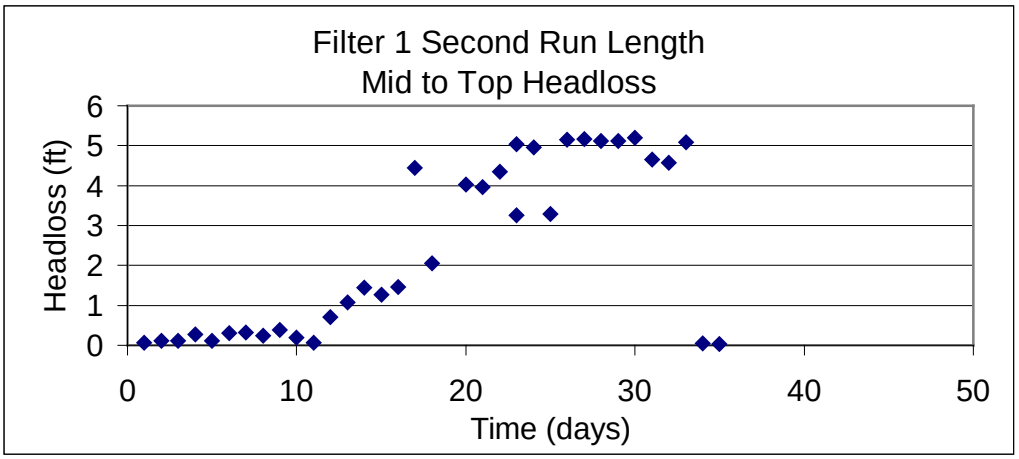


Figure 19d. Filter 1 headloss between middle and top piezometer second run.

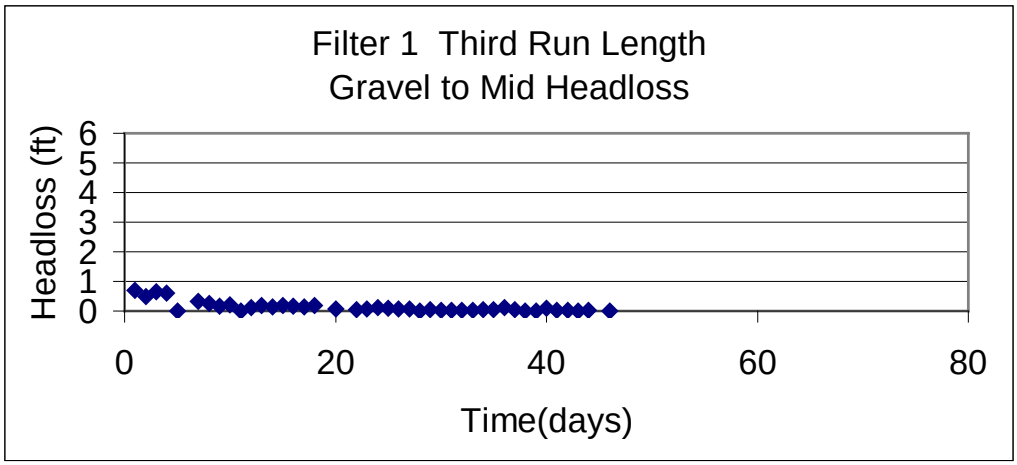


Figure 19e. Filter 1 headloss between gravel and middle piezometer third run.

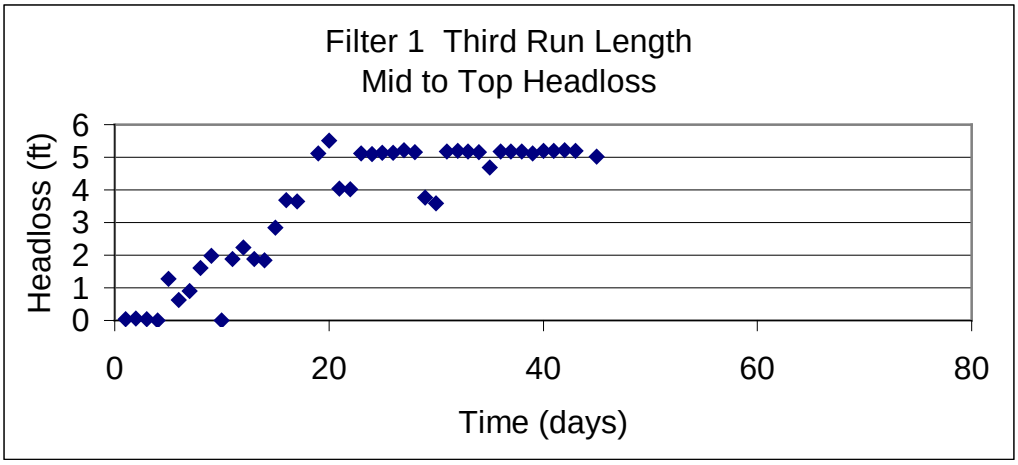


Figure 19f. Filter 1 headloss between middle and top piezometer third run.

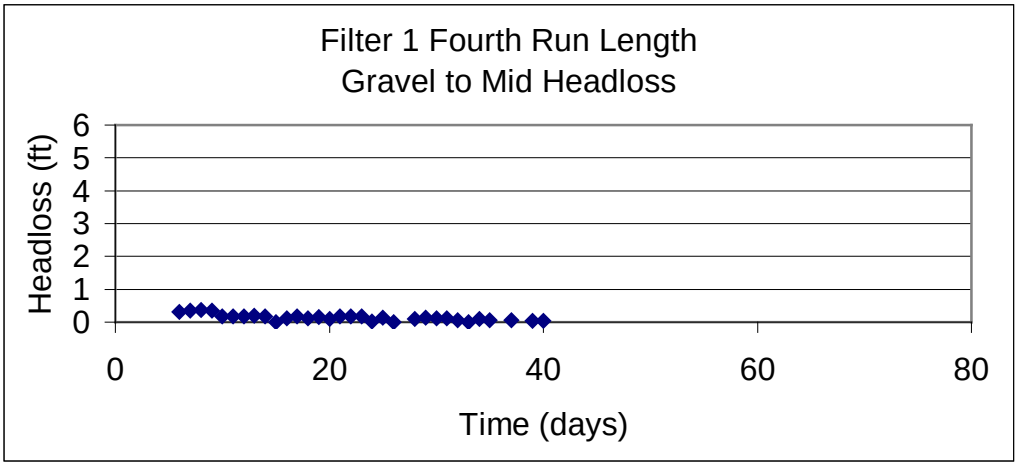


Figure 19g. Filter 1 headloss between gravel and middle piezometer fourth run.

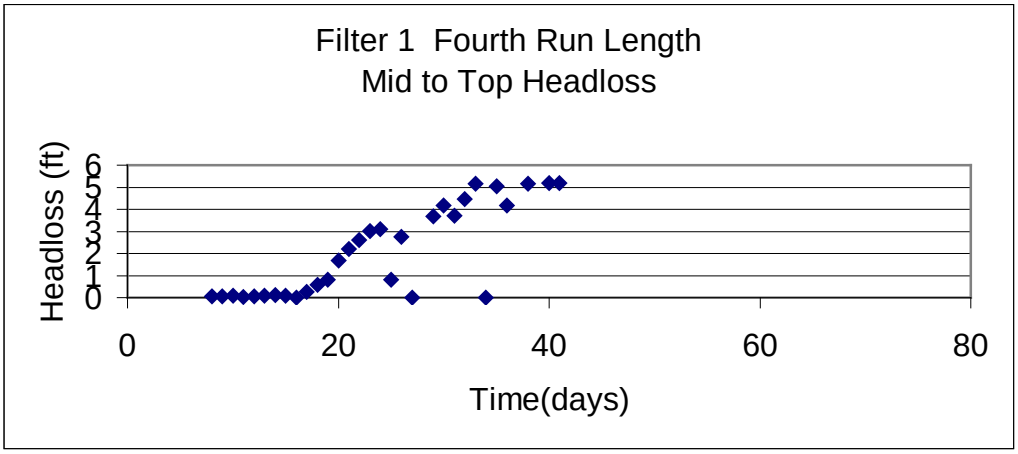


Figure19h. Filter 1 headloss between middle and top piezometer fourth run.

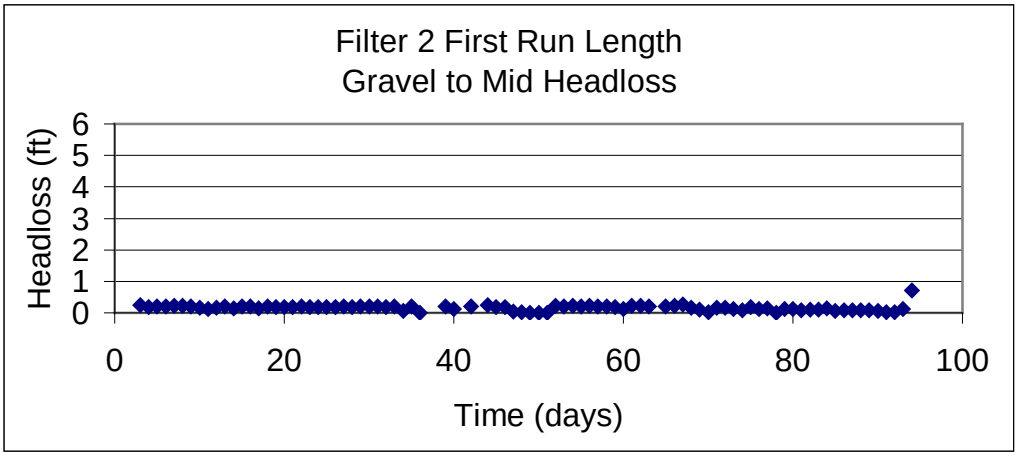


Figure 20a. Filter 2 headloss between gravel and middle piezometer first run.

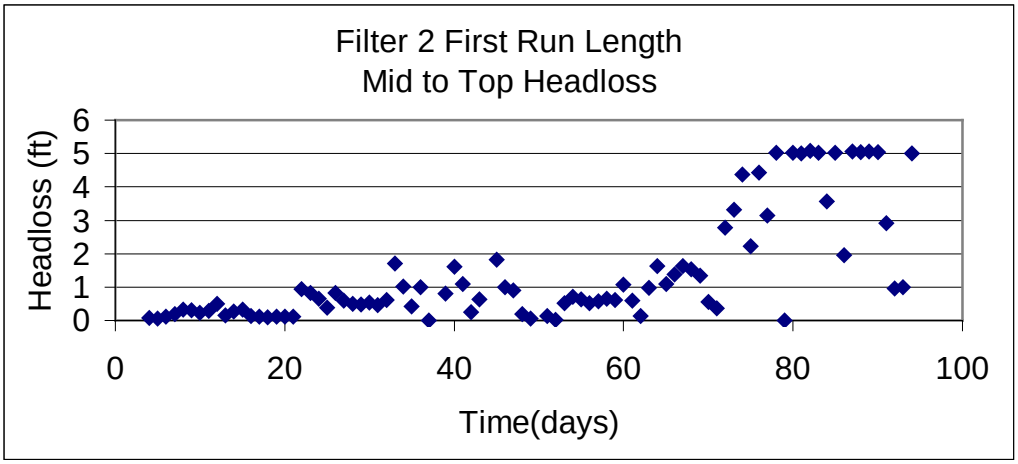


Figure 20b. Filter 2 headloss between middle and top piezometer first run.

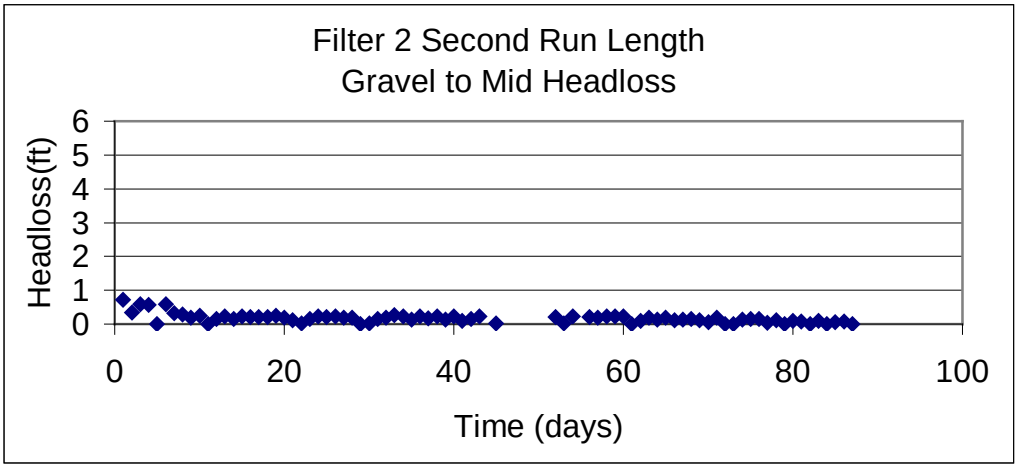


Figure 20c. Filter 2 headloss between gravel and middle piezometer second run.

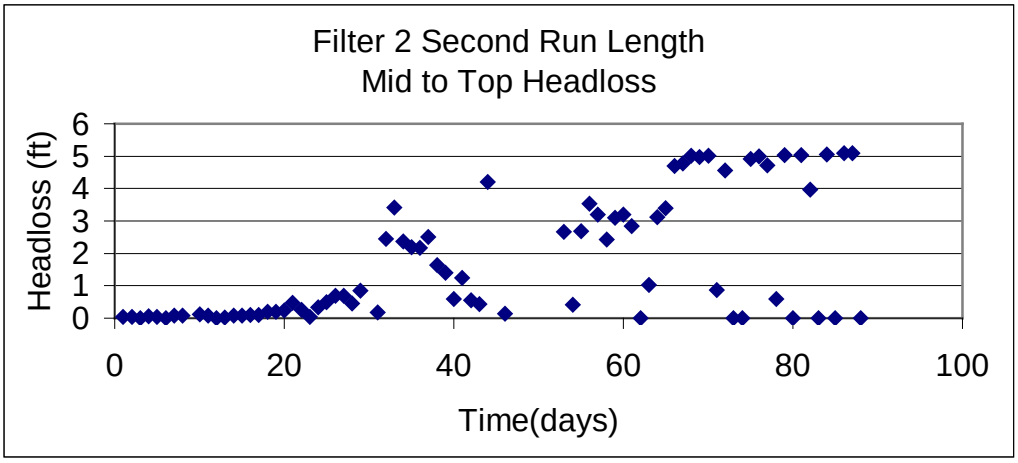


Figure 20d. Filter 2 headloss between middle and top piezometer second run.

Conversely, headloss across the upper portion of the filter bed was minimal in Filters 3 and 4. In both of these filters, headloss across the gravel-mid region was the major contributor to total headloss as shown in Figures 21a-d for Filter 3 and Figures 22a-b for Filter 4.

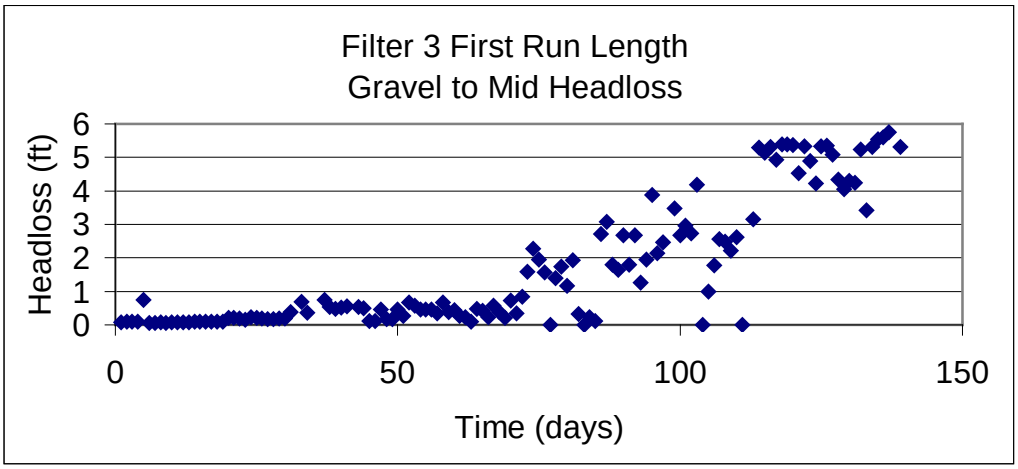


Figure 21a. Filter 3 headloss between gravel and middle piezometer first run.

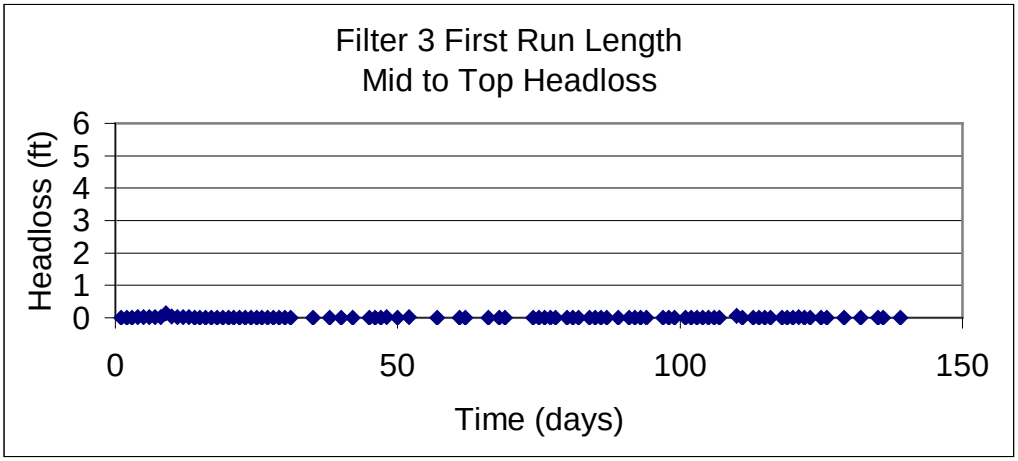


Figure 21b. Filter 3 headloss between middle and top piezometer first run.

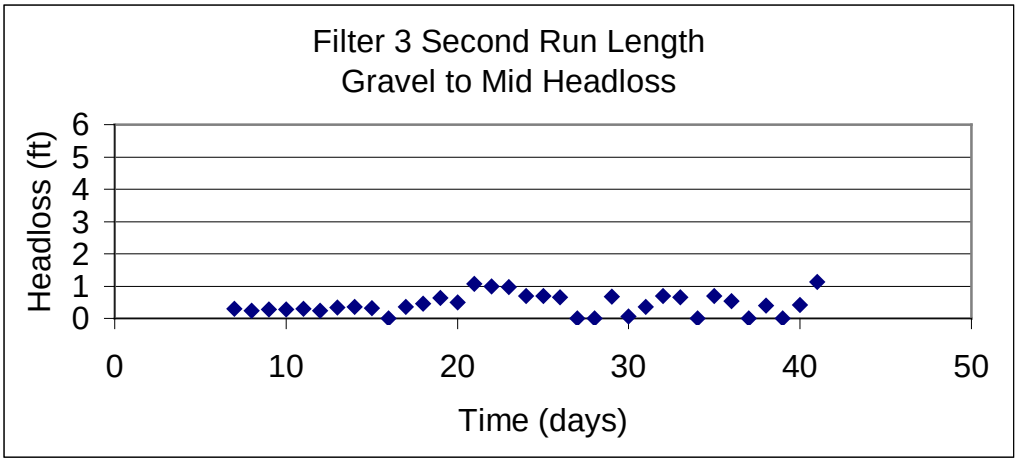


Figure 21c. Filter 3 headloss between gravel and middle piezometer second run.

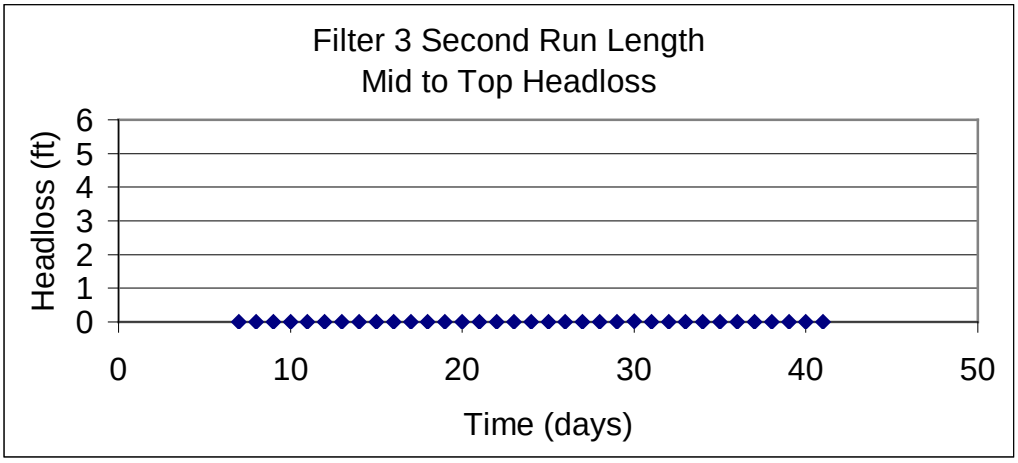


Figure 21d. Filter 3 headloss between middle and top piezometer second run.

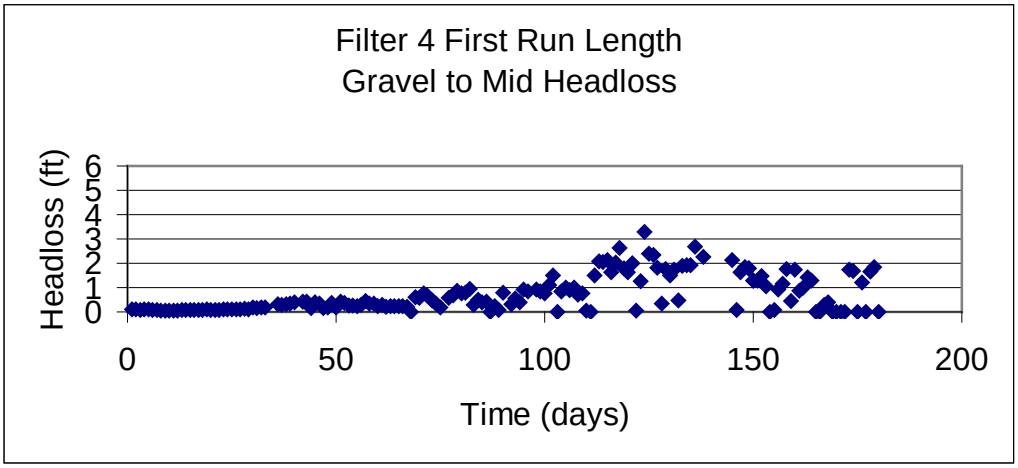


Figure 22a. Filter 4 headloss between gravel and middle piezometer first run.

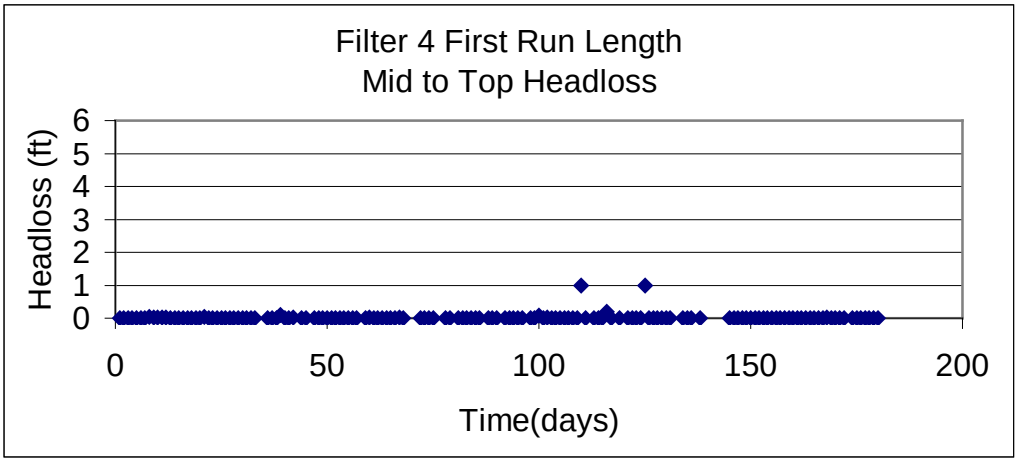


Figure 22b. Filter 4 headloss between middle and top piezometer first run.

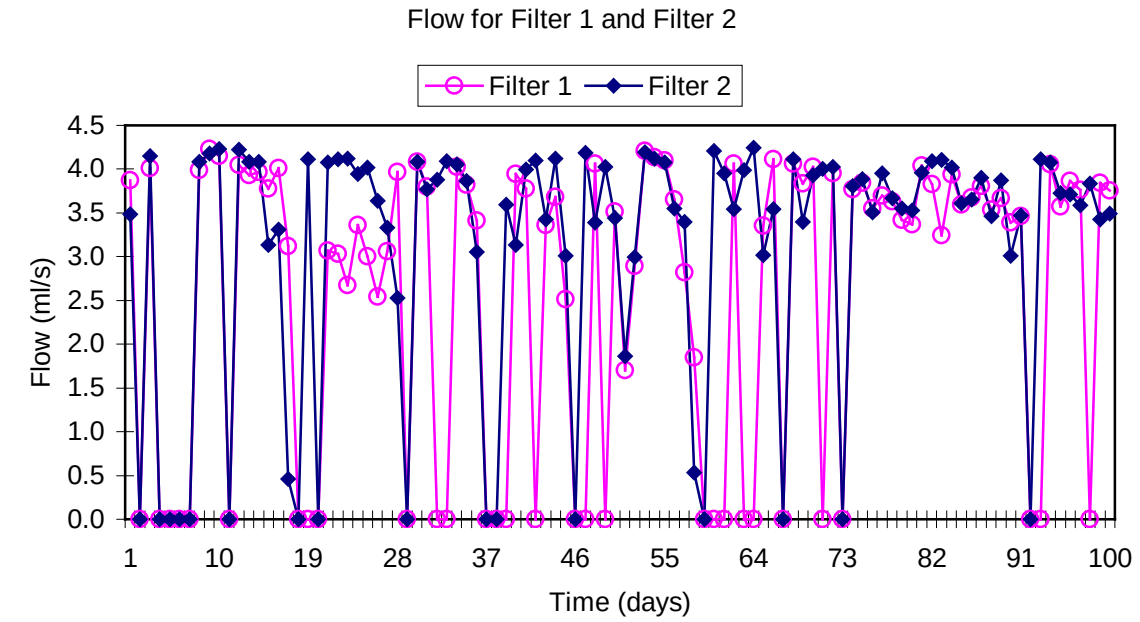
Di Bernardo and Rivera (1996) also observed that filters with higher uniformity coefficients experienced increased retention of smaller particles in the *schmutzdecke* as the run continued. Furthermore, the *schmutzdecke* was thicker and extended deeper into the upper area of the sand bed. It is possible that similar processes are driving *schmutzdecke* behavior in Filters 3 and 4.

4.1.3 The Effect of Flow Fluctuation

All filters experienced highly inconsistent flow that consequently influenced headloss development rate and hence, filter run length Run lengths are not considered to be indicative of steady hydraulic loading of 0.2 m/hr, given the frequency of flow interruption.

Due to frequent, high intensity rain events, often of long duration, stream flow rises dramatically and the amount of organic/inorganic material transported in the streams also increases. Turbidity levels commonly increase by one to two orders of magnitude (<10 NTU to $>250^{+}$ NTU personal observation June 2000) during such events.

Particulate matter suspended in the stream during such events entered the flow meters, thereby contributing to partial or total blockage of the flow meter orifices. Furthermore, the intake structure for the Tofol dam frequently became blocked during high intensity, prolonged rain events, causing either a complete stoppage of flow to the municipal system or dramatic drops in system pressure. The 100-day flow records (Figure 23) are indicative of flow fluctuations that occurred throughout the study (see Appendix C for complete flow records).



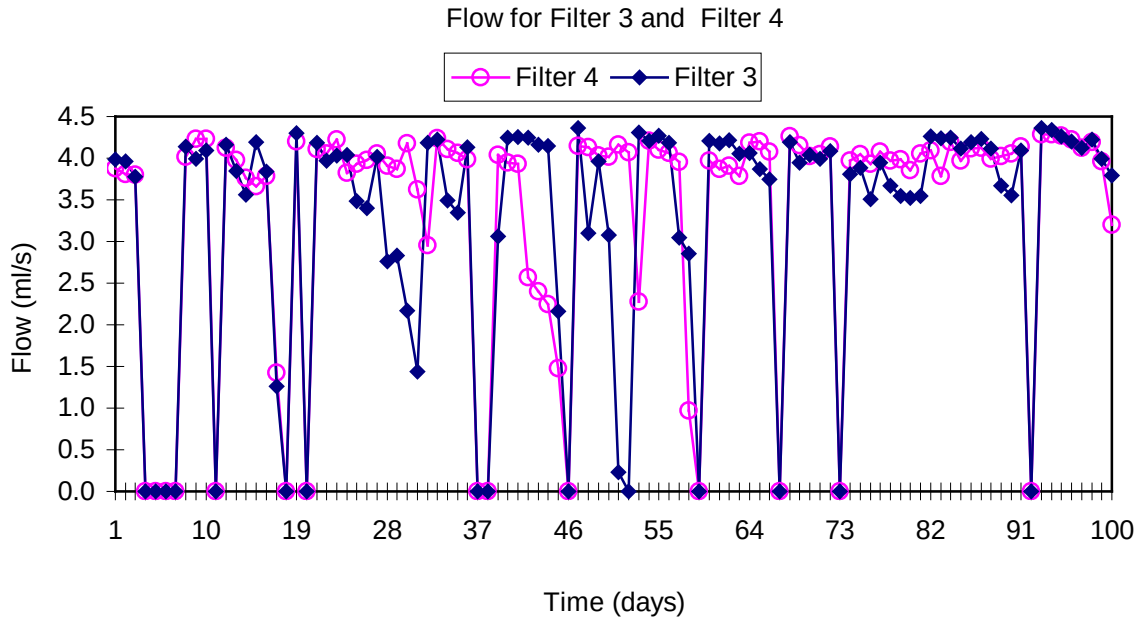


Figure 23. Flow comparison for July 7 through October

The importance of flow irregularities should not be understated, due to the potentially significant impact on headloss development rate and therefore, filter run length. To more accurately assess filter run length at a specified hydraulic loading rate, flow must be consistent at all times.

4.1.4 Summary of Run Length

Flow fluctuation and differences in UC values influence the rate of headloss development in all filters. The relatively long first run length for all filters, when compared to their succeeding run lengths, appears to be a function of sand media settling time, with an associated decrease in voids.

Filter 2 received unwashed, dry imported sand at start-up and the run lengths suggest initially less packing than Filter 1 (that received pre-washed imported sand).

The extended first run lengths of Filter 3 and Filter 4, combined with a large UC and identical patterns of headloss behavior, suggest that greater amounts of sediment deposition occurred deeper in the sand bed of these filters. Flow problems (air binding) associated with the line supplying Filter 4 are also considered to have contributed to extending filter run length for that filter. In future studies care should be taken to maintain a constant downhill slope from the discharge elbow below the flow meter to the elbow connecting to the vertical discharge pipe. For this reason, Filter 3 is considered a better indicator of local media performance regarding run length.

Filter 3 exhibited a long first run, followed by a decreased second run length. The extended first run length is attributed to deeper deposition of sediment in the sand bed, as evidenced by the development of significant headloss across the gravel to mid sections. The

second run length (42 + days) suggests that pore space through time was decreasing within the filter, however, this is not readily apparent in the graphs indicting the location of majority headloss (Figure 21c and 21d). Headloss data was not recorded following the week of December 03, but upon arrival on Kosrae December 18, it was noted that Filter 3 had a headloss of 4 feet, a nearly 3-foot increase in two weeks time. Given the location of headloss development in Filter 3 (Figure 21c and 21d), it would indicate that additional pore space was filled during the period between December 3 and December 18. This suggests that filters containing local media may require deep bed cleaning to restore run length (back-flushing or complete removal and washing of media) if the UC is not lowered. Further study regarding the behavior of succeeding runs is needed to accurately assess this hypothesis.

Succeeding runs for Filter 1 suggest an average run length of 26 days. Since the majority of headloss is across the top of the filter bed, deep bed cleaning does not appear necessary for the imported sand media filters. To more accurately determine if this is the case, testing of filters containing imported media should be continued. By providing steady flow at all times, through several succeeding runs, the initial and succeeding filter run lengths with an HLR of 0.2 m/hr could be identified.

4.2 Determination of Filter Bed Maturation

4.2.1 Results of First Spike Tests

A summary of the average concentration of headwater and tailwater fecal coliforms obtained during the first spiking tests is presented in Table 11 (see Appendix D for first spike data).

Table 11

Headwater MPN's and Tailwater MPN's for all Filters During First Spike Test

	Average MPN's of Fecal Coliforms per 100ml sample							
	June 03		June 04		June 05		June 06	
	Filter 1		Filter 2		Filter3		Filter 4	
ET (h:min)	Head	Tail	Head	Tail	Head	Tail	Head	Tail
0:00	27,197	103	4,132	76	72,050	34	106,985	63
0:30	19,264	33	1,458	83	60,395	11	84,007	105*
1:00	28,506	35	987	36	41,995	14	57,300	40
2:00	16,164	35	4,943	15	41,657	3	35,220	26
4:00	8,066	627	2,484	6	21,160	9	11,074	13
6:00	7,230	269	1,650	4	8,931	25	8,474	8
8:00	5,689	157	730	139	7,561	43	3,878	42
Max MPN	28, 506	627	4,943	139	72,050	43	106,985	105*,42

* represents highest average MPN of bacteria arriving earlier than expected

The concentration of fecal coliforms in the headwater and tailwater were plotted as a time series for each filter as shown in Figures 24-27 (headwater/tailwater counts through time).

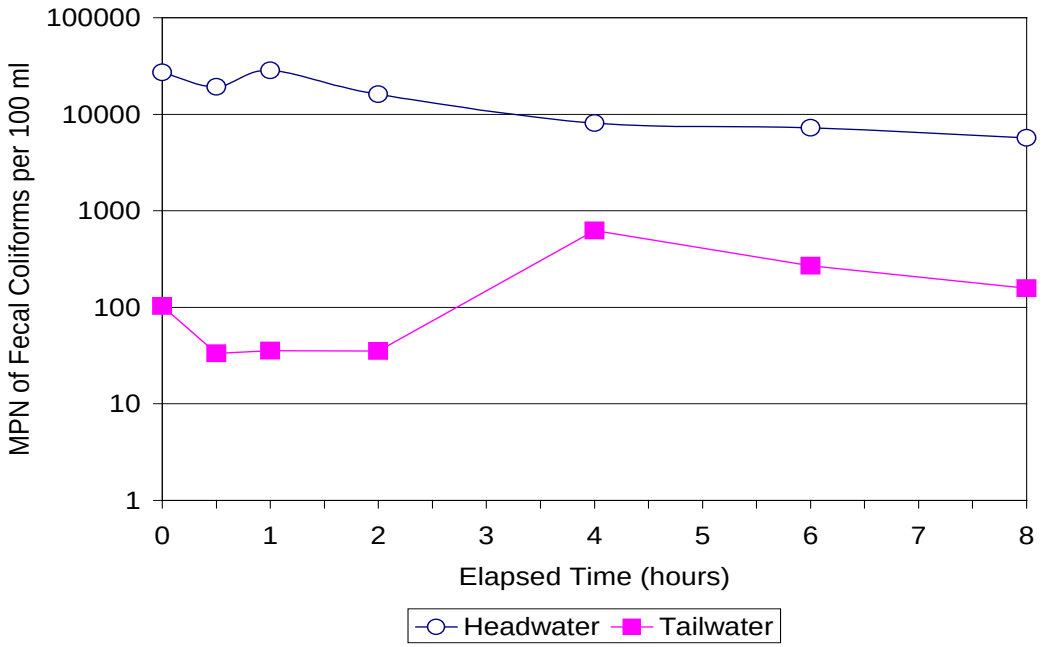


Figure 24. Filter 1: Average MPN's of fecal coliforms for first spike test.

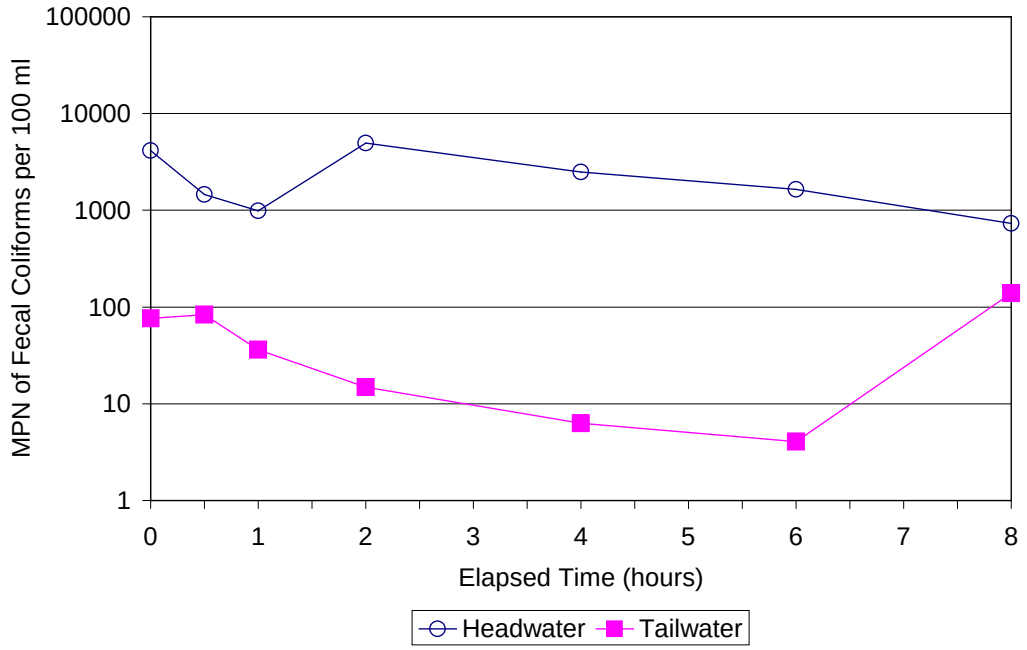


Figure 25. Filter 2: Average MPN's of fecal coliforms for first spike test.

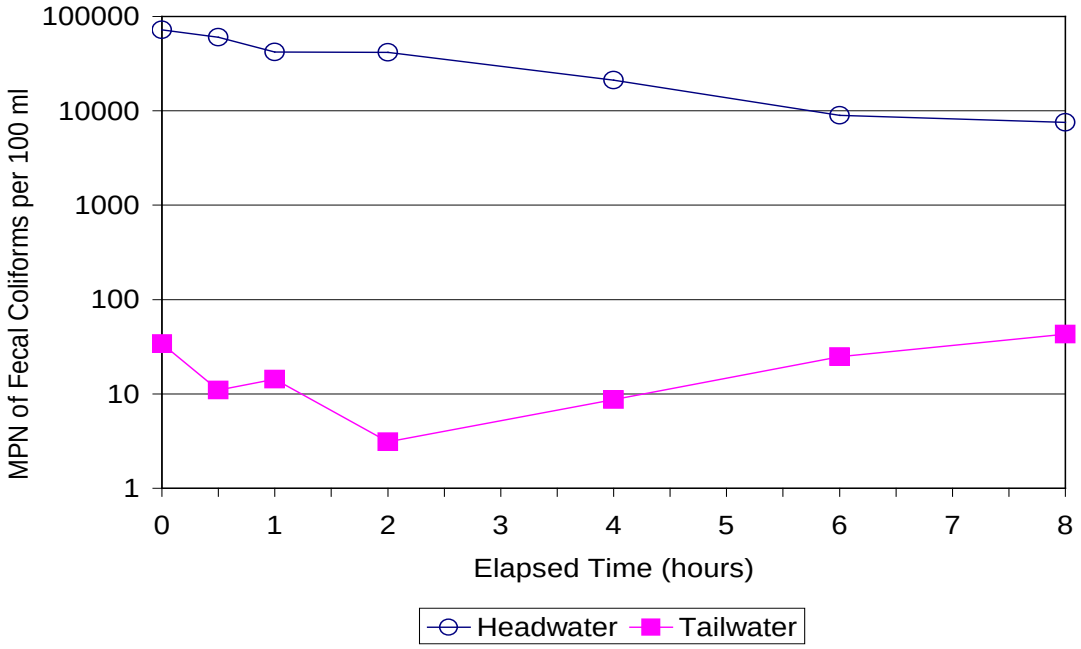


Figure 26. Filter 3: Average MPN's of fecal coliforms for first spike test.

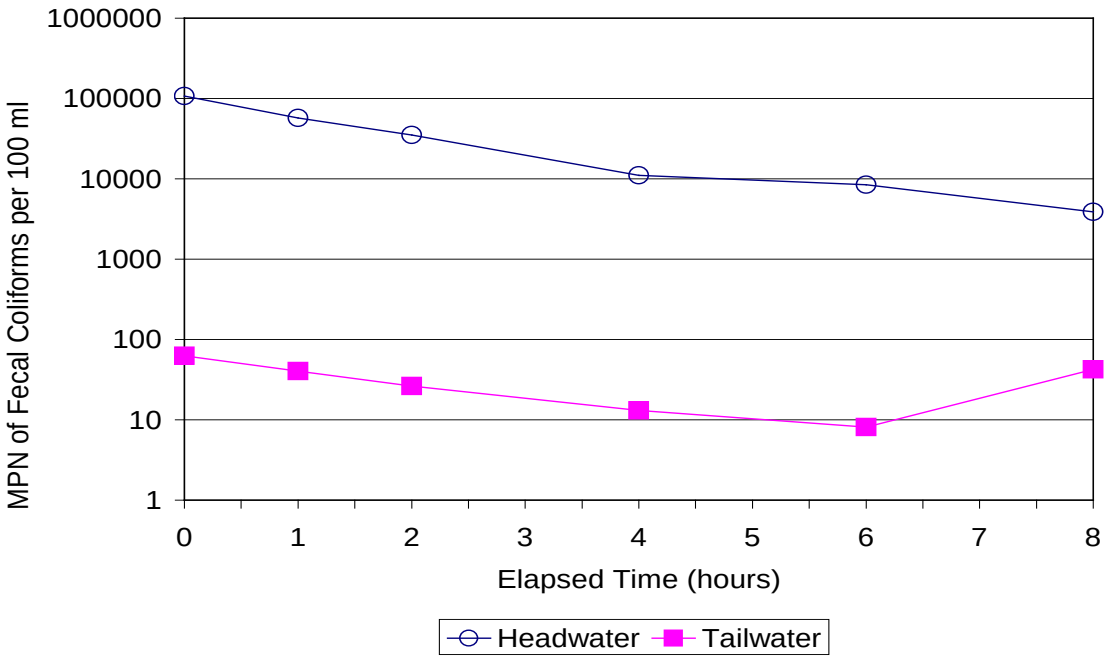


Figure 27. Filter 4: Average MPN's of fecal coliforms for first spike test

The variance in initial headwater fecal coliform concentrations was attributed to four factors: 1) natural variability in viable fecal coliforms per bacterial pellet, 2) daily variability in fecal coliform concentrations of ambient source water, 3) variation in initial headwater volumes 4) intentional increases in the concentration of fecal coliform stock solution added to the headwater.

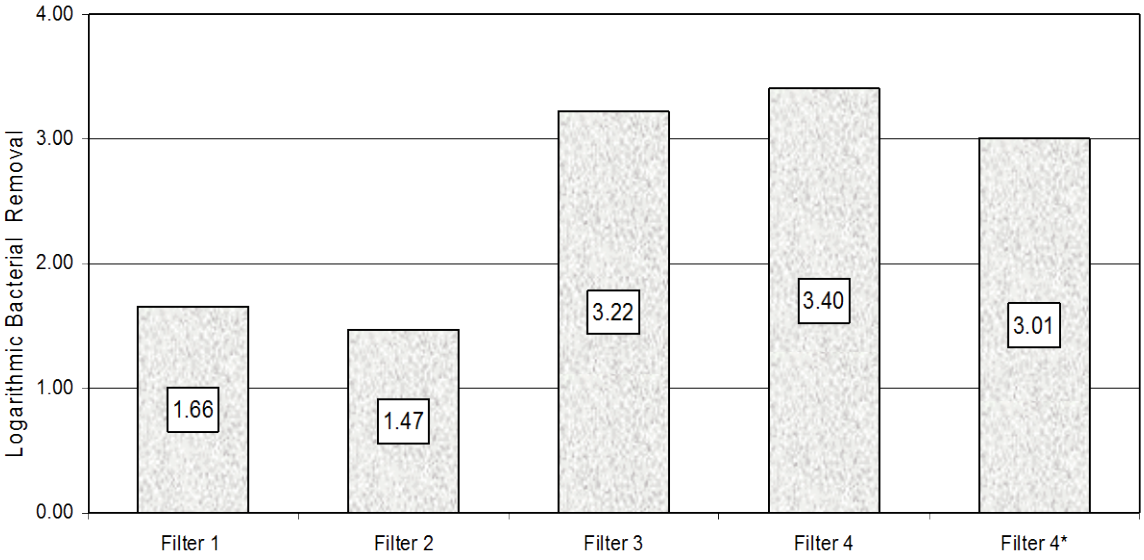
The uncertainty regarding the number of viable fecal coliforms in each pellet presented a challenge for estimating the correct concentration of bacterial stock solution needed for spiking. Preliminary testing at WERI indicated that a sufficient number (6 orders of magnitude) of live fecal coliforms were obtainable in a single pellet dissolved in 100 ml of distilled water. However, the same series of tests indicated differences of one order-of-magnitude between pellets. These differences required consideration, since the requisite number of dilutions depended upon the expected maximum bacterial concentrations in the headwater. Furthermore, the total number of dilutions that could be performed was limited by the amount of space available for incubation of the samples.

Results from the spike of Filter 1 indicated an initial concentration of fecal coliforms of less than 30,000 per 100 ml. After examining the results of the first spike, it was evident that greater amounts of bacteria could be added to the remaining filters, without increasing dilutions and exceeding the space limitations of the incubator. Intentional efforts were made in succeeding spikes to increase the maximum headwater concentration of fecal coliforms by adding additional bacterial stock solution to the headwaters to ensure that quantitative analysis was strengthened, however, this measure was unsuccessful in the spike of Filter 2.

Filter 2 received a greater amount of stock solution (1.5 pellets reconstituted) than Filter 1, however, the headwater concentrations were the lowest of all four filters. This was attributed

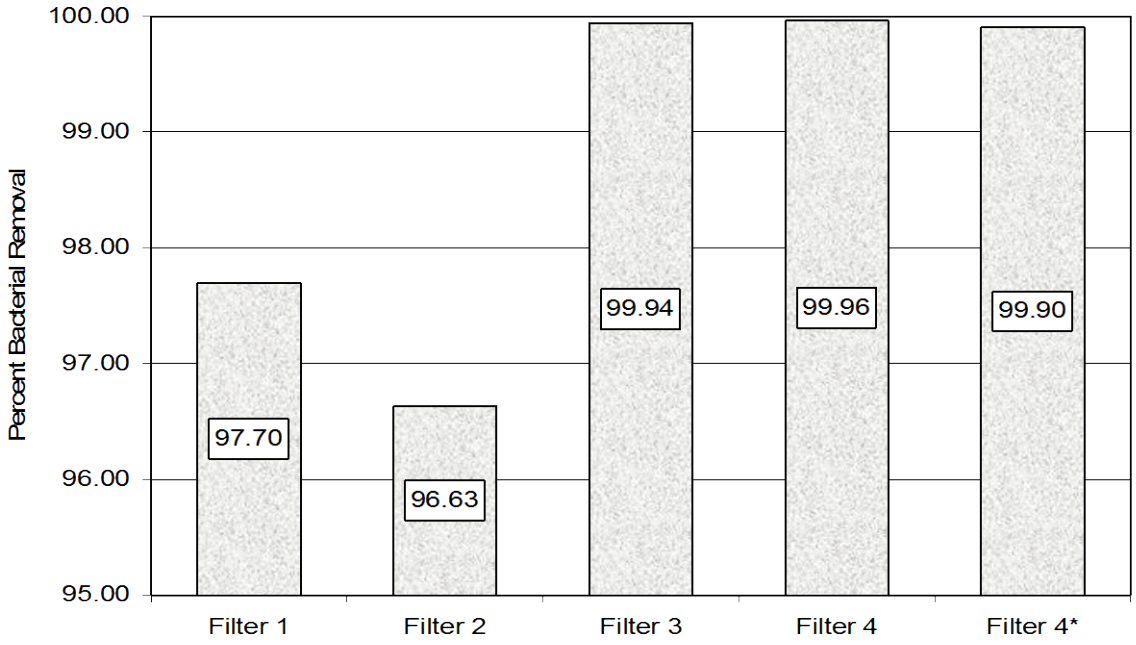
primarily to the variability in pellets. To ensure a higher number of bacteria were introduced in the remaining spike tests, 2 pellets were added to the headwater of Filters 3 and 4. This adjustment is reflected in the higher counts for Filter 3 and Filter 4, as shown in Table 15.

Logarithmic bacterial removal values (Figure 28) and percent bacterial removals (Figure 29) calculated after the first spiking tests indicate two such calculations for Filter 4 (4 and 4*). This was done because a high count of tailwater fecal coliforms was evident in the water sample drawn at ET 0:30 and in the tailwater sample drawn at ET 8:00. This last sample was drawn at a time closer to the theoretical arrival time of 6.1 hours (6.1hrs = HLR (0.21m/hr)× bed depth (1.2 m)) for spike fecal coliforms to reach the tailwater. It is possible, however, that the high MPN value obtained at the 30-minute sample time was due to a preferential flow path within the filtration media or some other factor (contamination of filtration media at start-up). Therefore, logarithmic bacterial removal and percent bacterial removal calculations were made for each value.



* represents highest average MPN of bacteria at ET 0:30

Figure 28. Logarithmic bacterial removals for first spike tests.



* represents highest average MPN of bacteria at ET 0:30

Figure 29. Percent bacterial removals for first spike tests.

The logarithmic bacterial removals and percent bacterial removals obtained in the first spiking series were used to establish baseline bacterial removal efficiency. However, the sampling period following the headwater spikes may have been inadequate for detecting the highest concentrations of tailwater fecal coliforms. This is evident in the spike time series (Figures 25, 26, and 27) for Filters 2, 3 and 4 that show tailwater fecal coliform concentration increasing when the 8:00 hour sample was drawn. The significance this may have had on establishing an accurate baseline for filter performance, especially the higher removal values for Filter 3 and Filter 4 (Figure 28 and Figure 29), is addressed in the discussion of the second spike test. To prevent similar problems in the second and third spike tests, the post-spike sampling period was extended to 16-hours.

4.2.2 Dissolved Oxygen as an Indicator of Filter Maturation Level

Following the first spike, dissolved oxygen (DO) levels were measured to assist in determining filter maturity. Differences of 2 mg/L to 4 mg/L of DO between headwater and tailwater have indicated bed maturation in some studies (Hendricks, 1991). Average differences in daily DO measurements for the first month of operation are shown in Table 12. Daily differences in DO arranged in ascending order, to emphasize the range of differences for each filter, are shown in Table 13 (see Appendix G for daily DO measurements in June).

Filters 3 and 4 never exceeded 2 mg/L difference in DO, while Filters 1 and 2 had only 2 readings and 1 reading respectively that equaled or exceeded 2 mg/L. The readings that exceeded 2 mg/L occurred in mid-June and were associated with days that flow to the filters had been low due to pressure drops in the municipal water line. Such flow reduction decreases the HLR of the filters, consequently lowering the rate that water moves through the filter. This would have allowed a longer retention of water in the filter bed, where decreases in DO would be expected due to both temperature-induced release of DO and oxygen consumption by aerobic organisms associated with the *schmutzdecke* and biofilms.

Table 12

Average Concentration of DO in Headwater and Tailwater of All Filters First Month

Average Dissolved Oxygen for First Month of Study (mg/L)		
Filter	Head	Tail
1	5.00	4.13
2	5.16	4.05
3	5.08	4.34
4	5.12	4.32

Table 13

Ascending Order of Differences Between Headwater and Tailwater DO for the First Month of Pilot Plant Operation

Differences in Dissolved Oxygen Concentration Between Headwater and Tailwater			
Filter 1 (mg/L)	Filter 2 (mg/L)	Filter 3 (mg/L)	Filter 4 (mg/L)
0.78	0.84	0.45	0.42
0.85	0.86	0.54	0.44
0.88	0.91	0.58	0.59
0.89	0.92	0.61	0.78
0.93	0.95	0.70	0.83
0.98	0.98	0.72	0.84
1.00	1.01	0.72	0.87
1.00	1.05	0.72	0.87
1.00	1.05	0.75	0.88
1.05	1.10	0.77	0.89
1.07	1.12	0.81	0.95
1.10	1.15	0.85	0.95
1.10	1.16	0.86	0.98
1.17	1.22	0.87	1.00
1.20	1.26	0.89	1.04
1.20	1.27	0.89	1.06
1.22	1.32	0.96	1.07
1.23	1.32	0.99	1.08
1.24	1.33	1.05	1.10
1.37	1.39	1.06	1.11
1.39	1.39	1.07	1.12
1.50	1.48	1.11	1.14
1.51	1.48	1.17	1.15
1.62	1.89	1.17	1.16
1.8	1.92	1.35	1.27
2.58	2.58	1.54	1.35
2.85	3.10		
Average			
1.28	1.34	0.86	0.92

Low DO differences suggest that any or all of the following could contribute to such values: 1) the filters had not ripened as evidenced by acquiring an aerobic, biologically active biofilm around the sand particles and an aerobically active *schmutzdecke* after thirty days, 2) high temperature (28° C) prevented the retention of oxygen in the water, or 3) that the composition of the *schmutzdecke* was primarily inorganic material (sediment).

To gain a better perspective on the degree of bed maturation, a second spike test was conducted the first week of July.

4.2.3 Results of Second Spike Tests

The average MPN's of headwater and tailwater fecal coliform concentrations obtained from the second spike tests for each filter are shown in Table 14 (see Appendix I for all second spike results).

Table 14

Headwater MPN's and Tailwater MPN's for all Filters During Second Spike Tests

	Average MPN's of Fecal Coliforms per 100ml							
	July 03		July 03		July 04		July 04	
ET	Filter 1		Filter 2		Filter 3		Filter 4	
(hr)	Head	Tail	Head	Tail	Head	Tail	Head	Tail
0:00	63,146	19	62,278	1	47,605	8	114,002	7
2:00	30,502	4	26,813	2	24,707	9	26,325	6
4:00	6,625	50	9,703	10	6,483	92	2,905	214
6:00	4,477	220	5,075	31	5,906	218	8,649	592
9:00	4,373	187	4,215	59	7,634	205	9,123	260
12:00	3,195	74	2,931	65	4,602	165	4,911	543
16:00	1,562	40	1,772	14	2,556	142	1,649	226
Max. MPN	63,146	220	62,278	65	47,605	218	114,002	592

All filters except Filter 4 had maximum headwater concentrations exceeding those of the first spike test (see Table 11). Headwater volumes during the second spike ranged from 24.5 L - 42.3 L due to differences in headloss development, therefore, the amount of bacteria added to the filters was adjusted (Table 15). The spike solution volumes listed for Filters 1 and 2 reflect 1 pellet dissolved in 100 ml distilled water combined with 40 ml and 68 ml respectively of stock solution (1 pellet dissolved / 100 ml distilled water) that remained from the first spike tests and was stored at 12° C in the lab refrigerator.

Filters 3 and 4 received 150 ml each of bacterial stock solution. This solution was made from three pellets, each reconstituted in 100 ml of distilled water. Each filter received roughly 1.5 pellets by using 100 ml of individual solution and half of the third 100 ml solution.

Table 15

Headwater Volumes and Respective Spike Solution Volumes for Second Spike

Filter	Headwater Volume (L)	Spike Solution Volume (ml)
1	35.3	140
2	42.3	168
3	31.1	150
4	24.5	150

The results of the second spike differ from the initial spike tests in that clearly defined peaks in maximum concentrations of tailwater fecal coliforms are evident as shown in Figures 30 through 33. It appears that a closer capture of the maximum concentrations of fecal coliforms passing through the filters was obtained in this spike series. The decline in headwater fecal coliforms through time is also smoother than that of the first spike. This is attributed to settling of the filter media and development of the *schmutzdecke*, in addition to improved sample preparation.

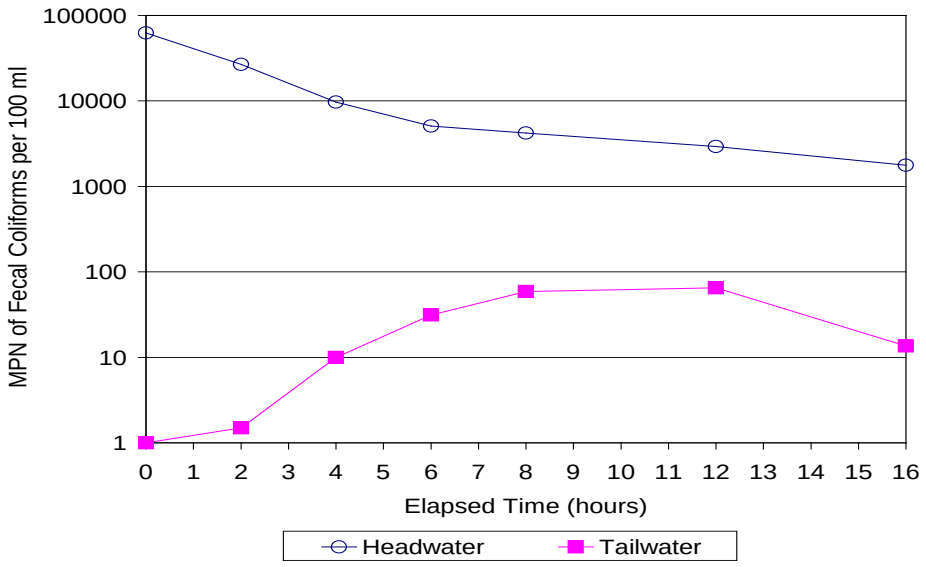


Figure 30. Filter 1: Average MPN's of fecal coliforms for second spike test.

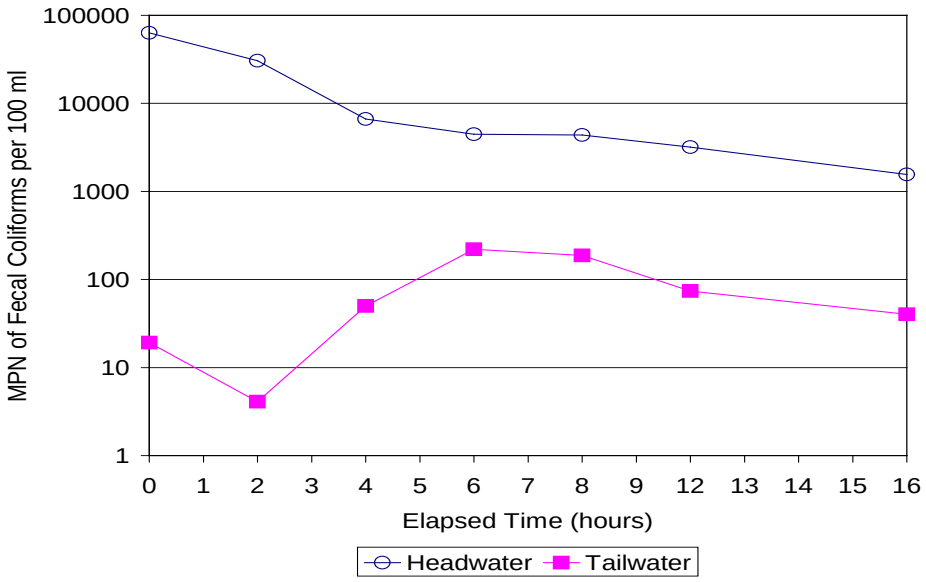


Figure 31. Filter 2: Average MPN's of fecal coliforms for second spike test.

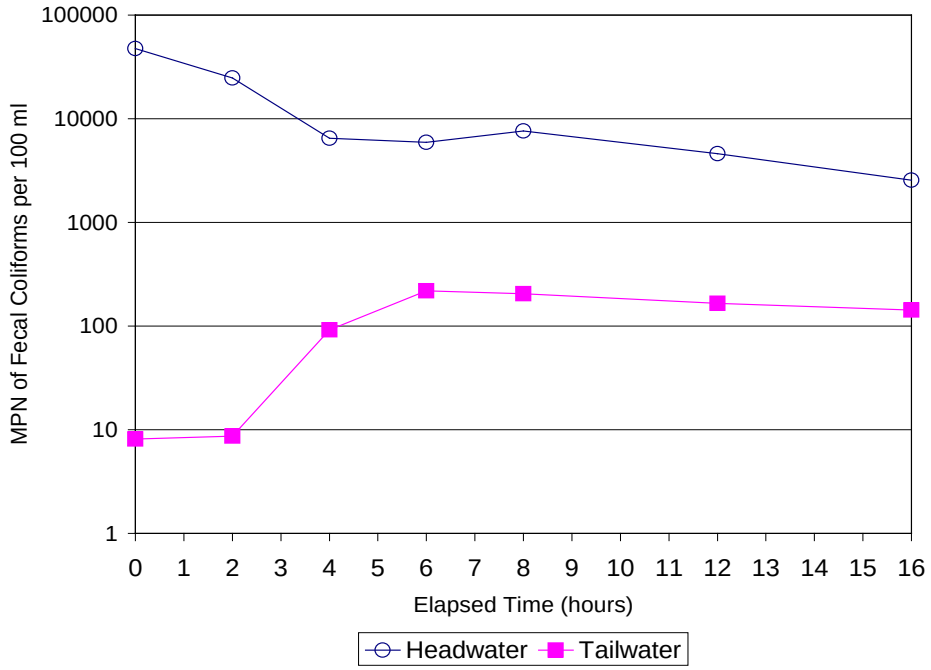


Figure 32. Filter 3: Average MPN's of fecal coliforms for second spike test.

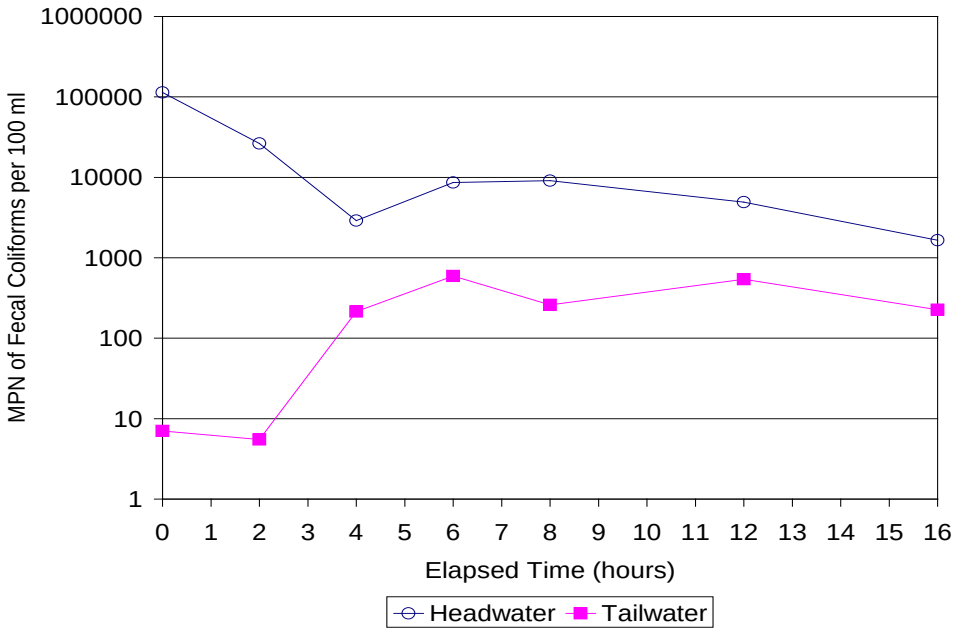


Figure 33. Filter 4: Average MPN's of fecal coliforms for second spike test.

Logarithmic bacterial removal rates ranged from 2.28 - 2.98 (Figure 34); percent bacterial removal efficiency was above 99% in all filters (Figure 35).

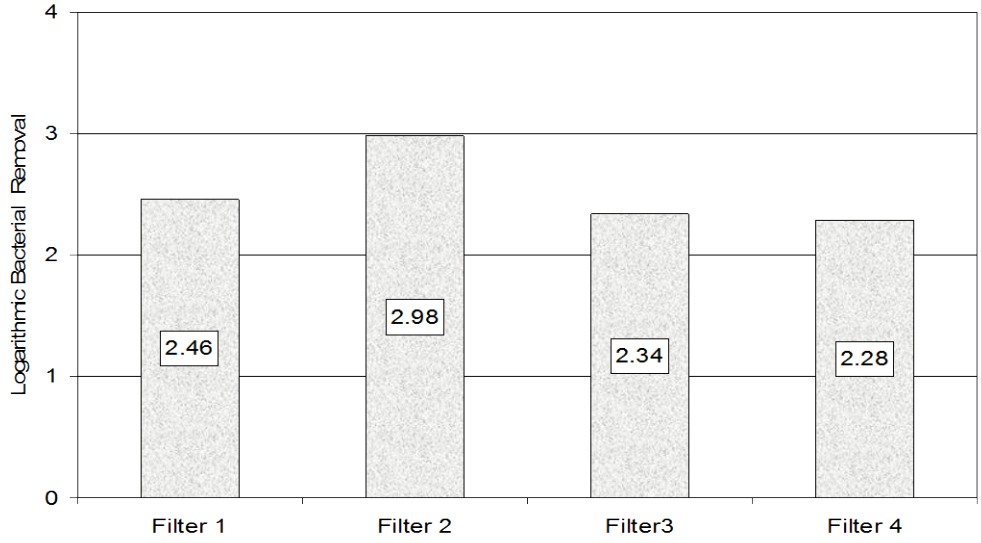


Figure 34. Logarithmic bacterial removals for second spike tests.

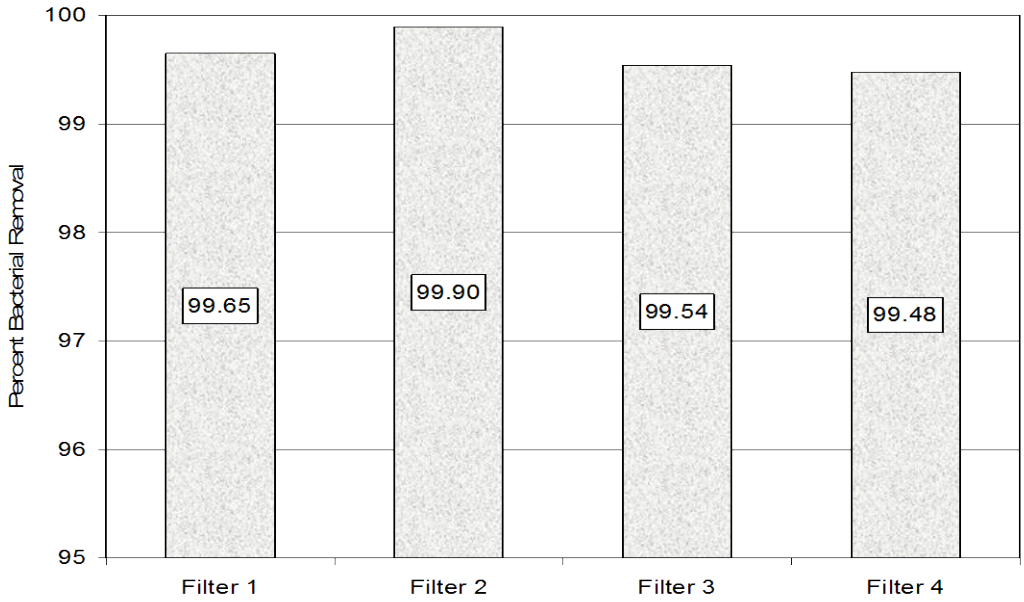


Figure 35. Percent bacterial removals for second spike tests.

Compared to the first spike test, Filters 1 and 2 showed increases in bacterial removal efficiency, whereas a decrease in bacterial removals was observed in Filters 3 and 4 (Table 16). The increase in bacterial removals observed in Filters 1 and 2 is attributed primarily to 1) more accurate maximum tailwater fecal coliform concentrations than was obtained in Filters 3 and 4 in the first spike test, and 2) maturation of the filter bed and the assumed development of the *schmutzdecke* in Filter 1 and 2. The maturation and filtration behavior of Filters 3 and 4 is difficult to ascertain based on the observed decrease in removal rate. It appears that decreased removals are actually an artifact of inaccurate baseline data regarding removal efficiency obtained in the first spike.

Table 16

Comparison of Bacterial Removals for all Filters' First and Second Spike Tests

	Logarithmic Removals		Percent Removals	
Filter	First Spike	Second Spike	First Spike	Second Spike
1	1.66	2.46	97.70	99.65
2	1.47	2.98	96.63	99.98
3	3.22	2.34	99.94	99.54
4	3.40/3.01*	2.28	99.96/99.90*	99.48

* refers to high MPN at ET 0:30

The maximum tailwater MPN for Filter 1 appeared to have been obtained during the 8-hour sampling period following the first spike (see Figure 24, ET 4:00) therefore the baseline removal efficiency established in the first spike for Filter 1 appears reasonable. Filter 2 was showing an increase in tailwater fecal coliform concentration at the time the 8-hour sample was drawn (see Figure 25, ET 8:00) as was seen with Filters 3 and 4 (Figures 26 and 27). However, Filter 2's relatively small maximum headwater MPN and relatively high tailwater MPN at ET

8:00 suggests that the tailwater MPN obtained at ET 8:00 hours may have closely approximated the maximum tailwater MPN, as the relative difference in maximum headwater MPN to the 8-hour tailwater MPN was one order of magnitude.

In contrast, Filter 3 had a three-order of magnitude difference between maximum headwater MPN and the 8-hour tailwater MPN (Figure 26). Filter 4 had a four-order of magnitude difference in these values (Figure 27). It is possible that the 8-hour sampling period failed to capture maximum tailwater bacterial. Thus, baseline removal rates obtained in the first spike may have overestimated removal efficiency in Filters 3 and 4 at start-up.

Since the filters were not disassembled for examination of filter bed surfaces following the second spike test, assumptions regarding bed maturation and *schmutzdecke* development are made based on normal processes associated with filter bed ripening. Filters 1 and 2 exhibited expected results following these assumptions. Filters 3 and 4, however, did not indicate maturation, but it is possible that the abbreviated (8 hour) sampling period in the first spike test failed to provide enough time for the maximum concentrations of fecal coliforms to reach the tailwater.

4.2.4 Results of Third Spike Tests

The final spiking tests were conducted in December 2000. Due to budgetary constraints, it was decided to limit the spiking tests to two filters. Filters 1 and 3 were selected for the spike, with selection based upon filter maturity and differences in filtration media. Unforeseen difficulties arose, however, following the cleaning of Filter 1 the week prior to the spike. Following the scraping of Filter 1, workers did not re-start flow, a condition unnoticed until December 19. This posed a problem for the spike, as the only alternative choice for spiking a filter with imported media was Filter 2, which had recently reached terminal headloss and

required scraping. Since it was decided that one of each type of filter media would be spiked, it was decided to spike Filter 1 as planned, since bacteria would be lost in overflow of Filter 2.

The results of the third spike test, shown in Table 17, indicate considerably higher concentrations of fecal coliforms in the tailwater of Filter 1 (relative to Filter 3), as would be expected following scraping (see Appendix F for all results of third spike tests). The time series plots for the average MPN's of fecal coliforms in Filter 1 and 3 are presented in Figures 36 and 37.

Table 17

Headwater and Tailwater MPN's for Filters 1 and 3 Third Spike Tests

	Average MPN's of Fecal Coliforms per 100ml							
	December 21				December 21			
	Filter 1		Filter 2		Filter 3		Filter 4	
ET (hr:min)	Head	Tail	Head	Tail	Head	Tail	Head	Tail
0:00	38,580	11.10			26,877	1<		
0:30	____	____			54,530	1<		
2:00	31,695	20.35			54,530	1<		
4:00	31,183	681.35			32,943	1<		
6:00	3,210	144.80			10,726	2.05		
9:00	1,532	126.65			8,146	1.55		
12:00	933	82.35			5,974	9.85		
16:00	393	50.25			4,249	2.60		
Max. MPN	38,580	681.35			54,530	9.85		

The sample drawn thirty minutes after the spike of Filter 3, was taken in an effort to obtain a more accurate estimate of the maximum headwater fecal coliform concentration, since Filter 3 had a headwater volume two and a half times that of Filter 1 (81.7 L versus 32.7 L) due to differences in bed maturation and associated headloss. To address this situation, additional stirring of Filter 3's headwater was done, but the four-foot depth made it less likely that adequate

mixing of the bacteria would occur. Therefore, a sample was drawn at ET 0:30 for Filter 3. Since the headwater volume of 32.7 L in Filter 1 was close to the volume at start-up in June (29.2L), it was determined that an additional sample at ET 00:30 was not necessary for Filter 1.

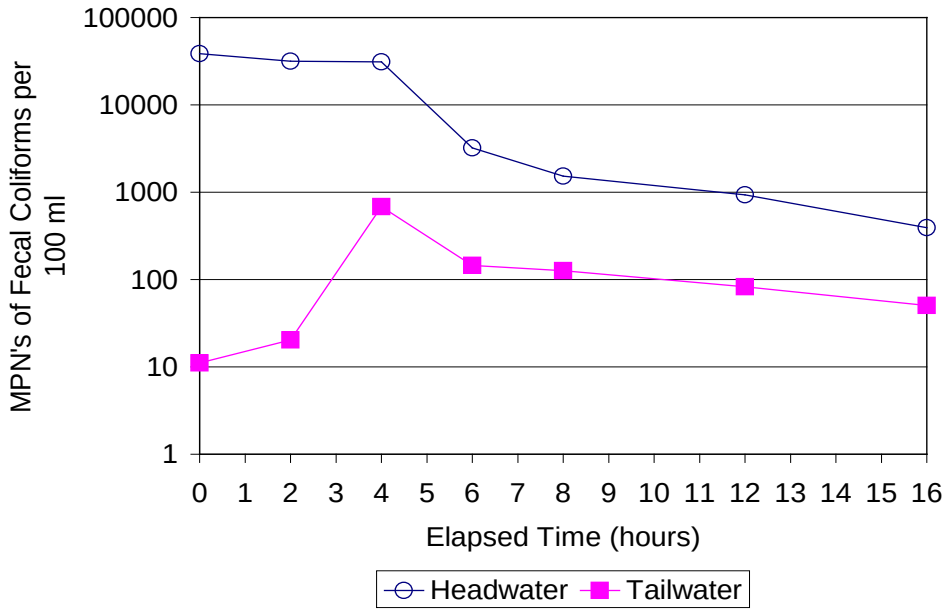


Figure 36. Filter 1: Average MPN's of fecal coliforms for third spike test.

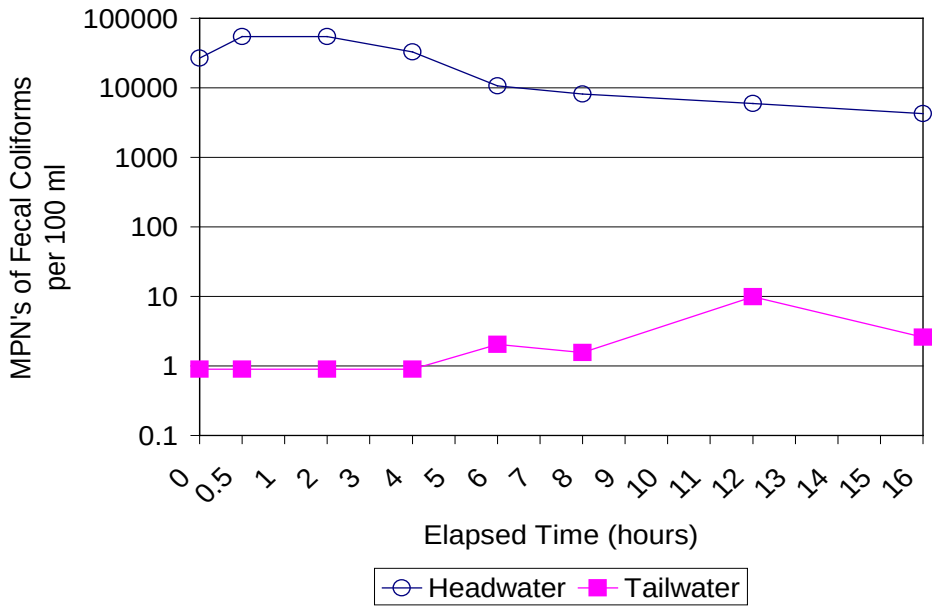


Figure 37. Filter 3: Average MPN's of fecal coliforms for third spike test.

Logarithmic bacterial removals and percent bacterial removals indicated considerable differences in removal efficiency between filters during the third spike test

(Figure 38 and Figure 39). The results also indicate that bacterial removals decreased considerably following scraping. These results may provide a baseline for future tests to determine how long it takes for filter recovery of removal efficiency after scraping.

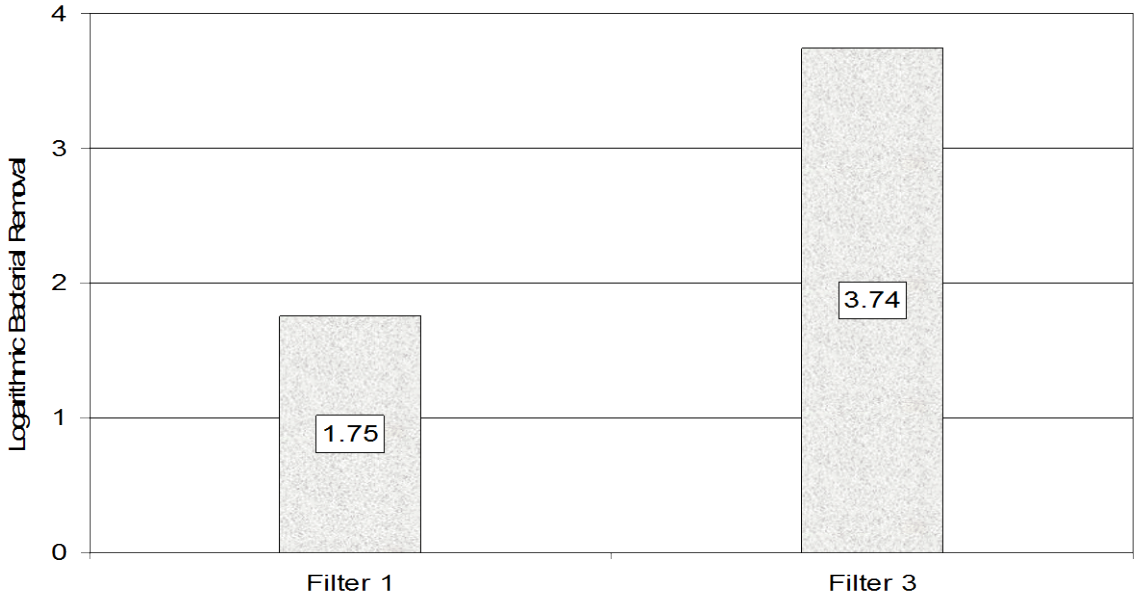


Figure 38. Logarithmic bacterial removals for third spike tests.

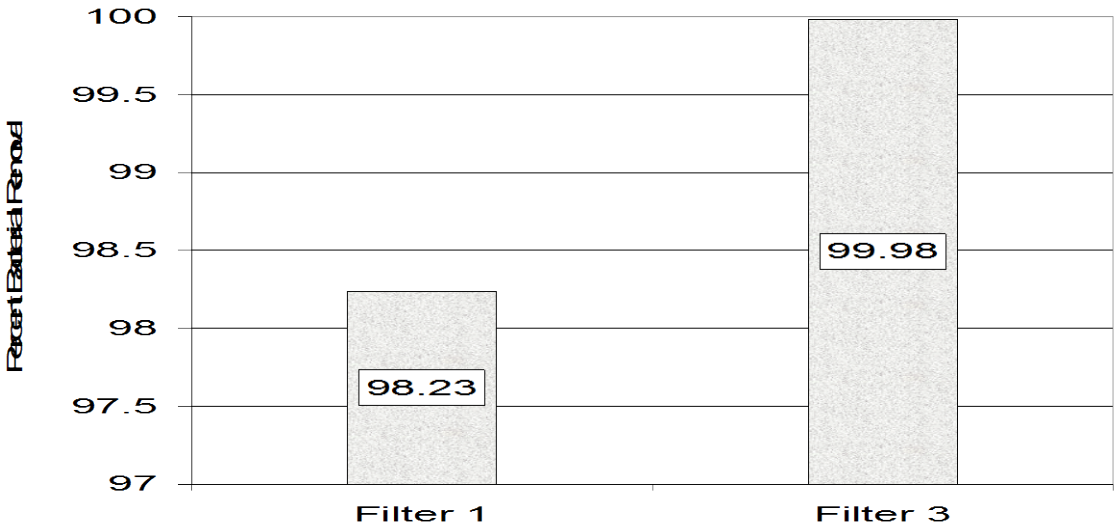


Figure 39. Percent bacterial removal for third spike tests.

4.2.5 Summary of Results from All Spike Tests

Logarithmic fecal coliform removals and percent bacterial removals are summarized for all spike tests in Table 18 and Table 19.

Table 18

Summary of Logarithmic Removals for All Filters and All Spike Tests

Summary of Logarithmic Removal of Fecal Coliforms For All Spike Tests Conducted On All Filters					
Spike Test	Filter 1	Filter 2	Filter 3	Filter 4	Filter 4*
1	1.66	1.47	3.22	3.40	3.01
2	2.46	2.98	2.34	2.28	2.28
3	1.75		3.74		

* indicates high MPN at ET 0:30

Table 19

Summary of Percent Bacterial Removals for All Filters and All Spike Tests

Summary of Percent Removal of Fecal Coliforms For All Spike Tests Conducted On All Filters					
Spike Test	Filter 1	Filter 2	Filter 3	Filter 4	Filter 4*
1	97.69	96.63	99.94	99.96	99.90
2	99.65	99.89	99.54	99.48	
3	98.23		99.98		

*indicates high MPN at ET 0:30

4.3 Weekly Fecal Coliform Analysis

4.3.1 Results of Weekly Fecal Coliform Testing

The results of weekly testing for fecal coliforms show differences of generally one (+/-) order of magnitude between filter headwater and tailwater throughout the testing period (Figures 40-43). Additionally, the average concentrations of respective headwater fecal coliforms and tailwater fecal coliforms are similar, as shown in Table 20 (see Appendix I for all weekly data July through December).

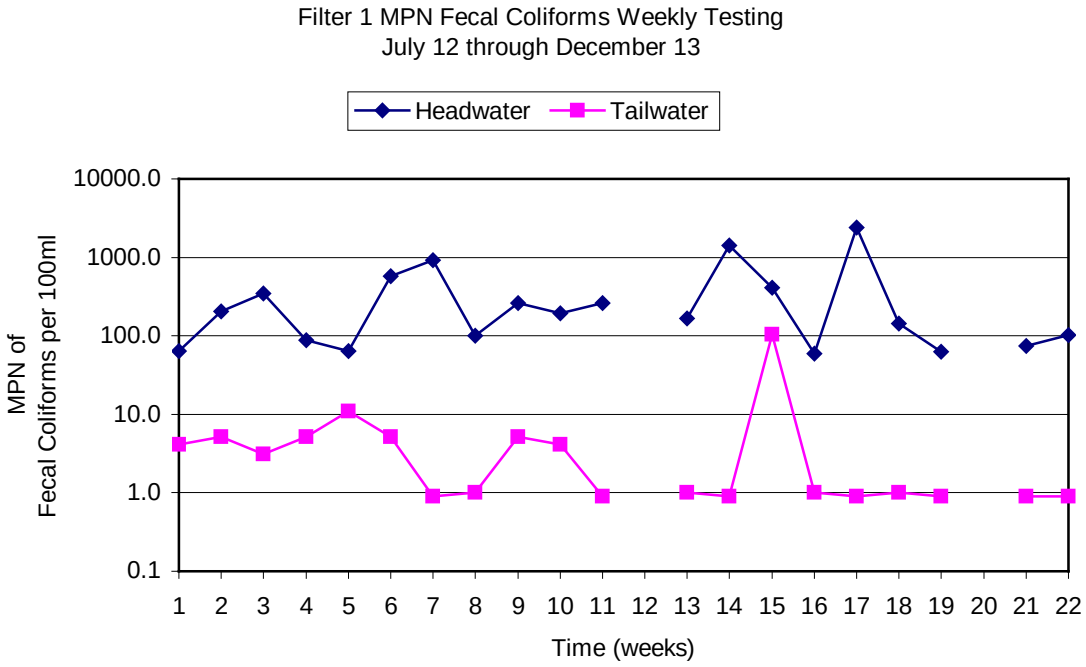


Figure 40. Filter 1 weekly headwater and tailwater MPN's of fecal coliforms.

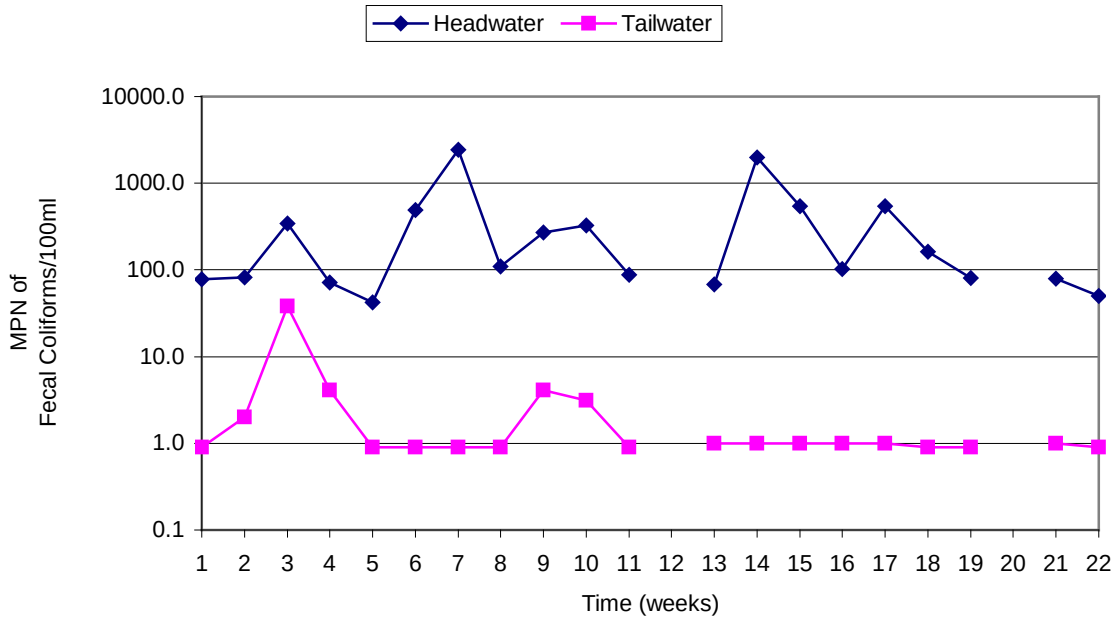


Figure 41. Filter 2 weekly headwater and tailwater MPN's of fecal coliforms.

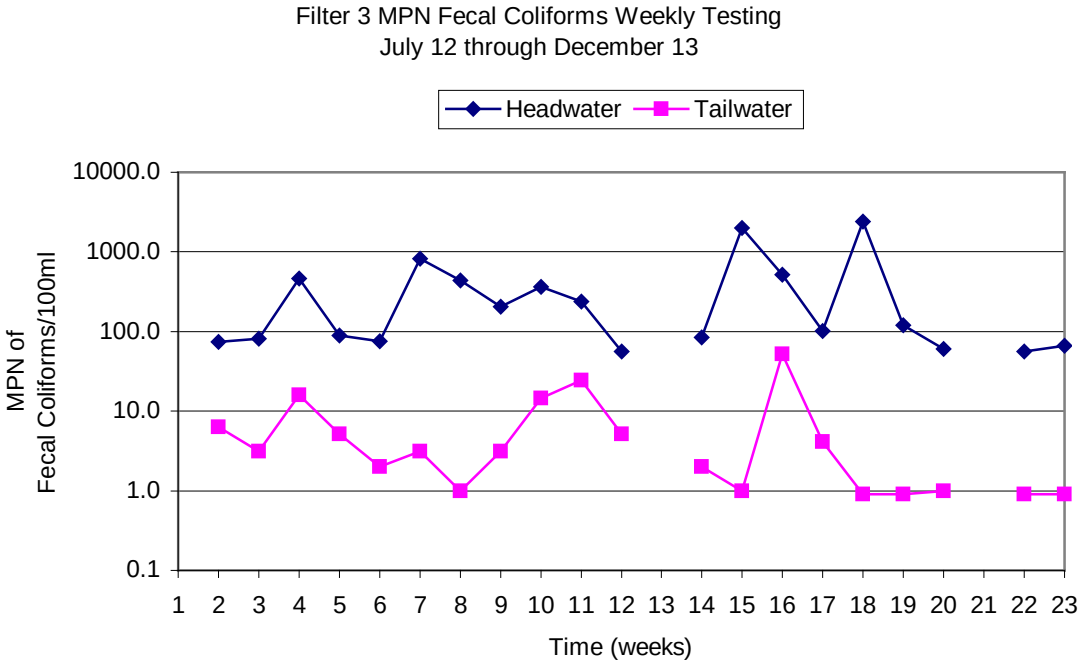


Figure 42. Filter 3 weekly headwater and tailwater MPN's of fecal coliforms.

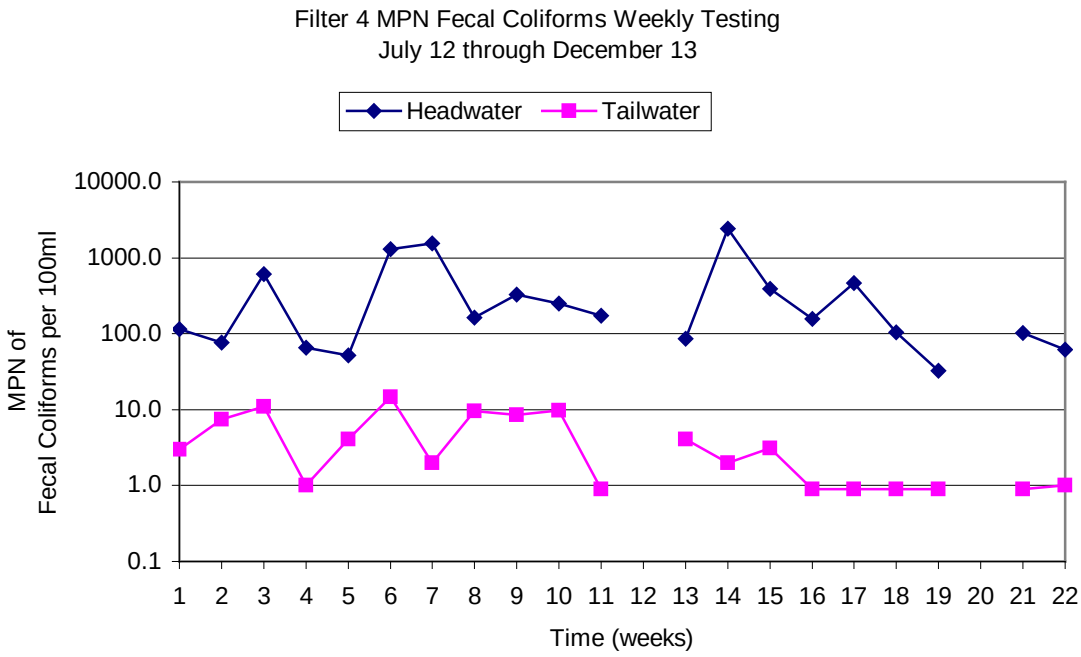


Figure 43. Filter 4 weekly headwater and tailwater MPN's of fecal coliforms.

Table 20

Average MPN's of Headwater and Tailwater Fecal Coliform Obtained Over Twenty Weeks

Five-Month Average MPN's of Fecal Coliforms per 100 ml		
Filter	Headwater	Tailwater
1	397	8
2	397	3
3	310	7
4	320	4

The weekly data is not sufficient for determining actual logarithmic bacterial removal or percent bacterial removal values, as is obtained through spiking, however, it does indicate similarities in all filters regarding the relative numbers of fecal coliforms in the headwater. The same is evident in concentrations of fecal coliforms in the tailwaters. The monthly ANOVA tests for headwater fecal coliforms revealed no significant difference ($P < 0.05$) in relative headwater MPN of the filters for any of the five months. Similarly, no significant difference ($P < 0.05$) in tailwater fecal coliforms was indicated.

The ANOVA constructed from grouping the twenty weeks of sampling data revealed no significant difference between means at the P 0.01 level in headwater MPN of fecal coliforms. The same was true for the tailwater MPN of fecal coliforms.

The results of the monthly (4-week) ANOVA tests and overall (20-week) ANOVA test indicate that headwater concentrations of fecal coliform in the four filters were not significantly different. Likewise, the MPN of fecal coliform in the tailwater were not significantly different in the four test filters. Therefore, the results of the ANOVA tests suggest that bacterial removal efficiency was not significantly different in mature filters consisting of either imported media or locally manufactured basalt media.

4.3.2 Recovery of Bacterial Removal Efficiency After Scraping

After scraping Filter 1 and 3 in November, it was observed that tailwater fecal coliform concentrations had increased (Figure 40 Week 15; and Figure 42 Week 16). One week after scraping, however, the tailwater counts for Filter 1 (Figure 40) returned to the low numbers seen prior to scraping; within two weeks, bacterial removals for Filter 3 (Figure 42) had also seen a decrease in tailwater MPN. This limited data suggests that a filter's bacterial removal efficiency appears to recover in one to two weeks, and may be influenced by media type. Further pairing of

scraping times and bacterial testing is necessary for obtaining a more accurate appraisal of the actual recovery time.

4.4 Turbidity Analysis

4.4.1 First Month of Plant Operation

The average headwater and tailwater NTU measurements for 31 readings taken during the first month of operation indicate similarities in headwater turbidity levels and differences in tailwater values (Table 21).

Table 21

Average NTU's of All Filters for First Month of Operation

Average NTU's for First Month				
	Filter 1	Filter 2	Filter 3	Filter 4
Headwater	3.3	2.8	2.8	3.1
Tailwater	18.4	7.1	3.2	3.7

The ANOVA applied to headwater measurements showed no significant difference in headwater turbidity among any of the filters at the p 0.10 level. Highly significant differences (P <.005) however, were found to exist in tailwater turbidity . To determine which filters were exhibiting these differences, Tukey's honestly significant difference test (HSD) was applied, revealing that Filter 1 was significantly different ($P<0.05$) from the other three filters regarding tailwater turbidity. Further analysis using Tukey's HSD test revealed significant differences ($P<0.01$) only between Filter 1 and Filter 3 and Filter 1 and Filter 4.

Lower average tailwater turbidities in Filters 3 and 4 during the first month were attributed to 1) larger sand sizes containing initially fewer fines and 2) better removal of fines in the washing of the basalt media prior to start-up. Initially high turbidity in Filters 1 and 2 was primarily attributed to smaller sand sizes and a greater amount of initial fines washing out of the media. However, of the two filters containing imported filtration media, Filter 2 had the lowest recorded average NTU, yet it received dry, unwashed sand at the start of operations.

4.4.2 Turbidity Measurements Obtained in September

A total of 24 samples per filter per source (headwater or tailwater) were taken over a 12-day period in September. No statistical differences in headwater or tailwater turbidity levels were found at the p 0.10 level.

4.5 pH Analysis

4.5.1 pH Levels During the First Month of Plant Operation

The average pH values obtained during the first month of the study suggested significant differences in tailwater pH between the imported and local media (Table 22) (Appendix G).

Table 22.

Average pH for Each Filter During First Month of Operation

	Average pH for First Month			
	Filter 1	Filter 2	Filter 3	Filter 4
Headwater	7.96	7.93	7.89	7.91
Tailwater	7.68	7.56	8.33	8.32

The ANOVA on tailwater pH showed significant differences and Tukey's HSD test revealed highly significant differences ($P<0.01$) in mean tailwater pH. The greatest differences are between the imported filters (1 and 2) and the local basalt media filters (3 and 4). There is also a significant difference between the two filters containing imported media, though it is considerably less than that which exists between them and Filters 3 and 4.

SUMMARY AND RECOMMENDATIONS

5.1 Performance Summary

5.1.1 Bacteria Removal

One of the criteria for adopting slow sand filter technology for treating raw water is how well the plant can remove contaminants such as bacteria and turbidity. According to Hendricks 1991, a 99% bacterial removal rate is the acceptable level for slow sand filters. From the pilot study the following results were obtained:

- The bacteria removals for filter 1 & 2 with off island sand media and filters 2 & 3 with local sand media after 30 days were above 2-log cycles**.** As reported in Table 19, the bacterial removal rates for filter 1 through 4 were 99.65, 99.89, 99.54, and 99.48% respectively after 30 days of operation.
- The local basalt media (Filter 3&4) is capable of bacterial removal rates up to 3-log cycles (99.8%). This removal rate was obtained for filter 3 after 6 month of operation.
- The average recovery time for bacteria removal after scraping varies between the two types of media tested. Recovery times varied from one week for imported media to two weeks for the local sand.

5.1.2 Turbidity

According to the US EPA, safe drinking water should have turbidity 1 NTU 95 percent of the time, never to exceed 5 NTU's. In the Kosrae pilot plant study we monitored turbidity only during the first month of operation (project startup) and 12 days of monitoring during the month

of September. Due to equipment break down, the Kosrae personnel weren't able to provide continuous daily monitoring of inflow/outflow turbidity. The results presented in Table 22 indicate that:

- For the first month of operation the local media (Filter 3 and 4) had lower tail water turbidity (3.2, 3.7 NTU for filter 3 and 4 respectively) than did the imported media (18.4, 7.1 NTU for filter 1 and 2 respectively).
- After the first month, the reading during the month of September indicated that filter 1 and 2 with imported media had turbidity of 2.4 and 2.5 NTU respectively. The filters with local media (3 and 4) had turbidity of 1.6 and 3.4 NTU respectively.

5.1.3 Scraping Time

The time between scraping or filter run length is the length of time a filter can effectively operate under a constant Hydraulic Loading Rate (HLR) before cleaning the top layer is required. The target HLR for the Kosrae pilot plant was 0.2 m/hr. However, the average actual hydraulic loading that was observed in the Kosrae study was between 0.17 and 0.2 m/hour. This change in HLR was due to fluctuations of inflow to the filters. The cause of these fluctuations was blockage of the flow meter orifices or obstruction of flow into the inflow pipe at the Tofol dam. The results of run time studies are as following:

- The local media had an initial run length of 114 and 180 days, but it showed decreases in length with subsequent runs.
- The imported media exhibited the shortest average run length (Filter 1) with less than 30 days (This is below the accepted run length time of 60 days identified by Hendricks, 1991.)

The decrease of run lengths for local basalt media was probably due to the large value of the Uniformity Coefficient (UC) for the local media (4.29). A large value of UC indicates a tendency for more of the sediment carried by the inflow to be deposited throughout the pore spaces deeper into the filter, thereby slowing sediment build-up on the top of the sand bed. While this kind of build up makes initial run times longer, successive runs will become shorter and shorter. At some point, costly deep cleaning or complete replacement of the filter material would be required.

5.1.4 Hydraulic Loading Rate

According to Kosrae's Water Master Plan, 1.6 million gallons per day of untreated water is being diverted from the Tofol River to Tofol-Lelu Municipality system via gravity flow. To treat this quantity of water using the hydraulic loading rate of 0.2 m/hour that was used in this study, requires a filter bed area of approximately 14,000 ft^2 (approximately 1/3 acre). Using a four (4) foot bed media, the sand volume would be 56,000 cubic feet or 2074 cubic yards. In order to avoid any interruption in water delivery during periodic filter scraping and maturation operations, it would be desirable to have a backup filter of equivalent size.

5.2 Recommendations

Given the successful levels of bacteria removal provided by both types of media, it is highly recommended that additional research be directed at resolving critical questions left unanswered by this pilot study such as:

- Can hydraulic loading rates for the local media be raised without loss in bacteria and turbidity removal rates?

If hydraulic loading rates can be increased without sacrificing bacteria or turbidity removal rates then initial cost can be reduced. In our study, the local media had run length of 114 and 180 days, which is much greater than normally experienced by slow sand filters. It is important to find out what would be the optimum hydraulic loading rate for local media, and how the plant would perform under this hydraulic loading.

- Would a different size distribution resulting in a lower uniformity coefficient increase the effectiveness of the local filter media in reducing inflow turbidities, and how would that affect filter run length times?

It is recommended that experimentation with the UC of local media be conducted to determine if lower UC values can be obtained using local equipment. If this is possible, then basalt media with lower UC should be tested. These tests would determine if deep bed deposition of sediment could be decreased and run length not adversely affected by using local media with lower UC values. Future studies should include more consistent turbidity readings and the size and composition of particulate matter causing the turbidity should be examined to evaluate potential for pre-treatment applications.

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APPENDICES

APPENDIX	A	Spreadsheets and rules for sizing gravel bed
APPENDIX	B	Headloss Data
APPENDIX	C	Flow records July through December
APPENDIX	D	Results of first spike tests
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