EVALUATION OF UPFLOW FILTERATION FOR THE TREATMENT OF STORMWATER

by

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LIST OF ABBREVATIONS AND SYMBOLS

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CHAPTER I

INTRODUCTION

1.1 Back Ground

The US EPA has reported that only 57% of the rivers and streams in the U.S. fully support their beneficial uses. A wide variety of pollutants and sources are the cause of impaired uses but runoff from urban and agricultural sources dominate. Even in non-industrialized urban areas, metallic and organic contamination of local streams can be high. Children and others, playing in and near the streams therefore are exposed to potentially hazardous conditions. In addition, inner city residents sometimes rely on close-by urban waterways for fishing opportunities, both for recreation and to supplement food supplies.

Stormwater runoff from paved parking and storage areas, and especially gas station areas, has been observed to be contaminated with concentrations of many critical pollutants. These paved areas are usually found to contribute most of the pollutant loadings of toxicants to stormwater outfalls. These critical source areas may contain pollutant loadings of hydrocarbons, toxic trace metals, nutrients, and pathogens that are greater than the loadings of "normal" runoff (Bannerman, *et al*. 1993; Pitt, *et al*. 1995; Claytor and Schueler 1996). Treating the runoff from these critical source areas before it mixes with the runoff from "non-problem" source areas such as residential areas, institutional developments, and non-industrial rooftops could be considered as an alternative to end-of-the-pipe treatment (Bannerman, *et al*. 1993; Pitt, *et al*. 1995; Claytor and Schueler 1996). This approach is highly encouraged by the industry due to reduction in treatment facility; however the disadvantage of such treatment is multiple installations (Pratap 2003).

The first concern when investigating alternative treatment methods is determining the needed level of stormwater control. This determination has a great affect on the cost of the stormwater management program and need to be carefully made. Problems that need to be reduced range from sewerage maintenance issues to protecting many receiving water uses. This treatment objective can be easily achieved using a number of cost effective source area and inlet treatment practices. In contrast, much greater levels of stormwater control are likely needed to prevent excessive receiving water degradation. Specific treatment goals usually specify about 80% reductions in suspended solids concentrations. In most stormwaters, this would require the removal of most particulates greater than about 10μm in diameter, about 1% of the 1mm size to prevent sewerage deposition problems. Obviously, the selection of a treatment goal must be done with great care.

Numerous manufacturers have developed proprietary devices to treat stormwater runoff. These devices have been designed to treat one or more of the common stormwater pollutants – solids, metals, oil and grease, nutrients and bacteria. Few have been designed to treat a broad range of pollutants in a single device. In addition, many of these devices provide inconsistent performance from one installation to another. Treatment of runoff from critical source areas requires a device with robust removal ability.

There are many stormwater control practices, but all not suitable in every situation. It is important to understand which controls are suitable for the site conditions and can also achieve the required goals. This will assist in the realistic evaluation for each practice of: technical feasibility, implementation costs, and long-term maintenance requirements and costs. It is also important to appreciate that the reliability and performance of many of these controls have not been well established, with most still in the development stage. This is not to say that emerging controls cannot be effective, however, they do not have a large amount of historical data on which to base designs or to be confident that performance criteria will be met under the local conditions. The most promising and best understood stormwater control practices are wet detention ponds. Less reliable in terms of predicting performance but showing promise, are stormwater filters, are wetlands and percolation basins. Grass swales also have shown great promise during the EPA’s Nationalwide Urban Runoff Program (NURP) (EPA 1983; Nara 2005).

Most stormwater needs to be treated to prevent harm either to the surface waters or the ground waters. One approach is to treat the runoff from critical source areas before it mixes with the runoff from less pollutant areas. The general features of critical source areas appear to be large paved areas, heavy vehicular traffic, and outdoor use or storage of problem pollutants. The control of runoff from relatively small critical source areas may be the most cost effective approach for treatment/reduction of stormwater toxicants. However, in order for a treatment device to be usable, it must be inexpensive, both to purchase and to maintain, and effective. Outfall stormwater controls being located at the outfalls of storm drainage systems treat all the flows that originate from the watershed. The level of treatment provided, of course, is greatly dependent on many decisions concerning the design of the treatment devices. Source area controls are, of course, physically smaller than outfall controls but may be difficult to be located on a crowded site, and there could be a great number of them located in a watershed. In all cases, questions must be answered about the appropriate level of control should be provided, and what controls should be used.

Upflow filtration which is the chosen treatment technology for this research has shown promising results. (See the “Future Research” section of Clark 2000). Extensive research on flow type and potential suitable media for downflow filtration has been carried out by Clark and Pitt (1999) and Clark (2000). But such information is not available for upflow filtration. Pratap (2003), in conjunction with Gill (2004), further studied and analyzed upflow filtration at a lab scale and evaluated several media for potential treatment effectiveness. The primary of this research is to incorporate those results in a full scale testing of upflow filter, currently being funded by SBIRI and SBIR2 research by the US EPA.

Upflow filtration was selected for this research due to the following drawbacks of downflow filtration:

1. Downflow filters clog at a fast rate, reducing their flow rate potential and treatment capacity. Earlier research on the effects of clogging on the flow rate through sand and mixed media filters have shown that flow rate of the water is dependent on the suspended solids loading on the media (Urbonas 1999 and Clark 2000). Clogging

does not occur as fast in upflow filtration, the reason being, heavier particles get drawn away from the filtration interface due to gravity and fall into the sump which is an integral part of UpFlow™ filter design.

2. "Successful interactions usually take place between the pollutants and the filtration media when the residence time in the filter is long, and when the flow rate is slow. However, drainage design is primarily concerned with rapidly removing the water from the drainage area, dictating relatively fast filtration rates, or very large filters. Also, immature clogging of standard filters rapidly reduces the flow rates below the design flows needed for adequate drainage, resulting in much more frequent and larger bypassing of the filter media than expected" (Pratap 2003).

3. "The clogging problem leads to the need for regular maintenance that is integral to long-term downflow operation. In locations where the filter is receiving large suspended solids loadings, the filter size must be large to have a long filter run period before needed maintenance. To reduce the large filter surface area, the stormwater runoff must be pretreated to remove the solids loading prior to entry to the filter, with the filters left to act as a polishing step" (Pratap 2003).

1.2 Objectives of Thesis

This research is part of the Phase II project of the US EPA's Small Business Innovative Research (SBIR). Earlier work on upflow filtration has been carried out by Gill (2004) and

Pratap (2003) in lab scale obtaining promising results for treating stormwater and also by a pilot scale upflow filtration setup in Phase I of this project. However, there were many questions that were unanswered by the lab scale and the pilot scale testing due to their limitations. The objectives of this thesis are listed below:

1. To determine how the head loss and associated treatment rate change during filtration and how does the head recover flow rate change between filtration events. Will a recovery in head loss be seen due to the removal of solids from the filter surface to the sump during the quiescent periods.
2. To find out the effect of decreasing/increasing the flow rate through the filters on the treatment efficiency and also the effect of aging on filter efficiency.
3. To verify if the UpFlow^{TM} configuration is feasible for actual operation
4. Part of the benefit of the upflow design will be that the surface of the filter will strain out the larger particulates during operation. Between storms, the water will be quiescent and the sump would be used to catch the particulates that fall from the filter's surface. Hence it is important to find out what size sump would be required and how will be the particulates collected in the sump be managed efficiently.
5. To compare different media for their head loss and particulate trapping capabilities.

CHAPTER 2

LITERATURE REVIEW

2.1 Background

A catchbasin is defined as a “chamber or well, usually built at the curbline of a street, for the admission of surface water to a sewer or sub drain, having at its base a sediment sump designed to retain grit and detritus below the point of over flow”(John A lager, William G. Smith, and George Tchobanoglous 1977). An inlet is defined as an opening on the curbline which provides a means for the stormwater to enter the main pipeline and does not have a sump to trap the sediment in the stormwater. It is important to note the difference between catchbasins and inlets as they are two different collection devices as part of the stormwater collection network. This research focus on filtration devices retrofitted to catchbasins having sumps and inserts to enhance their performance.

In many urban areas, stormwaters flow to, and then along, street-side gutters for short distances and then enters catchbasins or inlets that are connected to underground pipes which in turn discharge the stormwater to receiving waters. The original purpose of catchbasins was to trap sediment and debris before it can accumulate in the pipe drainage system. This was particularly important prior to the existence of paved streets when the stormwater carried substantial amounts of large debris (American Public Works Association, 1969). However, catchbasins were considered to be only partially useful to trap sediment (Folwell, A.P. 1928). The use of catchbasins was considered more of a custom than to meet technical needs, but they are still used widely in many jurisdictions (American Public Works Association, 1973).

Figures 2.1 and 2.2 show typical catchbasins that are in use in the US, Canada and Europe. There are large variations in catchbasin sizes. The major types shown here are simple catchbasins and selective inlets where a bucket sieve is used to collect gross solids (John A. Lager et. al, 1977). European catchbasins are generally smaller in size, serving smaller catchment areas. In the UK, catchbasins are called gullypots. All of the locations shown on these two figures have combined sewers and the hooded sumps act as a trap to prevent sewer gasses from escaping from the sewerage. Storm drainage system inlet structures for small areas can be considered in the following general categories (Ashley, et al. 2004):

(i) Simple inlet. An inlet comprising a grating at the curb and a box with the discharge located at the bottom of the box, which connects directly to the main storm drainage or combined sewerage system. This inlet simply directs the runoff to the drainage system and contains no attributes that would improve water quality and solids removal. However, large debris (several cm in size) may accumulate in the inlet box if present in the stormwater.

(ii) Inlet with a sump. Similar to the simple inlet above, but incorporating a sump that typically extends about 0.5 to 1 m below the bottom of the outlet. This is termed a catchbasin in the US, or a gully pot in Europe, and has been shown to trap moderate amounts of coarse sediment. A hooded outlet is important to improve trapping of debris and floatable material.

(iii) Screened inlet. Similar to the simple inlet, but includes a screen to trap large debris. This type typically includes small perforated metal buckets placed under the street grating (common in Germany), large perforated stainless steel plates placed under the inlet grating (used in Austin, Texas), and a number of proprietary devices incorporating filter fabric or other types of screening placed to intercept the stormwater flow. These may trap large debris and litter, depending on the overflow arrangement, but have not been shown to produce significant water quality improvements and typically have a very limited useful life before clogging and requiring maintenance.

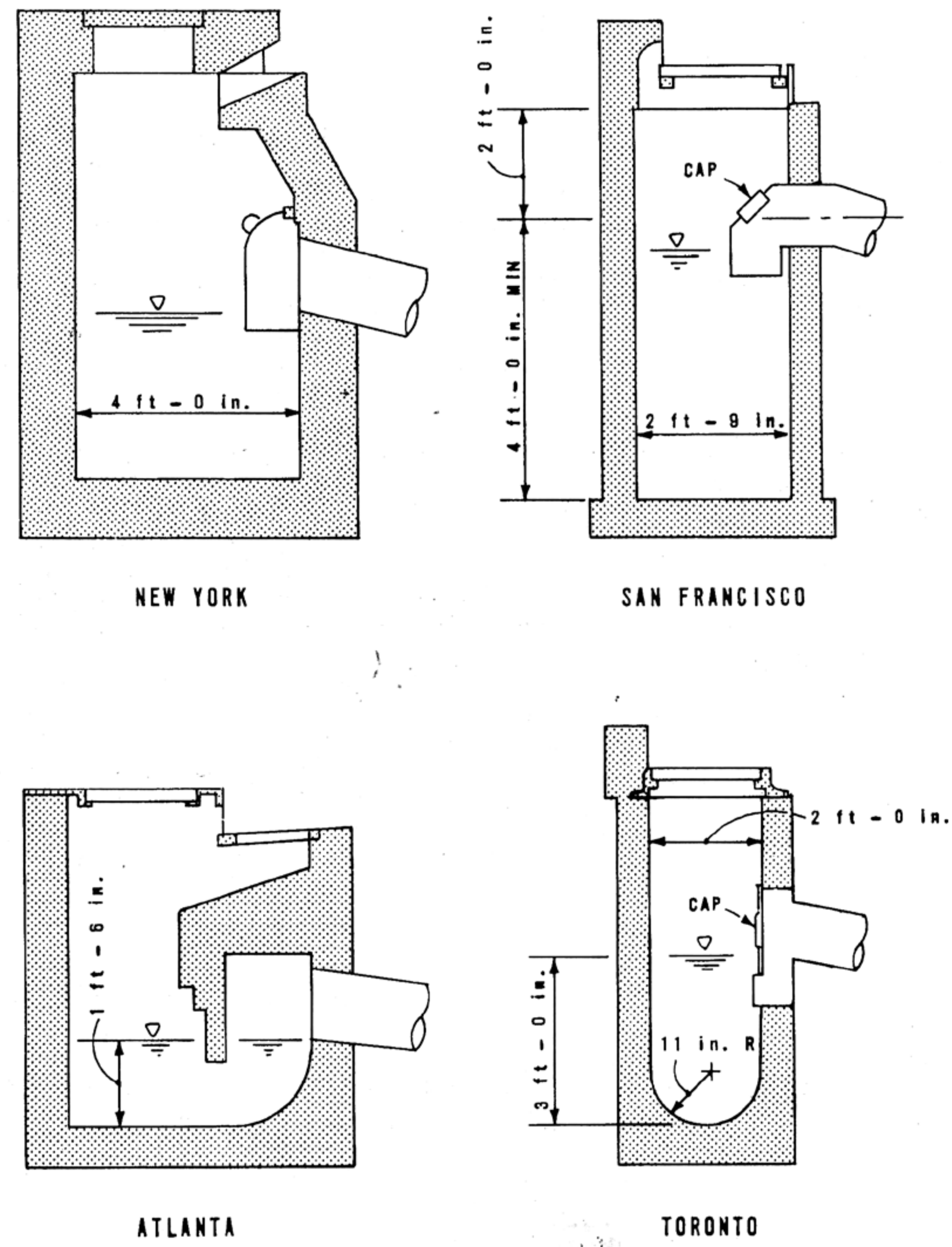

Figure 2.1: Typical types of catchbasins used in United States and Canada

(John A. Lager et. al, 1977)

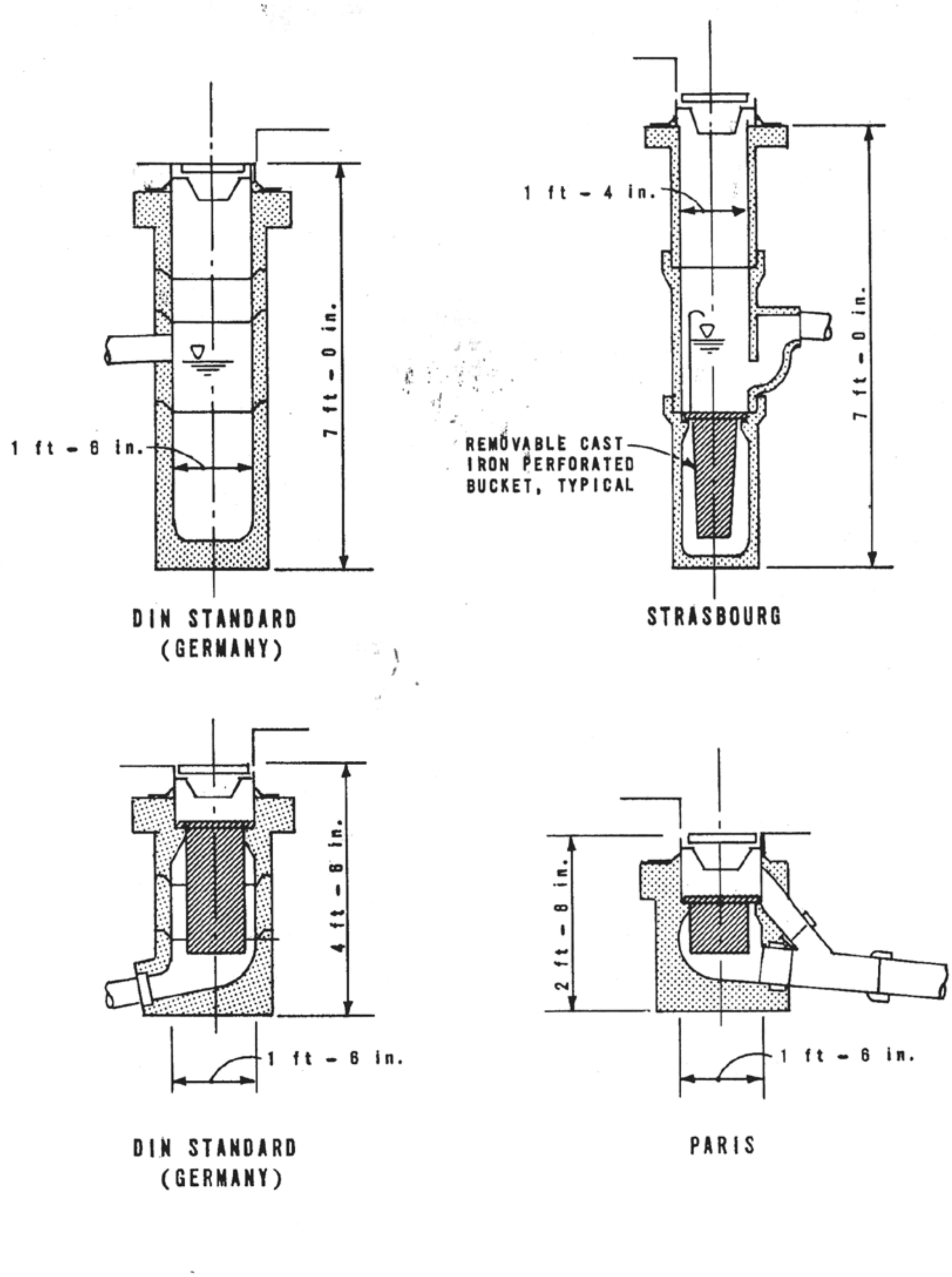

Figure 2.2: Typical types of catchbasins and inlet baskets used in Europe (John A. Lager et. al, 1977).

2.2 Pollutant Characteristics

The major pollutant categories are floatable, dissolved, suspended and settleable. Catchbasins have been found to be effective in removing coarse inorganic particles, but are less efficient in removing finer solids and organic matter (Sartor, J.D. and G.B. Boyd. Water Pollution Aspects of Street Surface Contaminants, USEPA, 1972). Maestre and Pitt (2005) evaluated variations in stormwater quality from about 85 US cities (Table 2.4), contained in the National Stormwater Water Quality Database (NSQD). These data are from the EPA's MS4 Stormwater Permit Program. Generally, industrial and freeway areas have more contaminated runoff than other land uses. Few seasonal variations were identified, except for bacteria (highest in warm weather). Geographical regions were also found to have significantly different stormwater characteristics. These data were obtained from outfall locations, while stormwater inlet concentrations are generally greater. Inlet (and source area) stormwaters generally have a larger abundance of larger particles than outfalls, as these large materials tend to accumulate in the sewerage system and are seldom discharged.

Table 2.4: Summary of MS4 Stormwater Outfall Data from National Stormwater Quality Database NSQD, version 1.1 (Maestre and Pitt, 2005)

	Area (acres)	% Impervious	Precipitation Depth (in)	Runoff Depth (in)	Conductivity (µS/cm @25ºC)	Hardness (mg/L CaCO3)	Oil and Grease (mg/L)	pH	Tempe-rature (C)	TDS (mg/L)	TSS (mg/L)	BOD_5 (mg/L)	COD (mg/L)
Overall Summary (3765)													
Number of observations	3765	2209	3316	1495	685	1082	1834	1665	861	2956	3493	3105	2750
% of samples above detection	100	100	100	100	100	98.7	66.1	100	100	99.0	97.9	96.2	98.4
Median	57.3	50.0	0.48	0.15	121	38.0	4.3	7.5	16.5	80	59	8.6	53
Coefficient of variation	3.7	0.4	1.0	1.9	1.6	1.4	9.7	0.1	0.4	3.4	1.8	7.4	1.1
Residential (1042)													
Number of observations	1042	614	919	372	104	215	483	286	181	814	978	908	748
% of samples above detection	100	100	100	100	100	100	54.9	100	100	99.1	98.3	97.1	98.7
Median	57.3	37.0	0.48	0.10	102	32.0	4.0	7.2	17.0	72.0	49	9.0	54.5
Coefficient of variation	4.8	0.4	1.0	1.5	1.6	1.1	7.8	0.1	0.4	1.1	1.8	1.5	0.93
Commercial (527)													
Number of observations	527	284	462	146	78	156	331	191	98	418	503	452	393
% of samples above detection	100	100	100	100	100	100	71.9	100	100	99.5	95.2	97.6	98.5
Median	38.8	84.5	0.42	0.29	107	36.5	4.6	7.4	16.0	72	43	11.0	58
Coefficient of variation	1.2	0.1	1.0	1.0	1.0	1.1	3.0	0.1	0.4	1.9	2.0	1.1	1.0
Industrial (566)													
Number of observations	566	292	482	215	102	132	315	248	140	431	521	455	386
% of samples above detection	100	100	100	100	100	96.2	64.8	100	100	99.5	97.7	95.4	99.0
Median	39.5	75.0	0.50	0.16	139	39.0	4.8	7.50	17.9	86	81	9.0	58.6
Coefficient of variation	1.1	0.3	0.9	1.2	1.3	1.5	11.8	0.1	0.3	3.6	1.6	10.0	1.2
Freeways (185)													
Number of observations	185	154	182	144	86	127	60	111	31	97	134	26	67
% of samples above detection	100	100	100	100	100	100	71.7	100	100	99.0	99.3	84.6	98.5
Median	1.6	80.0	0.54	0.41	99	34.0	8.0	7.10	14.0	77.5	99	8	100
Coefficient of variation	1.4	0.13	1.1	1.7	1.0	1.9	0.6	0.1	0.4	0.8	2.6	1.3	1.1
Open Space (49)													
Number of observations	49	37	41	11	2	8	19	19	2	45	44	44	43
% of samples above detection	100	100	100	100	100	100	36.8	100	100	97.8	95.5	86.4	76.74
Median	85	2.0	0.52	0.05	113	150	1.3	7.70	14.6	125	48.5	5.4	42.1
Coefficient of variation	1.5	1.0	1.2	1.4	0.5	0.6	0.7	0.08	0.7	0.7	1.5	0.7	1.5

Table 2.4: Summary of MS4 Stormwater Outfall Data from National Stormwater Quality Database NSQD, version 1.1 (Maestre and Pitt, 2005) (continued)

	Fecal Coliform (mpn/100 mL)	Fecal Strepto-coccus (mpn/100 mL)	Total Coliform (mpn/100 mL)	Total E. Coli (mpn/100 mL)	NH3 (mg/L)	N02+NO3 (mg/L)	Nitrogen, Total Kjeldahl (mg/L)	Phospho-rus, filtered (mg/L)	Phospho-rus, total (mg/L)	Sb, total (µg/L)	As, total (µg/L)	As, filtered (µg/L)	Be, total (µg/L)
Overall Summary (3765)													
Number of observations	1704	1141	83	67	1908	3075	3191	2477	3285	874	1507	210	947
% of samples above detection	91.2	94.0	90.4	95.5	71.3	97.3	95.6	85.1	96.5	7.2	49.9	27.1	7.7
Median	5091	17000	12000	1750	0.44	0.60	1.4	0.13	0.27	3.0	3.0	1.5	0.4
Coefficient of variation	4.6	3.8	2.4	2.3	1.4	0.97	1.2	1.6	1.5	1.7	2.6	1.0	2.5
Residential (1042)													
Number of observations	402	257		14	572	889	922	690	926		395		282
% of samples above detection	87.8	87.9		100	82.2	97.6	96.5	83.5	96.8		40.8		7.8
Median	7000	24300		700	0.31	0.60	1.5	0.18	0.31		3.0		0.5
Coefficient of variation	5.2	1.7		1.6	1.1	1.1	1.1	0.9	1.1		2.2		2.5
Commercial (527)													
Number of observations	253	201			300	445	469	343	466		235		
% of samples above detection	88.9	92.5			83.3	98.0	97.4	81.0	95.9		33.6		
Median	4600	12000			0.50	0.6	1.5	0.11	0.22		2.3		
Coefficient of variation	3.0	2.7			1.2	1.1	0.9	1.3	1.2		2.9		
Industrial (566)													
Number of observations	315	189			272	461	483	344	478	152	255		197
% of samples above detection	87.3	93.7			78.3	96.3	96.3	88.1	96.2	14.5	52.9		10.7
Median	2400	12000			0.42	0.69	1.4	0.10	0.25	3.7	4.0		0.38
Coefficient of variation	5.7	7.0			1.3	0.92	1.1	1.2	1.4	1.4	1.4		2.5
Freeways (185)													
Number of observations	49	25	16	13	79	25	125	22	128		61	72	
% of samples above detection	100	100	100	100	87.3	96.0	96.8	95.5	99.2		55.7	50.0	
Median	1700	17000	50000	1900	1.07	0.28	2.0	0.20	0.25		2.4	1.4	
Coefficient of variation	2.0	1.2	1.5	2.2	1.3	1.2	1.4	2.1	1.8		0.7	2.0	
Open Space (68)													
Number of observations	23	22			32	44	45	44	46		19		
% of samples above detection	91.3	90.9			18.8	84.1	71.1	79.6	84.8		31.6		
Median	7200	24900			0.18	0.59	0.74	0.13	0.31		4.0		
Coefficient of variation	1.1	1.0			1.24	0.9	0.9	0.9	3.5		0.4		

Table 2.4: Summary of MS4 Stormwater Outfall Data from National Stormwater Quality Database NSQD, version 1.1 (Maestre and Pitt, 2005) (continued)

	Cd, total (µg/L)	Cd, filtered (µg/L)	Cr, total (µg/L)	Cr, filtered (µg/L)	Cu, total (µg/L)	Cu, filtered (µg/L)	Pb, total (µg/L)	Pb, filtered (µg/L)	Hg, total (µg/L)	Ni, total (µg/l)	Ni, filtered (µg/L)	Zn, total (µg/L)	Zn, filtererd (µg/L)
Overall Summary (3765)													
Number of observations	2574	389	1598	261	2722	411	2949	446	1014	1430	246	3007	381
% of samples above detection	40.6	30.3	70.2	60.5	87.4	83	77.7	49.8	10.2	59.8	64.2	96.6	96.3
Median	1.0	0.50	7.0	2.1	16	8.0	17.0	3.0	0.20	8.0	4.0	116	52
Coefficient of variation	3.7	1.1	1.5	0.7	2.2	1.6	1.8	2.0	2.5	1.2	1.5	3.3	3.9
Residential (1042)													
Number of observations	695		404		771	90	762	108	275	392	25	784	87
% of samples above detection	31.1		53.2		83.1	63.3	69.4	33.3	6.9	44.1	44.0	96.2	89.7
Median	0.5		4.5		12	7.0	12.0	3.0	0.20	5.6	2.0	73	31.5
Coefficient of variation	3.4		1.2		1.8	2.0	1.9	1.9	0.9	1.2	0.5	1.3	0.8
Commercial (527)													
Number of observations	379	47	257	27	408	48	399	59	170	242	23	414	49
% of samples above detection	41.7	23.4	60.7	40.7	92.9	79.2	85.5	52.5	6.5	60.3	47.8	99.0	100
Median	0.96	0.30	6.0	2.0	17	7.57	18.0	5.0	0.20	7.0	3.0	150	59
Coefficient of variation	2.7	1.3	1.3	0.6	1.5	0.8	1.6	1.6	0.8	1.2	0.8	1.2	1.4
Industrial (566)													
Number of observations	435	42	250	36	455	42	452	51	199	237	36	473	42
% of samples above detection	49.0	54.8	72.0	55.6	88.6	90.5	75.0	52.9	13.9	61.6	58.3	98.9	95.2
Median	2.0	0.60	12.0	3.0	20.8	8.0	24.9	5.0	0.20	14.0	5.0	199	112
Coefficient of variation	2.2	1.1	1.2	0.7	2.0	0.7	1.9	1.6	2.7	1.0	1.4	1.5	3.6
Freeways (185)													
Number of observations	95	114	76	101	97	130	107	126		99	95	93	105
% of samples above detection	71.6	26.3	98.7	78.2	99.0	99.2	100	50.0		89.9	67.4	96.8	99.1
Median	1.0	0.68	8.3	2.3	34.7	10.9	25	1.8		9.0	4.0	200	51
Coefficient of variation	0.9	1.0	0.7	0.7	1.0	1.5	1.5	1.7		0.9	1.4	1.0	1.9
Open Space (68)													
Number of observations	38		36		39		45					45	
% of samples above detection	55.3		36.1		74.4		42.2					71.1	
Median	0.38		5.4		10		10.0					40	
Coefficient of variation	1.9		1.7		2.0		1.7					1.3	

2.3 Factors Effecting Catchbasin Efficiency

Catchbasin performance has been extensively investigated in the US. For example, Lager *et al.* (1977) conducted an early EPA-funded study to investigate sedimentation in catchbasins, and to develop effective designs. Pitt (1979) undertook long-duration sediment mobility tests using a retro-fitted "idealized" catchbasin based on Lager *et al*.'s 1977 design. These studies showed that the sediment that accumulated in catchbasins was large (in size) in comparison with the solids found in storm runoff and the sediments were not very mobile once trapped in the catchbasin sumps.

Catchbasins, simple inlets, manholes, and sewer sediment accumulations were monitored at more than 200 locations for three years in Bellevue, Washington, at two mixed residential and commercial study areas (Pitt, 1985). A few atypical locations were affected by erosion of the sediment from steep hillsides adjacent to the storm sewer inlets. The sewer and catchbasin sediments had a much smaller median particle size than the street dirt, but the particles were much larger than those generally found at stormwater outfalls. Hence, catchbasins are effective at removing the largest particles that are washed from surfaces during rainfall, preventing them from being deposited in downstream sewerage and in the receiving waters. If the catchbasins are full, they cannot remove the particles from the runoff. Cleaning catchbasins twice a year was found to be most effective in the Washington study. This cleaning schedule was found to reduce the annual discharges of total solids and lead by between 10 and 25 %, and COD, total Kjeldahl nitrogen, total phosphorus, and zinc by between 5 and 10 % (Pitt, 1985).

Butler and Karanuratne (1995) reported particle sizes trapped in gully pot sumps using UK and German data. The median particle size of the sump particles was shown to be between about 300 and 3000 μm, with less than 10 % of the particles smaller than 100 μm, a typical near-upper limit of particles found in stormwater. Relatively few pollutants are associated with these coarser solids found in the sumps, compared with the finer particles. Memon and Butler (2002) investigated the transformation processes in gully pots in simulated storm drainage conditions, and concluded that conditions approximated to those assumed in conventional batch reactors at treatment plants.

These studies found that the major factors that affect the efficiency of catchbasins to collect debris are

- Catchbasin hydrology
- Catchbasin hydraulics
- Solids characteristics

2.3.1 Catchbasin Hydrology

The size and character of the drainage area leading to a catchbasin affects the amount and rate of runoff entering the catchbasin. Larger flows decrease the catchbasin trapping efficiency. The runoff amount and rate entering a catchbasin is dependent on many features of the drainage area, including drainage area size, soil characteristics, impervious area types and amounts, slopes, flow path (surface sheet flow and gutter flow characteristics), plus many storm characteristics (especially rain intensities for short time periods), etc. Therefore, defining a typical hydrograph for catchbasin evaluation purposes would be unrealistic (John A. Lager, et al. 1977). However, the drainage areas for an inlet are mostly dependent on land use for a city, which incorporates most of the drainage area characteristics and level of service objectives.

2.3.2 Catchbasin Hydraulics

The hydraulics of the catchbasin are mostly determined by the geometric configuration of the device. The usual geometric configuration is 6 ft deep barrel with a 4 ft diameter and a top grated opening. The sump should be at least 3 ft deep. The hydraulics of such a system are flow defined through the top entrance and then to the storage in the barrel and finally outflow through the outlet pipe. The water permanently fills the sump up to the outlet, offering scour protection to the sediment. When the outlet is submerged, the storage capacity (very small) depends on the hydraulic gradient between the head in the barrel and the head at the end of the outlet pipe. (John A. Lager, et al. 1977)

The main factors which control the flow through the catchbasin are the geometric configuration of the top entrance, the capacity of catchbasin, the elevation, slope, and the entrance geometry of the outlet conduit. Figure 2.3 compares the flow capacities for different geometric top entrance configurations.

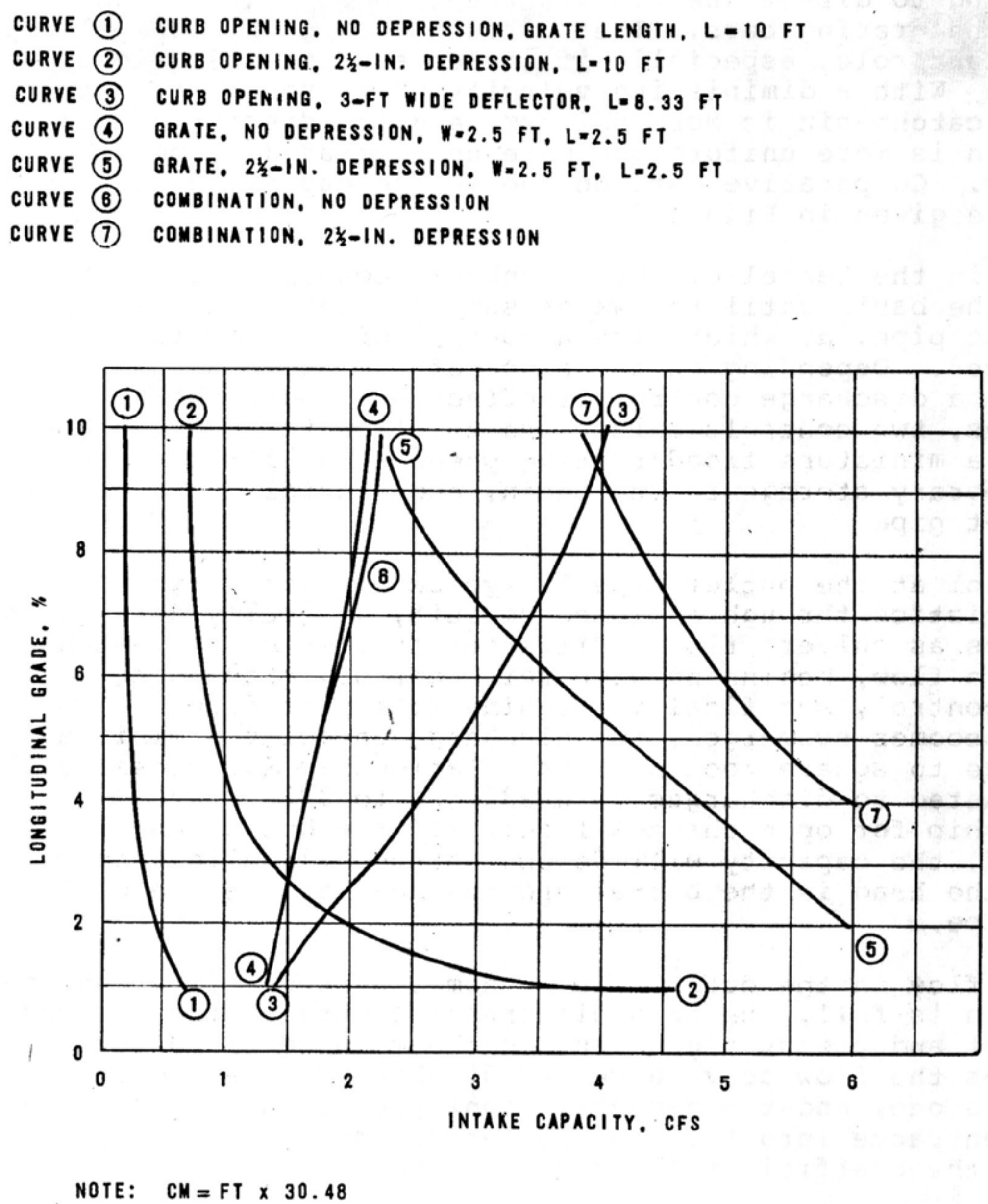

Figure 2.3: Comparison of inlets: Intake capacity at 95% capture of gutter flow: Manning's n = 0.013; cross slope = 0.0417 ft/ft (Design of Stormwater Inlets, John Hopkins University, Dept. of Sanitary Engineering and Water Resources, 1956).

2.3.3 Characterization of Litter and Floatables in Storm Drainage

The report titled *The Removal of Urban Litter from Stormwater Conduits and Streams* (Armitage, *et al.* 2000a and 2000b) noted that little data was available on the nature and quantity of litter in stormwater drainage systems (Marais, *et al.* 2001). Armitage and Rooseboom (2000a) demonstrated that large quantities of litter are being transported in South African stormwater runoff, and that the amount of litter produced was related to land use, vegetation, the level of street cleaning, and type of rainfall. The benefits of litter reduction were documented using their work in Australia and New Zealand, and design equations for sizing litter traps were proposed (Armitage and Rooseboom 2000b). The Council for Scientific and Industrial Research estimated in 1991 that 780,000 tonnes of waste a year entered the drainage systems of South Africa.

The Solids Transport and Deposition Study (STDS) characterized the rates and patterns of solids transfer to, and the collection within, stormwater drain inlets located along Caltrans highway facilities (Quasebarth, *et al*. 2001). The primary objective was to determine if certain distinguishable site characteristics controlled the transport and deposition of sediment, metals, vegetation, litter, and petroleum hydrocarbons to highway drain inlets. The ANOVA results indicated that the four primary factors (erosion control/sediment loading [vegetation factor], litter management [litter factor], toxic pollutant generation potential [adjacent land use factor], and roadway design [design factor]) likely had little overall control on solids accumulation or metals mass accumulation, although roadway design and litter management were possibly important in some cases.

2.4 Control of Litter and Floatables in Storm Drainage Systems

Because more than 780,000 tonnes of solids is washed into the drainage systems in South Africa, the Water Research Commission of South Africa and the Cape Metropolitan Council funded a four year investigation into the reduction of urban litter in the drainage systems through the development of catchment-specific litter management plans (Armitage, *et al*. 2001). A physical model of the design of litter traps for urban storm sewers was also carried out at the hydraulic laboratories at the Universities of Cape Town and Stellenbosch (Armitage and Rooseboom 2000). They conducted a review of about 50 designs for litter traps which have been suggested for urban drainage systems. A preliminary assessment of the seven most promising trapping structures concluded that three designs, two utilizing declined self-cleaning screens, and the other using suspended screens in tandem with a hydraulically actuated sluice gate, are likely to be the optimal choice in the majority of urban drainage situations in South Africa (Armitage and Rooseboom 2000a and 2000b).

The California Department of Transportation (Caltrans) conducted a 2-year litter management pilot study in the Los Angeles area to investigate the characteristics of highway litter and the effectiveness of stormwater controls for removing the litter (Lippner, *et al*. 2001). Half the catchments were treated with one of five stormwater controls; the others were left alone for comparison. The controls tested were increased street cleaning frequency, increased frequency of manual litter pickup, a modified drain inlet, a bicycle grate inlet, and a litter inlet deflector (LID). Roughly half the freeway stormwater litter was paper, plastic, and Styrofoam. Except for cigarette butts, the origins of most of the litter could not be identified because of its small size. Of the five controls tested, only increased litter pickup and the modified drain inlet demonstrated some apparent reduction of litter in the stormwater runoff, although the data were highly variable.

Some people have suggested annually removing sediment, vegetation, and litter from storm drain inlet vaults to improve the quality of Caltrans runoff before it enters the receiving waters (Dammel, *et al*. 2001; Irgang, *et al.* 2001). In response, Caltrans implemented an annual storm drain inlet inspection and cleaning program in selected urban areas to evaluate if this practice improved stormwater quality. Catchbasins within two of the four drainage areas were cleaned at the beginning of the study, while those within the other two areas were not cleaned. Pollutant concentrations and runoff loadings were compared between the two areas. Fine particle deposits remaining in catchbasins after cleaning could cause higher pollutant concentrations and loadings for several months, when compared to areas where catchbasins were not cleaned.

Caltrans also conducted limited laboratory- and full-scale tests of inserts (Fossil Filter and StreamGuard, plus an oil/water separator) to evaluate their ability to remove trash and debris, suspended solids and oil and grease in stormwater (Othmer, *et al*. 2001 Lau, *et al*. 2001). The results showed some reductions in metals, hydrocarbons, and solids; however, frequent flow bypasses due to clogging required more maintenance than anticipated. The oil/water separator results showed no discernable differences between influent and effluent hydrocarbon concentrations at the low levels measured.

Memon and Butler (2002) used a dynamic model to assess the impact of a series of water management scenarios on the quality of runoff discharged through catchbasins/gully pots. The simulation showed that the catchbasins/gully pots were effective at retaining solids, but they had an almost neutral performance in terms of removing dissolved pollutants. Improved solids retention was predicted if larger sumps with modified shapes were used. Lau and Stenstrom (2002) also conducted limited catchbasin insert tests to determine their ability to remove particulate pollutants, litter, and debris. Laboratory tests with used motor oil showed that the inserts could remove large amounts of oils, if present in large concentrations. Sand particles larger than the insert's screen mesh were completely removed, as expected. Field tests showed that median oil and grease, turbidity and total suspended solids concentrations in stormwater were reduced by 30 to 50%. The inserts were more effective in reducing maximum concentrations than low or median concentrations. Some of the inserts plugged and bypassed stormwater without treatment, but did not cause any surface ponding on the streets.

Grey, *et al.* (1999) examined the role of catchbasins in the CSO floatables control program in New York City. There are approximately 130,000 catchbasins, distributed over 190,000 acres, in New York City. They found that catchbasins were simple and very effective in controlling floatable material. The most important aspect of the catchbasins for enhanced floatable control was the presence of a hood covering the catchbasin's outlet. Their research found floatable retention efficiencies of 70 to 90% when the hoods were used. Catchbasin hoods were also very cost-effective, at a cost of about $100 per acre. New York City therefore implemented a catchbasin inspection, mapping, cleaning, and hooding program as part of its CSO control program. Newman, *et al.* (1999) also reported that New York City improved its ability to control one source of floatables to New York Harbor through its “Illegal Dumping Notification Program.” This program takes advantage of coordinated efforts between different department personnel. They found that this program likely will reduce the number of illegal dumping sites by 15%.

Phillips (1999) described how the State Government of Victoria (Australia) provided funding to develop a litter trap (the In-line Litter Separator, or ILLS). The ILLS can be retrofitted into the drainage system downstream of shopping areas for better control of floatables.

Siegel and Novak (1999) reported on the successful use of the microbial larvicide VectoLex CG (R) (*Bacillus sphaericus*) for the control of mosquitoes in 346 tested Illinois catchbasins.

2.4.1 Accumulation of Solids in Catchbasins and Inlets

Valiron and Tabuchi (1992) quoted the results of a study carried out in the North of France, which showed that most of the solids trapped in gully pots were sand-sized with a mean diameter greater than 200 µm. The organic content was very low, ranging from 4 to 8 %. Table 4.1.4 gives the mean grain size distributions measured for five gully pots.

Table 4.1.4 Mean grain size distributions of solids collected in five gully pots (measurements have been carried out after preliminary drying of the solids) (from Valiron and Tabuchi 1992)

grain size (µm)	< 50	50 - 100	100 - 200	200 - 500	500 - 1000	1000 - 2000	> 2000
gully n° 1	24	3	6	11	6	8	42
gully n° 2	24	6	8	18	14	17	13
gully n° 3	5	2	5	16	13	15	44
gully n° 4	15	4	14	29	11	10	17
gully n° 5	56	6	8	12	7	8	4

Butler and Memon (1999) present sediment trapping equations for sediment in gully pots based on simulated wet weather events. Earlier studies (Butler and Karanuratne, 1995) had shown that the sediment trapping performance was dependent on the flow rate passing through the gully pot, and on the particle sizes of the sediment. The depth of sediment in the gully pot had a lesser effect on the capture performance than these other factors. In all cases, decreased flow rates substantially increased the trapping efficiencies and larger particles had substantially greater trapping efficiencies than smaller particles, as expected.

Butler and Clark (1993) studied the build-up rate of sediment in UK roadside gully pots. Figure 4.1.8 shows the build-up of sediments in 19 gully pots in one area, illustrating the temporal variations in sediment depth and the effects of cleaning. They found that for most gullies examined (average drainage area for each of the gully pots was 228 m^2), the build-up rate was fairly constant, at about 18 mm per month.

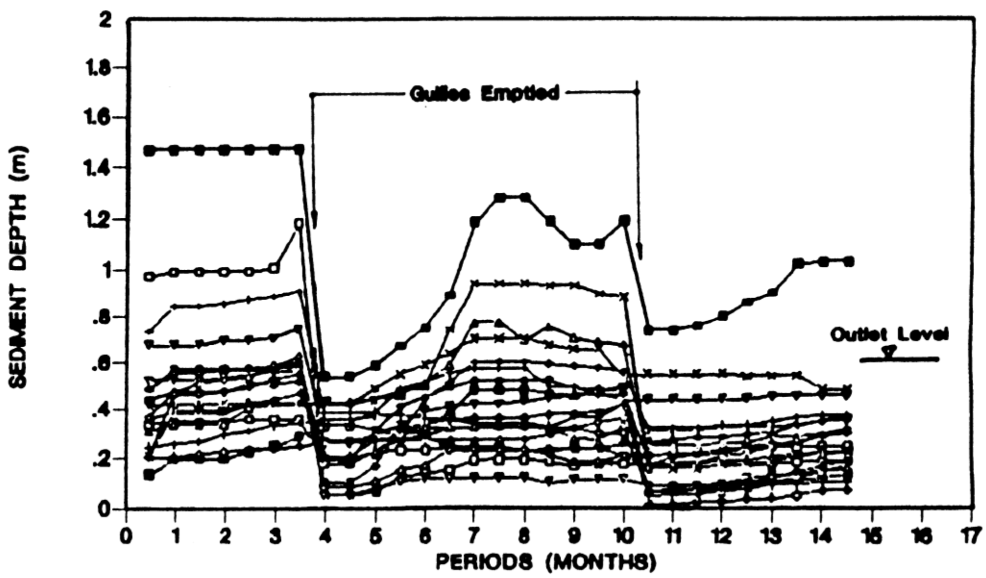

Figure 4.1.8 Gully Pot Sediment Build-up and Removal by Cleaning for 19 pots in the UK (Butler and Clark, 1993)

A study carried out on 4 gully pots in Toulouse, France (Dastugue, *et al*. 1990) has shown that the amount of solids trapped was highly variable and dependent on the type of urbanization and intensity of traffic. The annual mass of solids ranged from 300 kg in the residential or the highway areas up to 700 kg in the dense residential and market place areas (see Table 4.1.5). However, expressed as specific mass per m^2 of upstream catchment, the mass of solids was found to be twice as large in urbanized and highway areas (758 g/m^2/year) as in moderately urbanized areas (351 g/m^2/year).

Table 4.1.5. Mass of solids entering 4 gully pots in Toulouse, France (from Dastugue *et al.*, 1990)

Type of site	Highway	low density residential area	multistorey houses	buildings and market place
Impervious catchment surface (m^2)	630	600	365	875
Annual volume of runoff flowing through the gully pot (m^3/year)	504	484	292	700
Annual mass of solids entering the gully pot (kg/year)	288	313	350	686

An important part of the Bellevue portion of the EPA's Nationwide Urban Runoff Program (NURP) project was the measurement of the sediment accumulating in the inlet structures (Pitt 1985). The storm drainage system inlets were cleaned and surveyed at the beginning of the project. The 207 inlet structures were then surveyed nine times over two years to determine the depth of accumulating material (from December 1979 through January 1981). The first year rate of accumulation was relatively steady (based on 3 observation periods), while the sediment loading remained almost constant during the second year. During the second year, there was about twice as much contaminated sediment in the storm drainage system at any one time as there was on the streets. The flushing of the sewerage sediments out of the drainage systems during rains was not found to be significant during the project period. There was a period of heavy rains in October of 1981 (about 100 mm of rain during a week, very large for Bellevue) during the second year when the accumulated material did not decrease, based on observations made before and after the rain (August 1981 and January 1982). The lack of sediment movement from catchbasin sumps was also observed during earlier tests conducted in San Jose by Pitt (1979). During that study, an idealized catchbasin and sump were constructed based on Lager, *et al.* (1974) and was filled with clean material having the same particle sizes as typical sump material, along with layers of different colored fluorescent tracer beads. During one year of monitoring, freezing core samples were obtained and the sediment layers were studied to determine any flushing and new accumulations of material. The sediment material was found to be very stable, except for a very thin surface layer.

The first year accumulation rates (L/month per inlet) ranged from 1.4 in Lake Hills to 4.8 in Surrey Downs, as shown on Table 3. The catchbasins and inlets had sumps (the catchbasin sumps were somewhat larger), while the manholes were much larger, with more volume available for accumulation of sediment. The stable volume that occurred during the second year was about 60% of the total storage volumes of the catchbasins and inlets (sump volume below the outlet pipe). If the sumps were very shallow, the maximum sediment depth was only about 12 mm, while the deeper sumps had about 150 mm of accumulated sediment. Individual inlet structures had widely varying depths, but the depth below the outlet appeared to the most significant factor affecting the maximum sump volume available. This "scour" depth generally was about 300 mm. If the sumps were deeper, they generally were able to hold more sediment before their equilibrium depth was reached and would therefore require less frequent maintenance. About 100 L/ha/yr accumulated in Surrey Downs, while only about 2/3 of this value accumulated in Lake Hills. Nine of the most heavily loaded catchbasins in the first summer inventory in Surrey Downs were located very near two streets that did not have curbs and had extensive nearby sediment sources (eroding hillsides). These few catchbasins (about 10% of the total catchbasins) accounted for more than half of the total Surrey Downs sediment observed during that survey. They also represented about 70% of the observed increased loadings between the first winter and summer inventories.

Table 3. Accumulation Rate of Sediment in Inlet Structures in Bellevue, WA (Pitt 1985)

	Number of structures		Sediment accumulation (L/month)		Approx. months to stable volume	Stable volume (L)	
Surey Downs (38.0 ha)	total	per ha	per ha	per unit		per ha	per unit
Catchbasins	43	1.1	5.3	4.8	13	68	62
Inlets	27	0.7	2.0	2.8	20	40	57
Manholes	6	0.2	0.8	4.0	19	15	76
Average	76 total	2.0 total	8.1	4.2	15	123 total	62
Lake Hills (40.7 ha)							
Catchbasins	71	1.7	2.4	1.4	18	43	25
Inlets	45	1.1	1.5	1.4	14	22	20
Manholes	15	0.4	1.6	4.0	23	36	90
Average	131 total	3.2 total	5.5	1.7	18	101 total	31

Besides inlet sediment surveys, pipe surveys were also conducted during the study. Very few storm drain pipes in either test area had slopes less than one percent, the assumed critical slope for sediment accumulation. In Lake Hills, the average slope of the 118 pipes surveyed was about 4 percent. Only 7 percent of the Lake Hills pipes had slopes less than 1 percent. The 75 pipes surveyed in Surrey Downs had an average slope of 5 percent, and 12 percent had slopes less than 1 percent. A pipe sediment survey was conducted in October of 1980. Very little sediments were found in the storm drains in either study area. The pipes that had significant sediment were either sloped less than 1-1/2 percent, or located close to a source of sediment. The characteristics of the pipe sediments were similar to the characteristics of the sediment from close-by inlets and catchbasins, indicating a common source, and the eventual movement of the inlet sediments. The volume of sediment found in the Lake Hills pipes was about 1-1/2 m^3, or about 0.04 m^3 per ha, or about 40% of the total sediment in the inlet structures (about 0.1 m^3 per ha stable volume). This was equivalent to about 70 kg of sediment/ha. In Surrey Downs, much more sediment was found in the storm drainage: more than 20 m^3 of sediment was found in the pipes, or about 0.5 m^3/ha or 1,000 kg/ha. Most of this sediment was located in silted-up pipes along 108th St. and Westwood Homes Rd. which were not swept and were close to major sediment sources.

The chemical quality of the captured sediment was also monitored. Tables 4 and 5 show the sediment quality for Surrey Downs inlet structures sampled between January 13 and June 17, 1981. The sediment quality shown on this table is very similar to the street dirt chemical quality that was simultaneously sampled and analyzed. It is interesting to note that the COD values increase with increasing particle sizes, likely corresponding to increasing amounts of organic material in the larger particles. The nutrients are generally constant with size, while the metal concentrations are much higher for the smaller particles, as expected for street dirt. As indicated on the table, the lead values were likely much higher when these samples were taken compared to current conditions. Current outfall lead concentrations are now about 1/10 of the values they were in the early 1980s.

Table 4. Chemical Quality of Bellevue, WA, Inlet Structure Sediment (mg constituent/kg total solids) (Pitt 1985)

Particle Size (µm)	COD	TKN	TP	Pb*	Zn
<63	160,000	2,900	880	1,200	400
61-125	130,000	2,100	690	870	320
125-250	92,000	1,500	630	620	200
250-500	100,000	1,600	610	560	200
500-1,000	140,000	1,600	550	540	200
1,000-2,000	250,000	2,600	930	540	230
2,000-6,350	270,000	2,500	1,100	480	190
>6,350	240,000	2,100	760	290	150

* these lead values are much higher than would be found for current samples due to the decreased use of leaded gasoline since 1981.

Table 5. Annual Calculated Accumulation of Pollutants in Bellevue, WA, Inlet Structures (Pitt 1985)

	Total solids						
	L/ha/yr	kg/ha/yr	COD	TKN	TP	Pb	Zn
			kg/ha/yr	kg/ha/yr	kg/ha/yr	kg/ha/yr	kg/ha/yr
Surrey Downs	96	147	37	0.17	0.25	0.49	0.10
Lake Hills	66	100	7.5	0.07	0.07	0.07	0.02

2.4.2 Catchbasin Sediment and Supernatant Quality and Potential Water Quality Degradation

Catchbasins have been found to be effective in accumulating pollutants associated with coarser runoff solids. Large accumulations in total and suspended solids (up to 45% reduction for low gutter flow rates) were indicated by a number of studies (such as Pitt 1979, Aronson, *et al.* 1983, and Pitt 1985). Pitt (1985) found that catchbasins will accumulate sediments until the sediments reach about 60% of the total sump capacity (or to about 0.3 m under the catchbasin outlet). After that level, the sediment is at an equilibrium, with scour balancing new deposition.

Butler, *et al*. (1995) found that the median particle size of the sump particles was between about 300 and 3,000 μm, with less than 10% of the particles smaller than 100 μm, near the typical upper limit of particles found in stormwater. Catchbasin sumps therefore trap the largest particles that are flowing in the water, and allow the more contaminated finer particles to flow through the inlet structure.

Pitt (1985) statistically compared catchbasin supernatant with outfall water quality and did not detect any significant differences. However, Butler, *et al.* (1995) investigated gully pot supernatant water and concluded that it may contribute to the more greatly polluted first flush of stormwater reported for some locations. Specific problems have been associated with the anaerobic conditions that rapidly form in the supernatant water during dry weather, causing the release of oxygen demanding material, ammonium, and possible sulfides. These anaerobic conditions also affect the bioavailability of the heavy metals in the flushed water.

2.5 Recommended Catchbasin Designs

John A. Lager, et al. (1974) conducted hydraulic modeling analyses on catchbasins at varying flow rates, sump depth, catchbasin height, and with varying outlet pipe diameters to obtain the best catchbasin design. Figure 2.4 shows the variables that were tested and Figure 2.5 shows the recommended catchbasin design. By using a test stimulant, the recommended design was tested for particle removal efficiency. They concluded that if properly designed and maintained, catchbasins can be very efficient and the removal is high over a wide range of flow conditions for larger and heavy particles, although the removal efficiencies are very sensitive to particle size and specific gravity. The storage basin size is the key control and the efficiency improves with increasing depth.

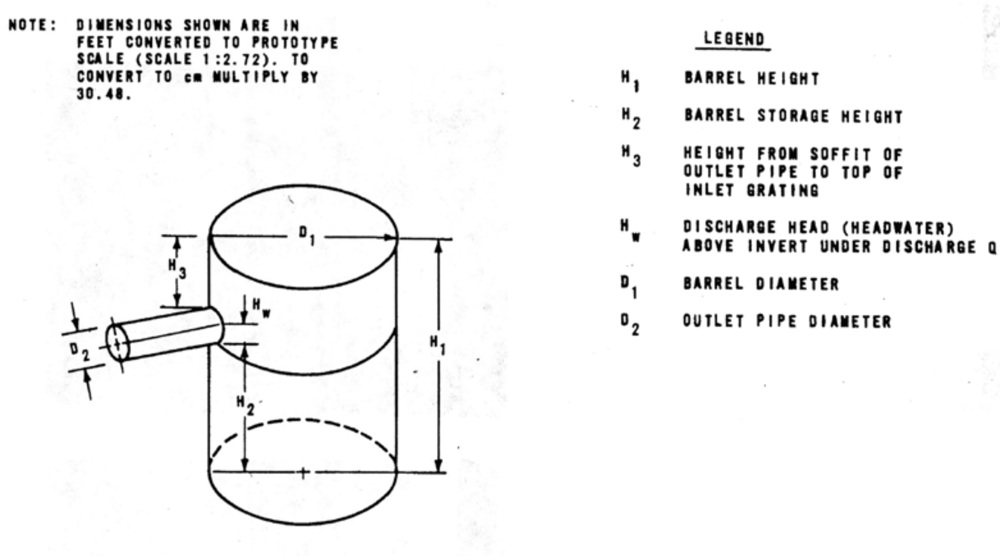

Figure 2.5: Different variables that were tested

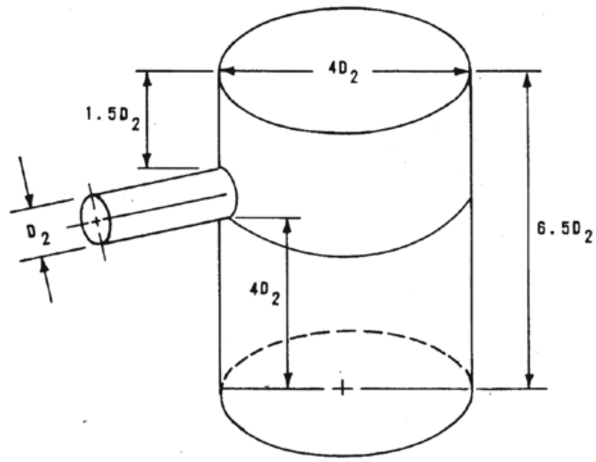

Figure 2.6: Recommended catchbasin design.

Table 2.6: Percentage of material retained in catchbasin for individual storms (John A. Lager, et al 1977)

Constituents	Probable % Retained	
	Worst	Best
Total Solids	42.1	75
Volatile Solids	15.2	25.5
BOD_5	15.5	26.6
COD	7.5	14.1
Kjeldahl Nitrogen	14.6	27.4
Nitrates	9.5	17.1
Phosphates	2.3	6
Total Heavy Metals	37.4	64.4

Total Pesticides	13.6	29.7

During actual field tests, however, catchbasins seldom show such high levels of control as indicated in this table.

2.6 Catchbasin Inserts and Performance Evaluation

Catchbasin inserts could be defined as "a device installed underneath a catchbasin inlet to treat stormwater through filtration, settling, absorption, adsorption, or a combination of these mechanisms." Catchbasins have various shapes and configurations, and inserts differ by the type of filter media used. Hence it is important to evaluate the performance of the insert device so that the best available insert could be selected. The Interjurisdictional Catchbasin Insert Committee (CBIC), King county, Port of Seattle (1995); evaluated three different catchbasin inserts for pollutant removal efficiency, hydraulic capacity and maintenance requirements. The study found insignificant removals of total suspended silt and clay sized particles, total or dissolved Pb, Cu, and Zn, and total phosphorous. The report concluded that the reduction of silt and clay-sized particles was never more than 20 mg/L, and for most sampling periods, the removal efficiency was zero. From visual observations and particle size distributions, the report concluded that the inserts were able to trap the coarsest particles that are found in street runoff. Under usual sump maintenance conditions, the committee saw no data evidence to claim that the inserts can reduce end of pipe concentrations, however they believed inserts would increase the maintenance cycle by providing additional sediment storage.

The inserts accumulated significant amounts of sand and coarse particles, the particle size distribution which matches reported street dirt by Sartor and Boyd (1972) and Pitt (1985).

William T. Leif (1998), of Snohomish County Department of Public Works, Everett, WA has evaluated two different commercially available inserts and rectangular catchbasins. He conducted a total of 54 sediment removal tests with 14 configurations. The variables for the configurations were influent flow path, type of catchbasin insert (including “none”), catchbasin sump depth, sediment size and flow rate. The catchbasin used for the experiment had a plan view dimensions of 24 inches by 30 inches, similar to the American Public Works Association Type I Catchbasin. The test catchbasin was a high density polyethylene tank (HDPE).

Table 2.8: Test System Configurations (William T. Leif, 1998)

Configuration #	**Insert Type**	**Influent Config.**	**Sump Depth**	**Sand Size (1)**	**Flow Rate (GPM)**	**Number of Tests**
1	none	pipe	19"	medium	25	5
2	none	top	19"	medium	25	5
3	sock	top	19"	medium	25	5
4	box	top	19"	medium	25	5
5	none	pipe	19"	fine	25	2
6	none	pipe	9"	medium	25	5
7	none	pipe	9"	fine	25	2
8	none	pipe	3"	medium	25	5
9	none	pipe	3"	medium	50	3
10	none	pipe	3"	medium	70	1
11	none	pipe	3"	medium	100	5
12	none	pipe	3"	fine	100	3
13	none	pipe	9"	medium	100	5
14	none	pipe	19"	medium	100	3

(1) medium sand = 250 µm to 710 µm; fine sand = 125 µm to 250 µm

The Sock insert is a tube of nonwoven geotextile fabric shut at one end, containing strips of polypropylene absorbent material. The second insert tested was a box insert that was a stainless steel structure with three removable sections and screens on top and bottom. The peak flow used for testing was 25 GPM which was computed using the Santa Barbara urban runoff hydrograph method. Higher flow rates were also tested with catchbasins connected in series and flows increasing in downstream units by increments of 25 GPM. Figures 2.9 and 2.10 show the two different experimental setups used for low and high flows. Before starting the test, the sump was filled with Seattle tap water and the test sediment (sand) was washed several times with Seattle tap water to remove any organic matter or adsorbed fine sediment. Each test was conducted for 11 minutes collecting duplicate samples every 1.5minutes.

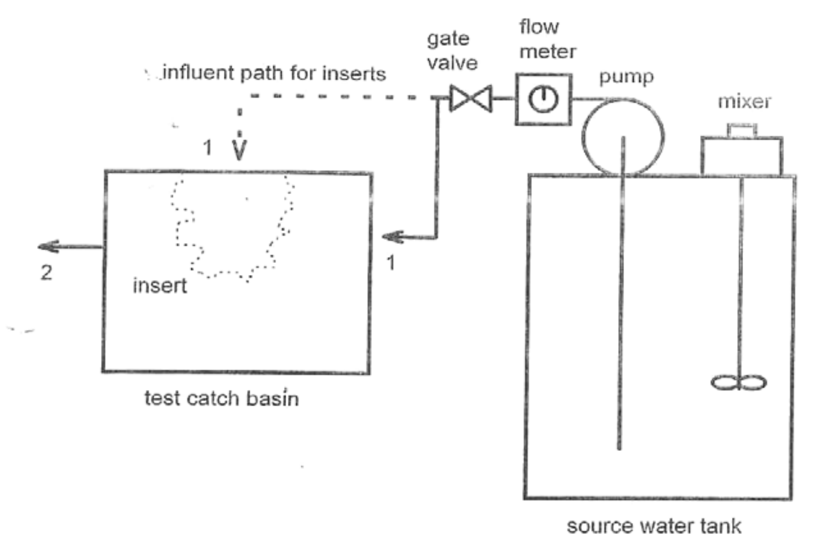

Figure 2.9: Experimental setup for configurations 1-9. The numbers 1 and 2 indicate influent and effluent sampling points respectively. (Leif, 1998)

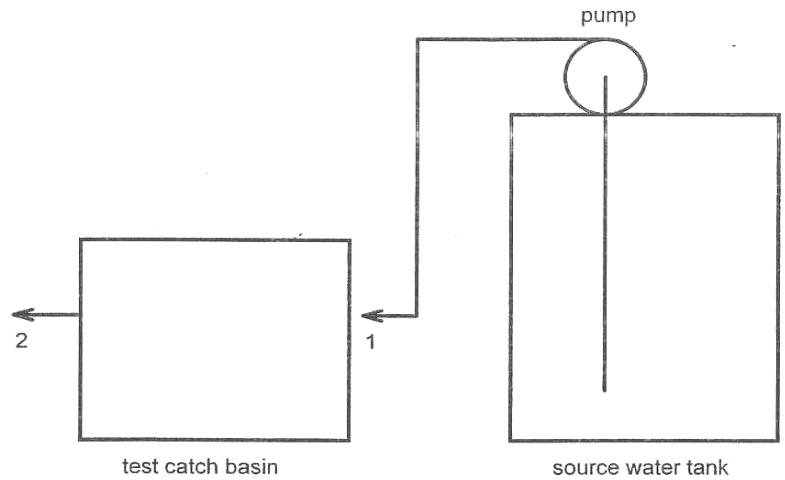

Figure 2.10: Experimental setup for configurations 10-14. The numbers 1 and 2 indicate influent and effluent sampling points respectively. (Leif, 1998)

Leif (1998) found that the catchbasin and the insert with a 19” sump removed more than 90% of the influent sand-sized particles. An 80% removal was also obtained for a 3” sump when using a lower flow rate (25 GPM). Removal efficiencies for sand of over 80% were observed for sump depths of 9” and 19” at a 100 GPM flow rate, as shown in Tables 2.9 and 2.10. By adding a sump, Leif (1998) achieved more than 90% sand removal rates using both the sock and box catchbasin inserts, which were considered to be ineffective by the CBIC (1995) committee. Medium sand removal variability with flow rate and sump depth is shown in Figure 2.11.

Table 2.9: Comparison of removal efficiencies among 19” sumps and inserts, Lief (1998)

Config #	Configuration	Removal Efficiency (RE) (%)					Avg RE
1	no insert; Pipe influent, 19” sump, medium sand	100	99	99	94	94	97
2	no insert; top influent, 19” sump, medium sand	95	98	88	82	97	92
3	sock insert; top influent, 19” sump, medium sand	98	99	99	99	90	97
4	Box insert; top influent, 19” sump, medium sand	98	97	99	77	94	93

Various inserts into gullies or catchbasins are available. These comprise inserts which filter or trap particles. Limited information is available concerning the performance of filter fabrics in removing stormwater pollutants. During controlled laboratory tests using a wide variety of available woven and non-woven filter fabrics, Clark and Pitt (1999) found that the filters provided moderate reductions (about 50 %) in suspended solids and COD. However, the filter fabrics can only withstand very thin accumulations of sediment before they clog. The maximum sediment thickness on the fabrics before clogging was only about 1 to 2 mm (the maximum sediment loading was about 3.8 kg sediment per m^2 of fabric).

Table 2.10: Removal efficiencies for medium sand at different sump depths and flow rates

Config #	Configuration	Removal Efficiency (RE) (%)					Avg RE
1	19” sump, 25 gpm	100	99	99	94	94	97
2*	19” sump, 25 gpm	95	98	88	82	97	92
6	9” sump, 25 gpm	97	98	95	98	98	97
8	3” sump, 25 gpm	88	88	91	93	95	91
9	3” sump, 50 gpm	79	85				82
10	3” sump, 70 gpm	80					--
11	3” sump, 100 gpm	71	42	82	69	64	66
13	9” sump, 100 gpm	85	80	78	83	81	81
14	19” sump, 100 gpm	75	77	80			77

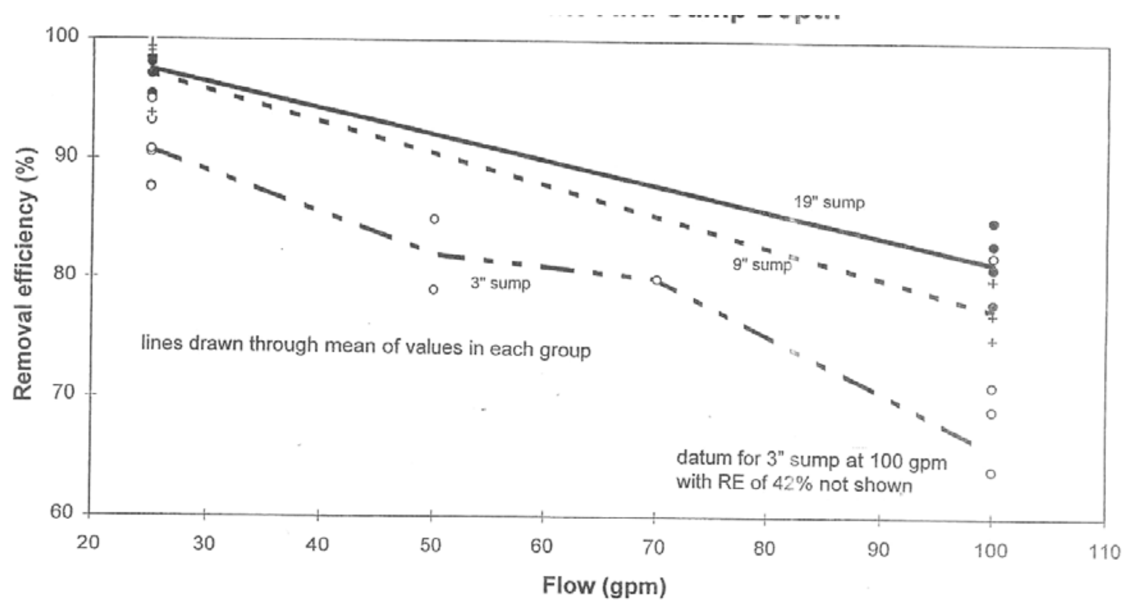

Figure 2.11: Medium sand removal variability with flow rate and sump depth (Leif, 1998)

The basket inserts used in Germany and the large stainless steel perforated trays used in Texas, could provide some benefit: direct water quality improvements would be minimal, but litter capture may be substantial. The Texan devices are cleaned frequently, typically after every storm, as they are located in an area of major litter production, and prevention of litter fouling to the receiving water bodies is a major local environmental and aesthetic issue. Similarly, the German inserts are also emptied frequently because they are relatively small (about 5 litres capacity) and would otherwise clog or create a nuisance.

Pitt and Field (1998) evaluated three storm drain inlet devices in Stafford Township, New Jersey. A conventional catchbasin with sump, and two other inlet devices with filters were tested for performance during actual storm conditions. A sump was installed in the bottom of a traditional inlet by excavation and placing a 36 inch concrete pipe on end (bottom sealed). The outlet pipe was reduced to 8 inches and the sump depth was 36 inches, close to the recommended Lager and Smith (1974) dimensions. One filtration unit retrofitted into an existing inlet was a filter fabric device having dual horizontal trays, each containing 0.1 m^2 of filter fabric. When the top filter clogs, stormwater overflows into a secondary chamber where a similar tray is used. When the lower tray clogs, the unit bypasses the filter fabric. According to the manufacturer, the unit had a treatment capacity of 300 gallons per minute. The third unit tested was a coarse filter which was also retrofitted into an existing stormdrain inlet. This unit was sealed along the bottom and sides on the outlet side, forcing the water through the unit before it could be discharged. This filter was basically designed to filter out debris, leaves, and grass clippings from stormwater.

The test location was a residential area and 12 inlet and effluent paired samples were collected and composited during 12 different rains. The analyses were conducted on filtered and unfiltered fractions of the samples. A wide range of toxicants and conventional pollutants were evaluated, using extremely low detection limits of about 1 to 10 µg/L. The constituents that were analyzed included heavy metals and organics (phenols, PAHs, chlorinated pesticides), particle size distributions using a Coulter Counter Multisizer II, COD, major ions, nutrients, suspended solids, turbidity, color, pH, and conductivity. The results of this test are summarized in Tables 2.7 and 2.8.

Table 2.7: Storm Drain Inlet Device Performance Summary for Selected Pollutants

(Percent Reduction and statistical Probability that Difference is Random) Pitt, et al, 1998

Pollutant	**Catchbasin with Sump % Reduction (p)**	**Coarse Screen Unit % Reduction (p)**	**Filter Fabric Unit % Reduction (p)**
Total Solids	22 (**0.03**)	-28 (**0.014**)	5.6 (0.28)
Dissolved Solids	8.3 (0.68)	-16 (0.13)	3.4 (0.32)
Suspended Solids	32 (**0.0098**)	-56 (**0.054**)	8.1 (0.70)
Volatile Total Solids	6.3 (0.62)	-40 (**0.049**)	0.0 (0.95)
Volatile Dissolved Solids	6.8 (0.77)	-21 (0.32)	4.4 (0.97)
Volatile Suspended Solids	34 (0.43)	-42 (0.55)	-8.3 (1.00)
Toxicity - unfiltered	7.8 (0.91)	-33 (0.15)	18 (0.20)
Toxicity - filtered	1.6 (0.92)	-2.9 (0.57)	-18 (0.62)
Turbidity - unfiltered	38 (**0.019**)	-6.6 (0.30)	0.95 (0.32)
Turbidity - filtered	34 (0.70)	12 (0.27)	-18 (0.62)
Color - unfiltered	16 (0.083)	-14 (0.15)	-1.1 (0.73)
Color - filtered	24 (**0.052**)	-36 (0.68)	-3.0 (0.85)
Conductivity - unfiltered	-11 (0.084)	-14 (**0.052**)	1.2 (0.91)
pH - unfiltered	0.2 (0.64)	-1.0 (0.10)	-0.58 (0.13)
COD - unfiltered	11 (0.47)	-19 (0.58)	-0.91 (0.85)
COD - filtered	-49 (0.42)	-36 (0.41)	19 (0.79)
Nitrate - filtered	-17 (0.12)	-12 (0.28)	6.1 (**0.0024%**)
Ammonium - filtered	-13 (0.84)	5.2 (0.64)	-19 (0.50)

- Non detects

Table 2.8: Mean and Coefficient of Variation of Influent and Effluent samples (Pitt, et al. 1998)

Pollutant		**Catchbasin with Sump**		**Coarse Screen Unit % Reduction (p)**		**Filter Fabric Unit % Reduction (p)**	
		Mean	**COV**	**Mean**	**COV**	**Mean**	**COV**
Total Solids, mg/L	Influent	122	0.54	73	0.94	86.1	0.57
	Effluent	95	0.52	93	0.92	81.2	0.56
Dissolved Solids, mg/L	Influent	48	0.51	51	1.00	46.2	0.71
	Effluent	44	0.49	59	1.08	44.6	0.76
Suspended Solids, mg/L	Influent	75	0.75	22	0.96	39.9	0.85
	Effluent	51	0.62	34	0.79	36.7	0.72
Volatile Total Solids, mg/L	Influent	28	0.52	20	0.85	21.9	0.49
	Effluent	26	0.51	28	0.77	21.9	0.46
Volatile Dissolved Solids, mg/L	Influent	12	0.41	9	0.87	9.58	0.74
	Effluent	11	0.78	11	1.00	9.17	0.66
Volatile Suspended Solids, mg/L	Influent	16	0.90	12	1.03	12	0.86
	Effluent	15	0.59	17	0.83	13	0.59
Turbidity - unfiltered, NTU	Influent	59.9	0.79	6.9	0.94	21.0	0.69
	Effluent	37.1	0.79	7.3	0.78	20.8	0.78
Turbidity - filtered, NTU	Influent	5.0	0.98	0.678	0.77	1.7	0.92
	Effluent	3.3	1.38	0.597	0.59	1.4	0.72
Color - unfiltered, HACH	Influent	62.6	0.54	25.0	0.85	37.3	0.43
	Effluent	52.6	0.56	28.6	0.83	37.7	0.46
Color - filtered, HACH	Influent	26.2	0.43	19.2	1.19	16.9	0.40
	Effluent	19.9	0.40	20.3	1.18	16.4	0.38
Conductivity - unfiltered, µS/cm	Influent	56.3	0.61	79.0	0.93	71.8	0.69
	Effluent	62.6	0.55	90.4	0.99	71.0	0.71

Only the catchbasin had significant removals, while the coarse screening unit actually caused increases in some pollutants (due to stormwater being forced through previously captured debris). The finer filter fabric unit clogged very quickly and almost all of the water bypassed the filters, untreated.

Pitt, et al. (1998) also recommend that a basic catchbasin having an appropriately sized sump with a hooded outlet) should be used in most areas. In almost all full-scale field investigations, this design has been shown to withstand extreme flows with little scouring losses, no significant differences between supernatant water quality and runoff quality, and minimal insect problems. It will trap the bed-load from the stormwater (especially important in areas using sand for traction control) and will trap a low to moderate amount of suspended solids (about 30 to 45% of the annual loadings). The largest size fractions of the sediment in the flowing stormwater will be trapped, in preference to the finer material that has greater amounts of associated pollutants. Their hydraulic capacities are designed using conventional procedures (grating and outlet dimensions), while the sump is designed based on the desired cleaning frequency. Figure 2.7 is this basic recommended configuration.

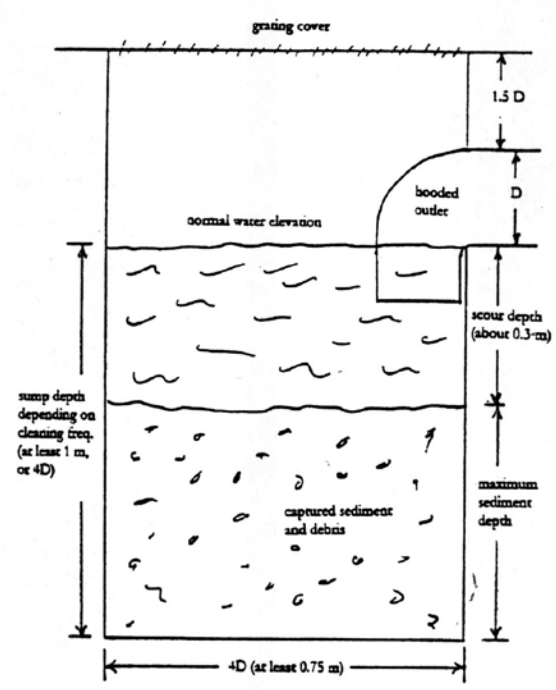

Figure 2.7: Conventional catchbasin with inverted sump and hooded outlet.

Pitt, et al. 1998 recommended an add-on to the above basic configuration which is an adversely-sloped inclined screen covering the outlet of the catchbasin as shown in Figure

2.8. The inclined screen would be a coarse screen to trap the floatables, trash and larger particles of concern. The screen must be stainless steel or aluminum. Zinc materials substantially add zinc to the water. The sides of the screen need to be sealed against the side of the catchbasin and the top edge would extend over the normal water level. Also, a solid top plate would hang over the outlet pipe, providing a slot for overflow and preventing inflowing water from short-circuiting into the outlet side of the screen. This design would help the trapped material to fall into the sump instead of forcing the trapped material through the screen.

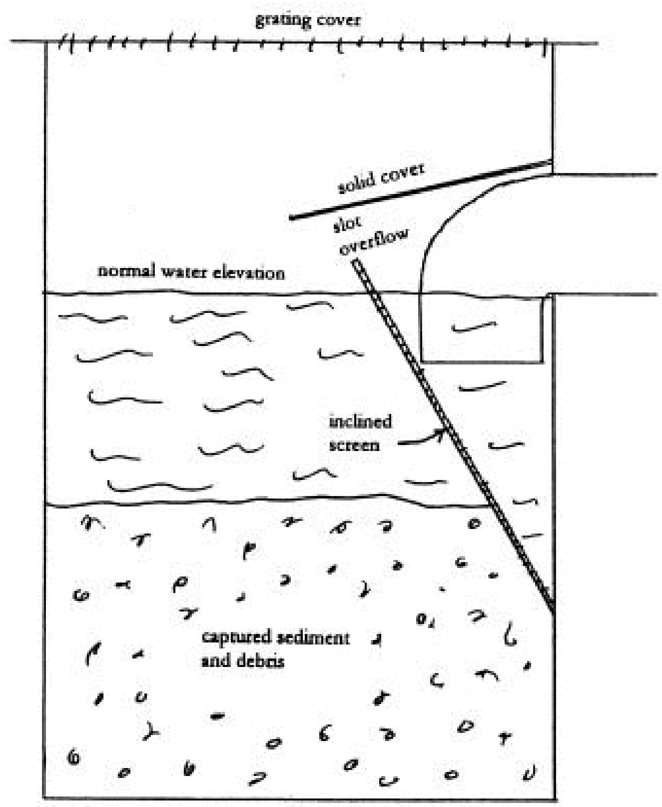

Figure 2.8: Conventional catchbasin with inverted sump, hooded outlet, and inclined screen (Pitt et. al; 1998)

Another option also proposed by Pitt, et al. is the use of bar screens which would be suitable for trapping large litter, such as Styrofoam cups and fast food wrappings also minimizing flow obstructions. The inclined medium screen, described above will trap smaller litter, such as cigarette butts. The configuration of the catchbasin inlet will remain the same with sump and inclined screen as shown above, but it also has a bar screen under the whole area of the inlet grating, especially under large curb openings. In most cases, storm drainage inlets have gratings that have moderate sized openings which prevent large trash from entering the inlet. However, most also have wide openings along the curb face where litter can be washed into the inlet. The bar screen is designed to capture litter that would enter through the wide openings. The bar screen is steeply sloped towards a covered litter trap, preferably in an adjacent chamber. The bars should be spaced no less than ¼ inch and possibly as much as one inch apart, as the objective is to capture large debris. Water passing through the bars would wash the debris towards the covered litter trap, with minimal clogging problems. The covered litter trap should be as large as possible and located above the water level, with drain holes. Since much of the debris would be floatables, any underwater storage volume would have minimal benefit. A nylon net bag, for example, could be inserted into a frame to make litter removal easy and to allow drainage.

Pitt, et al. do not encourage the use of filter fabrics as an integral part of a storm drain inlet as their biggest problem is their likelihood of quickly clogging. Also, their research showed that even though a reduction of 50% is observed in suspended solids and COD, the filter screens could not handle a sediment load greater than about 1 to 2mm thick, corresponding to about 4 kg of sediment per square meter of fabric. Assuming runoff had a suspended solids concentration of 100 mg/L, the maximum loading of stormwater tolerated would be about 40 meters. For a typical application (1 ha paved drainage area to a 1 m^2 filter fabric in an inlet box), only about 5 to 10 mm of runoff could be filtered before absolute clogging.

2.7 The Multi-Chambered Treatment Train (MCTT)

This discussion is excerpted from the first year SBIR2 report (Pitt, *et al*. 2005; *Upflow Filters for the Rapid and Effective Treatment of Stormwater at Critical Source Areas*, Small Business Innovative Research (SBIR) Phase II, Annual Interim Progress Report, Robert Pitt, Ramjee Raghaven, Shirley Clark, Mukesh Pratap, and Uday Khambhammettu, January 9, 2005).

The Multi-Chambered Treatment Train (MCTT) was developed to control toxicants in stormwater from critical source areas. The MCTT is most suitable for use at relatively small areas, about 0.1 to 1 ha in size, such as vehicle service facilities, convenience store parking areas, equipment storage and maintenance areas, and salvage yards. The MCTT is an underground device and is typically sized between 0.5 to 1.5 percent of the paved drainage area. It is comprised of three main sections, an inlet having a conventional catchbasin with litter traps, a main settling chamber having lamella plate separators and oil sorbent pillows, and a final chamber having a mixed sorbent media (usually peat moss and sand). During monitoring, the pilot-scale MCTT provided median reductions of >90% for toxicity, lead, zinc, and most organic toxicants. Suspended solids were reduced by 83% and COD was reduced by 60%. The full-scale tests substantiated these excellent reductions. The information presented in the following summary is based on the results from a series of related projects sponsored by the US EPA (Pitt, *et al.* 1996, Clark and Pitt 1999, Pitt, *et al*. 1999, and Clark 2000).

The main settling chamber provided substantial reductions in total and dissolved toxicity, lead, zinc, certain organic toxicants, SS, COD, turbidity, and color. The sand-peat chamber also provided additional filterable toxicant reductions. However, the catchbasin/grit chamber did not provide any significant improvements in water quality, although it is an important element in reducing maintenance problems by trapping bulk material. Zinc and toxicity are examples where the use of the final chamber was needed to provide high levels of control. Otherwise, it may be tempting to simplify the MCTT by removing the last chamber. Another option would be to remove the main settling chamber and only use the pre-treating capabilities of the catchbasin as a grit chamber before the peat "filtration" chamber (similar to many stormwater filter designs). This option is not recommended because of the short life that the filter would have before it would clog (Clark and Pitt 1999; Clark 2000). The concept of the Upflow Filter was developed in response to simplifying this design for a more suitable and less expensive option for smaller areas or areas that are not as heavily polluted as intended for the MCTT.

2.7.1 Description of the MCTT

Figure 2.9 shows a cross section of the MCTT. The catchbasin functions primarily as a protector for the other two units by removing large, grit-sized material. The settling chamber is the primary treatment chamber for removing settleable solids and associated constituents. The sand-peat filter is for final polishing of the effluent, using a combination of sorption and ion exchange for the removal of soluble pollutants, for example.

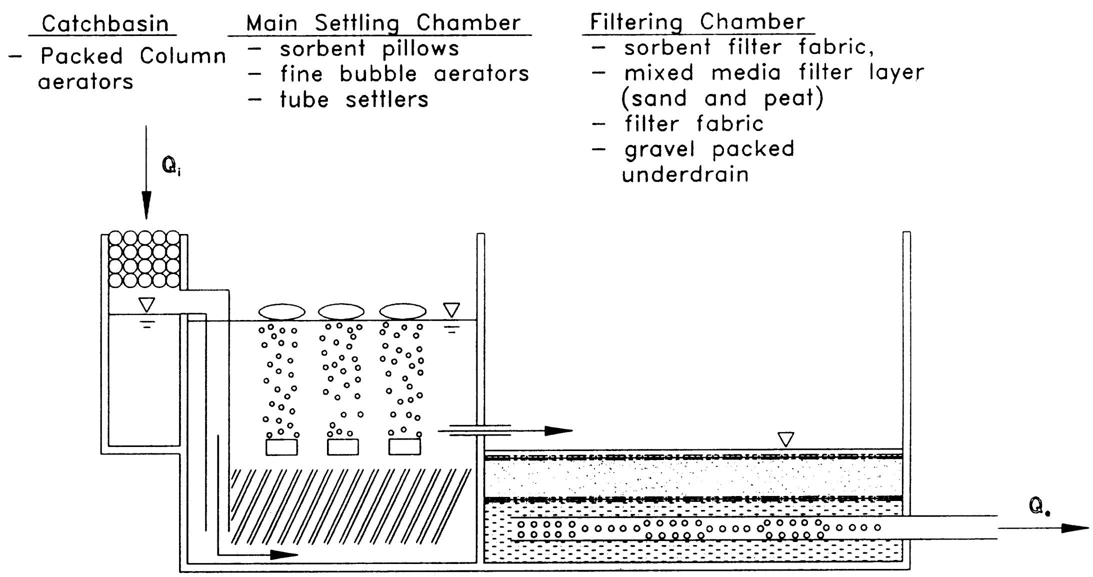

Figure 2.9: MCTT cross section.

2.7.1.1 Catchbasin/grit chamber

The MCTT catchbasin/grit chamber design is based upon a recommended design from previous studies of catchbasins (Lager, *et al.* 1977 and Aronson, *et al.* 1983). This design suggests using a circular catchbasin with the diameter 4 times the diameter of the circular outlet. The outlet is then placed 1.5 times its diameter from the top and 4 times its diameter from the bottom of the catchbasin, thus providing a total depth of 6.5 times the outlet diameter. The size of the MCTT catchbasin is controlled by three factors: the runoff flow rate, the SS concentration in the runoff, and the desired frequency at which the catchbasin will be cleaned so as not to sacrifice efficiency.

2.7.1.2 Main settling chamber

The main settling chamber mimics the completely mixed settling column bench-scale tests previously conducted and uses a hydraulic loading rate (depth to time ratio) for removal estimates. This loading rate is equivalent to the conventional surface overflow rate (SOR), or upflow velocity, for continuous-flow systems, or the ratio of water depth to detention time for static systems. The MCTT can be operated in both modes. If it uses an orifice, to control the settling chamber outflow, then it operates in a similar mode to a conventional wet detention pond and the rate is the upflow velocity (the instantaneous outflow divided by the surface area of the tank). If the outflow is controlled with a float switch and a pump, then it operates as a static system and the hydraulic loading rate is simply the tank depth divided by the settling time before the pump switches on to remove the settled water.

In addition to housing plate or tube settlers, the main settling chamber also contains floating sorbent “pillows” to trap floating oils and a fine bubble aerator that operates during the filling time of the MCTT. Plate settlers (or inclined tubes) increase solids removal by reducing the distance particles travel to the chamber floor and by reducing scour potential. Plate settler theory is described by Davis, *et al.*(1989). The main settling chamber operates much like a settling tank, but with the plate settlers increasing the effective surface area of the tank. The increase in performance is based on the number of plate diagonals crossing the vertical. If the plates are relatively flat and close together, the increase in performance is greater than if the plates are steeper and wider apart. The effective increase is usually about 3 to 5 fold.

The fine bubble aerator serves two functions: to support aerobic conditions in the settling chamber and to provide dissolved air flotation of particles. Aeration was used during the pilot-scale MCTT tests, but was not used during the full-scale Wisconsin or Caltrans MCTT tests. Dissolved air flotation has been utilized in industrial applications and combined sewer overflows (Gupta, *et al*. 1977). The settling time in the main settling chamber typically ranges from 1 to 3 d, and the settling depth typically ranges from 0.6 to 2.7 m (2 to 9 ft). These depth to time ratios provide for excellent particulate (and associate pollutant) removals in the main settling chamber.

2.7.1.3 Filter/ion exchange chamber

The final MCTT chamber is a mixed media filter (sorption/ion exchange) device. It receives water previously treated by the grit and the main settling chambers. The initial designs used a 50/50 mix of sand and peat moss, while the Ruby Garage full-scale MCTT in Milwaukee used a 33/33/33 mixture of sand, peat moss, and granulated activated carbon. The MCTT can be easily modified to contain any mixture of media in the last chamber. However, care must be taken to ensure an adequate hydraulic capacity. As an example, peat moss alone was not effective because it compressed quickly, preventing water from flowing through the media. However, when mixed with sand, the hydraulic capacity was much greater and didn't change rapidly with time.

2.7.2 Summary of MCTT Performance and Suggested Modifications

The pilot- and full-scale test results showed that the MCTT provided substantial reductions in stormwater toxicants (both in particulate and filtered phases) and suspended solids. Increases in color and a slight decrease in pH also occurred during the final treatment step when using peat as part of the filtering/ion-exchange media. The main settling chamber provided substantial reductions in total and dissolved toxicity, lead, zinc, certain organic toxicants, SS, COD, turbidity, and color. The sand-peat chamber also provided additional filterable toxicant reductions. However, the catchbasin/grit chamber did not provide any significant improvements in water quality, although it is an important element in reducing maintenance problems by trapping bulk material.

Zinc and toxicity are examples where the use of the final chamber was needed to provide high levels of control. Otherwise, it may be tempting to simplify the MCTT by removing the last chamber. Another option would be to remove the main settling chamber and only use the pre-treating capabilities of the catchbasin as a grit chamber before the peat "filtration" chamber (similar to many stormwater filter designs). This option is not recommended because of the short life that the filter would have before it would clog from the silt and fine sand in stormwater. In addition, the bench-scale treatability tests conducted during the development of the MCTT (Pitt, *et al.* 1999) showed that a treatment train was needed to provide some redundancy because of frequent variability in sample treatability storm to storm, even for a single sampling site.

Table 2.8: Caltrans Test Results for MCTTs

Constituent	**Average Influent Concentration (mg/L)**	**Average Effluent Concentration (mg/L)**	**Concentration Reduction (%)**
TSS	29.6	6	80
Nitrate	0.42	0.68	-62
TKN	1.27	0.82	35
N Total	1.69	1.50	11
P Total	0.18	0.11	39
Cu Total	0.008	0.005	38
Cu Dissolved	0.004	0.003	25
Pb Total	0.006	0.003	50
Pb Dissolved	0.001*	0.001*	NA
Zn Total	0.086	0.013	85
Zn Dissolved	0.050	0.013	74
TPH-Oil	0.34	0.20*	>41
TPH-Diesel	1.43	0.21	85
Fecal Coliform	973 MPN/100mL	171 MPN/100mL	82

*equals value of reporting limit

Note– TPH and Coliform collected by grab method and may not accurately reflect removal. The concentrations are the mean of the event mean concentrations (EMCs) for the entire monitoring period.

The MCTT operated as intended: it provided very effective reductions for both filtered and particulate stormwater toxicants and SS. Because of its high cost, it may only be suitable for critical source areas where high levels of toxicant reductions are needed, or where runoff is grossly contaminated. Much of the added expense is associated with the underground installation of the MCTT to enable it to be located in areas having little room for alternative stormwater control options. In addition, the pilot-scale and full-scale installations described above were all designed for very high levels of control. In addition, the Caltrans units were greatly oversized to meet the LA County required treatment volume of one inch of runoff. Computer simulations of long-term regional rainfall resulted in a recommended treatment volume of about 1/3 of this size to best balance cost and performance. This earlier research also examined treatability of stormwater toxicants in general, and this information can be used to develop or improve other stormwater treatment devices.

2.8 Summary of Sewerage Inlet Devices as Stormwater Control Practices

The best catchbasin configuration for a specific location would be dependent on site conditions and would probably incorporate a combination of features from several different inlet designs. The basic design should incorporate a catchbasin with a sump, as described by Lager, *et al.* (1977), and an inverted (hooded) outlet. If large enough, catchbasins with sumps have been shown to provide a moderate level of suspended solids reductions in stormwater under a wide range of conditions in many studies in the U.S. and Europe. The use of filter fabrics in catchbasins is not likely to be beneficial because of their rapid clogging from retained sediment and trash. The use of coarser screens in catchbasin inlets is also not likely to result in water quality improvements, based on conventional water pollutant analyses. However, well designed and maintained screens can result in substantial trash and litter reductions. It is important that the screen not trap organic material in the flow path of the stormwater. Tests during research found that stormwater flowing through decomposing leaves degraded the stormwater quality (Pitt, *et al.* 1997). Prior research (Pitt 1979 and 1985) has shown that if most of the trapped material is contained in the catchbasin sump, it is out of the direct flow path and unlikely to be scoured during high flows, or to degrade overlying supernatant water. Storm drainage inlet devices also should not be considered as leaf control options, or used in areas having very heavy trash loadings, unless they can be cleaned after practically every storm.

2.8.1 Optimization of Storm Drain Inlet Structures

The objective should be to use storm drainage inlet devices which have the following performance characteristics:

- not prone to flooding due to clogging with debris;
- control the discharge of settleable solids into the downstream system

- do not encourage stormwater to release any previously captured solids;
- do not have unacceptable hydraulic head loss properties;
- require only inexpensive and infrequent maintenance.

As noted above, the MCTT is relatively expensive, although it is very robust and capable of providing excellent treatment of a wide variety of stormwater pollutants. However, alternative treatment trains could be developed using the same ideas and technologies illustrated in the MCTT. Specifically, our more recent research on filtration started to examine alternative media and ways to reduce clogging. Upflow filtration was examined as a way to accomplish this goal, at the same time as providing a much greater treatment rate. Also, a smaller unit that could be used in areas not grossly contaminated with large amounts of large debris and oils would be much less expensive. The Upflow Filter was therefore conceived to allow many of the treatment train components of the MCTT to be used in a smaller area, while providing a much faster unit area stormwater flow treatment rates.

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CHAPTER 3

UPFLOW FILTER DESIGN AND PERFORMANCE EXPECTATIONS

This chapter describes the UpFlow^{TM} filter that was monitored as part of the EPA sponsored SBIR project conducted at Tuscaloosa, AL. This discussion is excerpted from the first year SBIR2 report (Pitt, *et al*. 2005; *Upflow Filters for the Rapid and Effective Treatment of Stormwater at Critical Source Areas*, Small Business Innovative Research (SBIR) Phase II, Annual Interim Progress Report, Robert Pitt, Ramjee Raghaven, Shirley Clark, Mukesh Pratap, and Uday Khambhammettu, January 9, 2005).

3.1 Upflow Velocity

Linsley and Franzini (1964) stated that in order to get a fairly high percentage removal of particulates, it is necessary that a sedimentation device be properly designed. In an ideal system, particles that do not settle below the bottom of the outlet will pass through the sedimentation device, while particles that do settle below/before the outlet will be retained. The path of any particle is the vector sum of the water velocity (V) passing through the device and the particle settling velocity (v). Therefore, if the water velocity is slow, slowly falling particles can be retained. If the water velocity is fast, then only the heaviest (fastest falling) particles are likely to be retained. The critical ratio of water velocity to particle settling velocity must therefore be equal to the ratio of the sedimentation device length (L) to depth to the bottom of the outlet (D):

$$\frac{V}{v} = \frac{L}{D}$$

as shown on Figure 3.1.

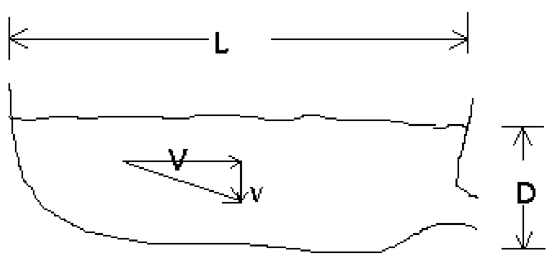


Figure 3.1: Critical velocity and pool dimensions.

The water velocity is equal to the water volume rate (Q, such as measured by cubic feet per second) divided by the pond cross-sectional area (a, or depth times width: DW):

$$V = \frac{Q}{a}$$

or

$$V = \frac{Q}{DW}$$

The outflow flow rate equals the inflow flow rate under steady state conditions, and for most all conditions for small hydrodynamic devices, including especially inlets, as they have minimal hydraulic storage capacity. The critical time period for steady state conditions is the time of travel from the inlet to the outlet in larger devices. During critical portions of a storm, the inflow rate (Q_{in}) will be greater than the outflow rate (Q_{out}) due to freeboard storage, if available. Therefore, the outflow rate controls the water velocity through the device, or is equal to the inflow rate for the smaller devices. By substituting this definition of water velocity into the critical ratio:

$$\frac{Q_{out}}{WDv} = \frac{L}{D}$$

The water depth to the outlet bottom (D) cancels out, leaving:

$$\frac{Q_{out}}{Wv} = L$$

Or

$$\frac{Q_{out}}{v} = LW$$

However, flow path length (L) times device width (W) equals the surface area (A). Substituting leaves:

$$\frac{Q_{out}}{v} = A$$

and the definition of upflow velocity:

$$v = \frac{Q_{out}}{A}$$

where Q_{out} = device outflow rate (cubic feet per second),

A = device surface area (square feet: device length times width), and

v = upflow velocity, or critical particle settling velocity (feet per second).

Therefore, for an ideal sedimentation device, particles having settling velocities less than this upflow velocity will be removed. Only increasing the surface area, or decreasing the outflow rate, will increase settling efficiency. Increasing the depth does lessen the possibility of bottom scour. Deeper devices may also be needed to provide sacrificial storage volumes for sediment between dredging operations. In stormwater inlets, at least one foot is needed below the outlet invert as protection against scour (Pitt 1985). An important feature of the UpFlow^{TM} filter, however, is the prevention of scour-carrying flows ever reaching the outlet, and the complete sump depth up to the bottom of the device is available for sediment storage. Obviously, cleaning of the sump is necessary well before this occurs.

3.2 Particle Settling Velocities

The settling velocities of discrete particles are shown in Figure 3.2, based on Stoke's and Newton's settling relationships. Probably more than 90% of all stormwater particulates are in the 1 to 100 μm range, corresponding to laminar flow conditions, and appropriate for using Stoke's law. This figure also illustrates the effects of different specific gravities on the settling rates. In most cases, stormwater particulates have specific gravities in the range of 1.5 to 2.5. This corresponds to a relatively narrow range of settling rates for a specific particle size. Particle size is much easier to measure than settling rates and can be measured using automated particle sizing equipment (such as a Coulter Counter Multi-Sizer 3). It is important to conduct periodic settling column tests to determine the corresponding specific gravities. If the particle counting equipment is not available, then small scale settling column tests (such as by using 50 cm diameter Teflon™ columns about 0.7 m long) can be used. These settling velocities (or particle sizes) are used with the outflow rate to determine the expected removal rates.

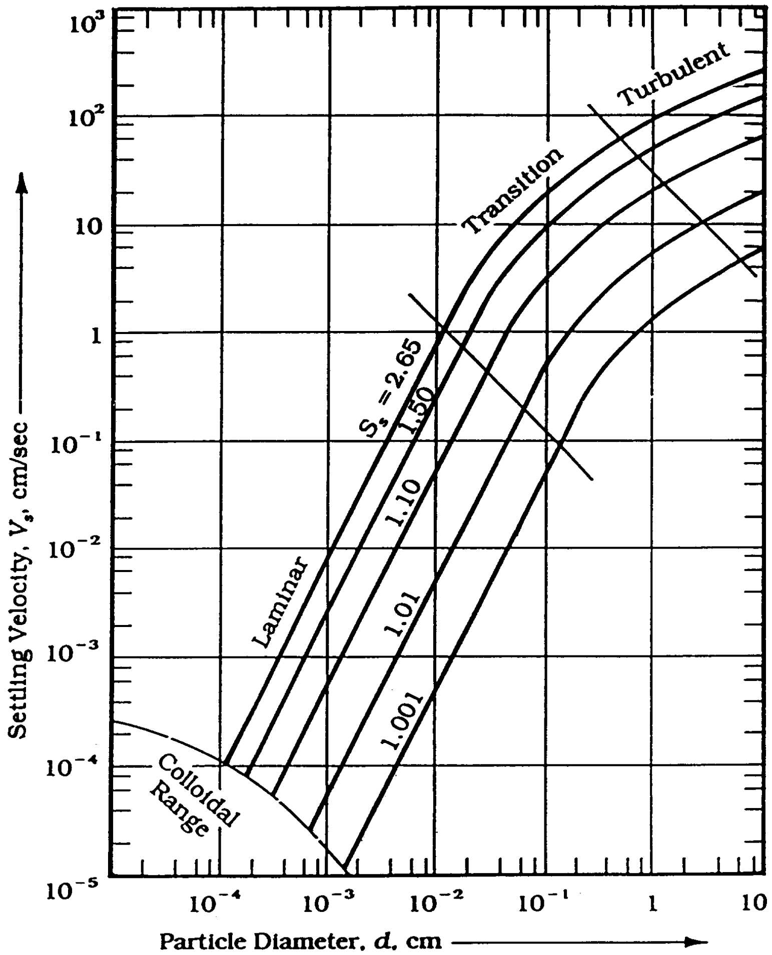


Figure 3.2: Type 1 (discrete) settling of spheres in water at 10 °C (Reynolds 1982).

3.3 Outflow Rates from Discharge Control Devices

An important aspect of predicting the performance of a sedimentation device is to determine the stage-discharge relationship for the device under study. In inlets, the surface area is constant with depth, while the outlet discharge rate changes with head. This relationship (the rating curve) is the outflow rate (expressed in cubic feet per second, or cfs) for different water surface elevations (expressed in feet). Figures 3.3 through 3.4

are approximate rating curves for several common outlet control weir types for water surface elevation ranges up to six feet above the weir inverts (Pitt and Voorhees 19XX). As an example, Figure 3.3 shows six separate curves for different lengths of rectangular weirs (from two to 18 feet wide). At a water surface elevation of 2.5 feet above the bottom of the weir (stage), not the bottom of the device, a three foot wide rectangular weir would discharge about 34 cfs, while a 12 foot wide rectangular weir at this same stage would discharge about 150 cfs. Obviously, the smaller inlet sizes require the use of smaller outlet flow control devices.

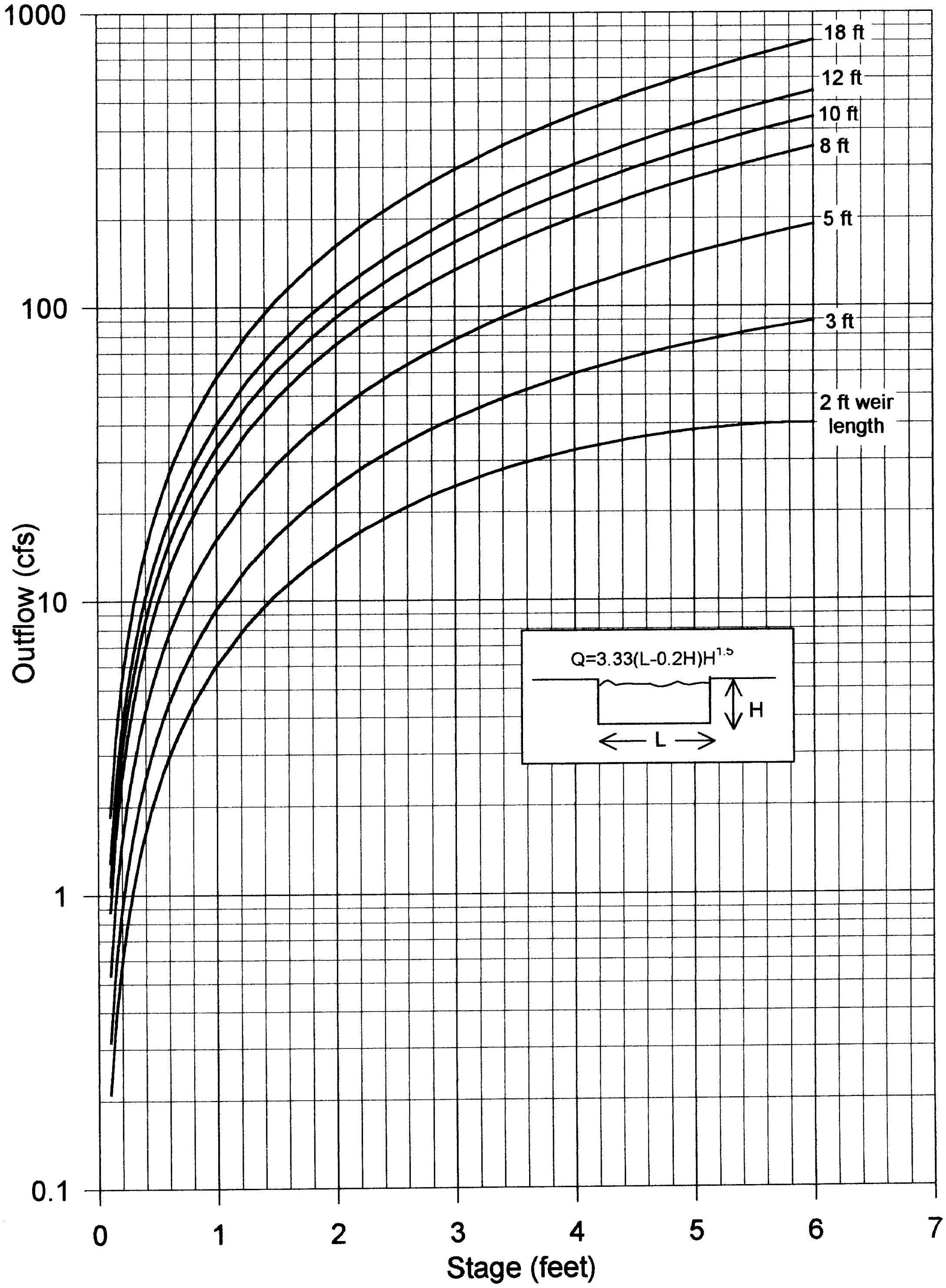


Figure 3.3: Approximate rating curves for rectangular weirs.

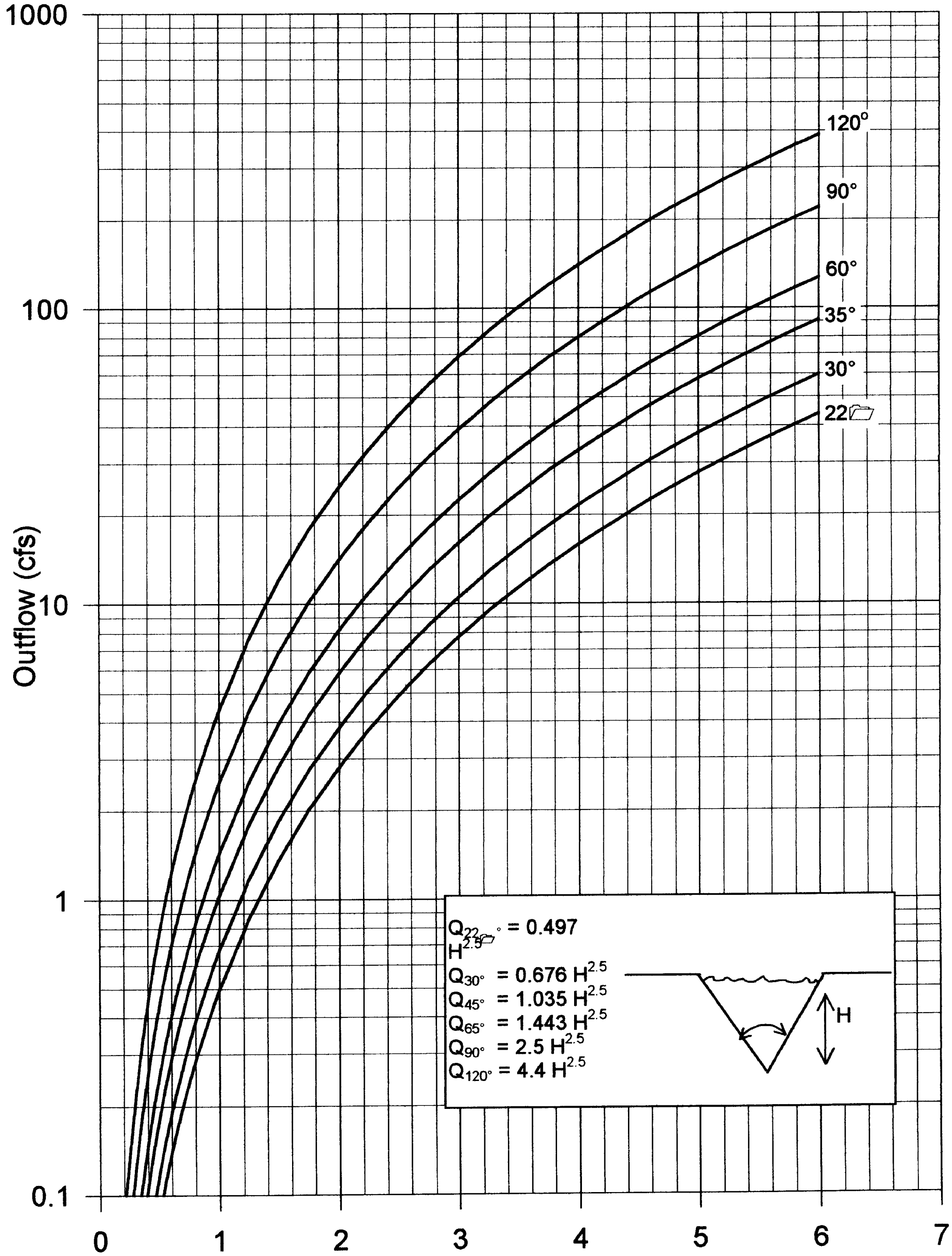


Figure 3.4: Approximate rating curves for V-notch weirs.

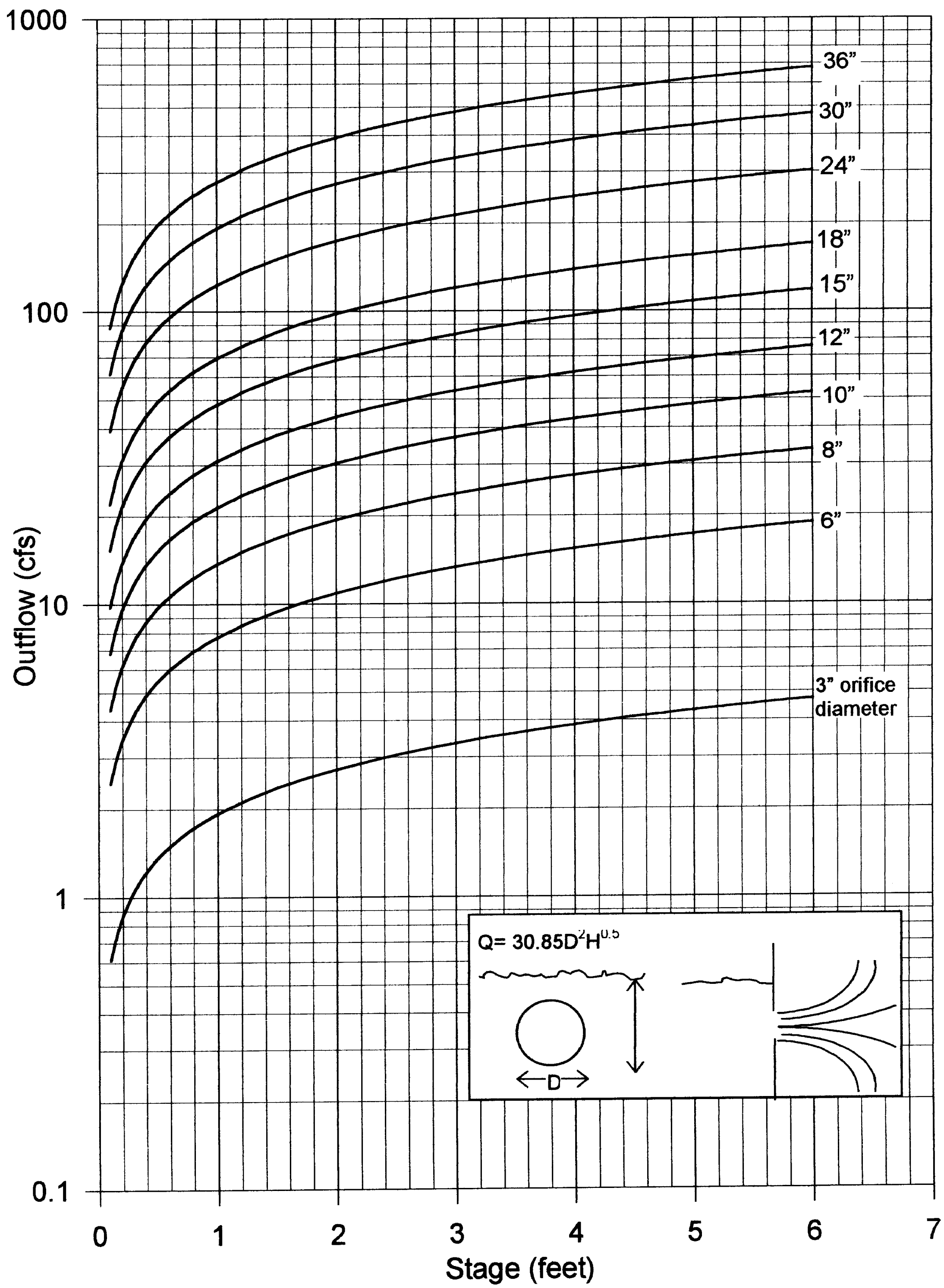


Figure 3.5: Approximate rating curves for orifice discharges.

3.4 Testing of Media and Performance of Upflow Filter

Pollutant removal mechanisms in the UpFlow^{TM} filter include several processes:

- Coarse solids and litter removal in sump and by screens
- Capture of intermediate solids by sedimentation in sump by controlled discharge rates
- Capture of fine solids in primary filtration media
- Sorption and ion-exchange capture of dissolved pollutants in primary and secondary media

The basic removal of solids is therefore dependent on physical sedimentation in the sump, and by filtration in the media. The following discusses these primary removal processes.

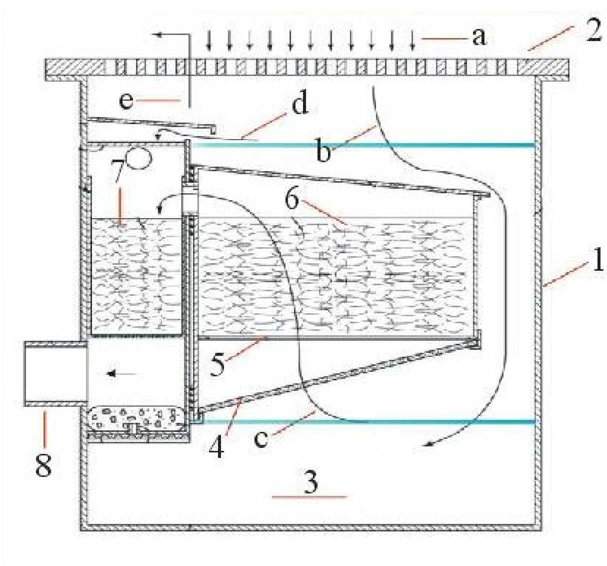


Figure 3.6: Schematic of UpFlow^{TM} filter. 1. Catchbasin, 2. Grate inlet cover, 3. Sump, 4. Coarse Screen, 5. Fine Screen, 6 and 7. Filter Media, 8. Effluent Pipe. a, b and c are showing the filter path in the filter; d and e are showing the overflow path.

3.4.1 Coarse Screening of Litter and Debris

As indicated in the figure 3.6 of the UpFlow™ filter and the photographs of the prototype unit being tested, a coarse screen is fitted on the bottom of the filter unit. This filter is at an angle and the water passes through this screen in an upflow direction. The apertures of the screen in the prototype unit are 5/16 inches in diameter (about 8 mm, or 8,000 µm). Debris larger than this size cannot penetrate the UpFlow™ filter and will be trapped in the catchbasin sump. The overflow slot is protected from floatable washout by an overhang.

The coarse screen on the bottom of the UpFlow™ filter is designed to trap this larger material and prevent it from being transported in the storm drainage system. The adverse angle and upward flow through this screen prevents this material from accumulating on the screen and forcing subsequent water through the trapped material. During previous catchbasin screening tests (Pitt and Field 1998), flows through previously captured material (mostly organic material, such as leaves) were found to degrade the material and increase the discharge of suspended solids and nutrients. This design is intended to minimize that problem. Also, Pitt (1979) found that debris and leaves captured in a catchbasin sump away from the flowing water were permanently trapped and later tests (Pitt 1985) found that the overlying water was not significantly degraded compared to the runoff. Pitt (1985) also found that about one foot of standing water above the debris, at least, was needed to prevent the previously collected material from being scoured during subsequent events. The bottom coarse screen is intended to minimize scour and to completely capture the large debris. In addition, all overflows are also protected with protruding overhangs to minimize the washout of floating debris during bypass conditions. Finally, a time-release natural pesticide can be used in the Spring to Fall in the sump to eliminate any possible mosquito problems in the standing water.

3.4.2 Sediment Capture in Catchbasin Sump

As noted in the above section on upflow velocities and particle settling, particles will be captured in the catchbasin sump as a function of the discharge flow rate and the surface area of the sump. Figure 3.7 is a plot showing the critical particle size (the size at which all larger particles will be trapped), which is equal to the upflow velocity in the catchbasin (the discharge rate divided by the surface area). Many of the contour lines are jagged and a bit uneven in this figure due to small inaccuracies when manually reading the particle sizes from the settling rate curve previously presented. The settling conditions assume a 2.5 specific gravity for the particulates (as indicated during many prior research projects) and 10^{o} C, representing winter conditions, but much cooler than expected during summer conditions in Tuscaloosa, Alabama, the test site for the prototype UpFlow™ filter.

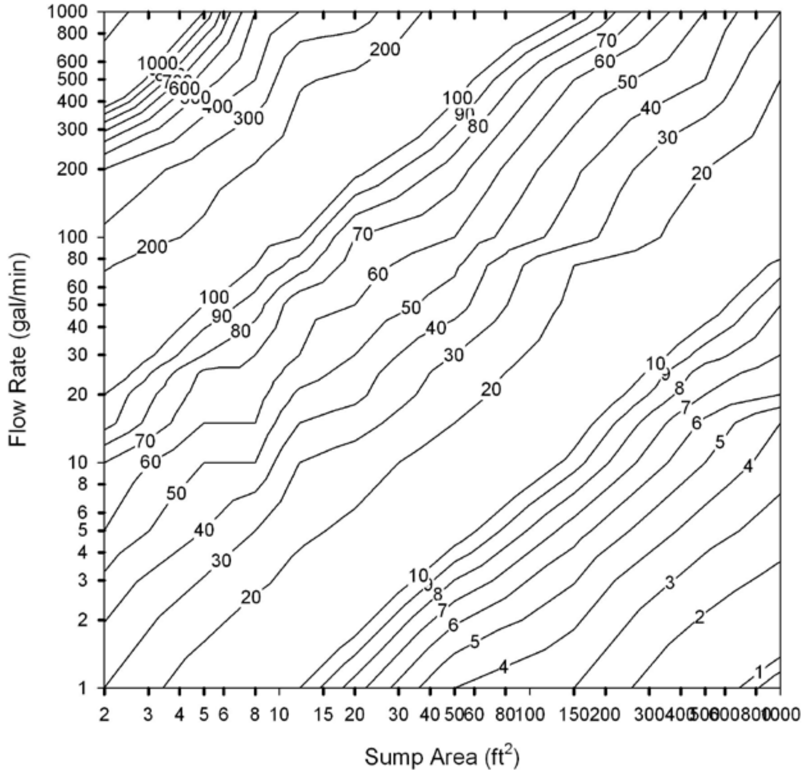


Figure 3.7: Critical particle sizes captured for different sump areas and flow rates.

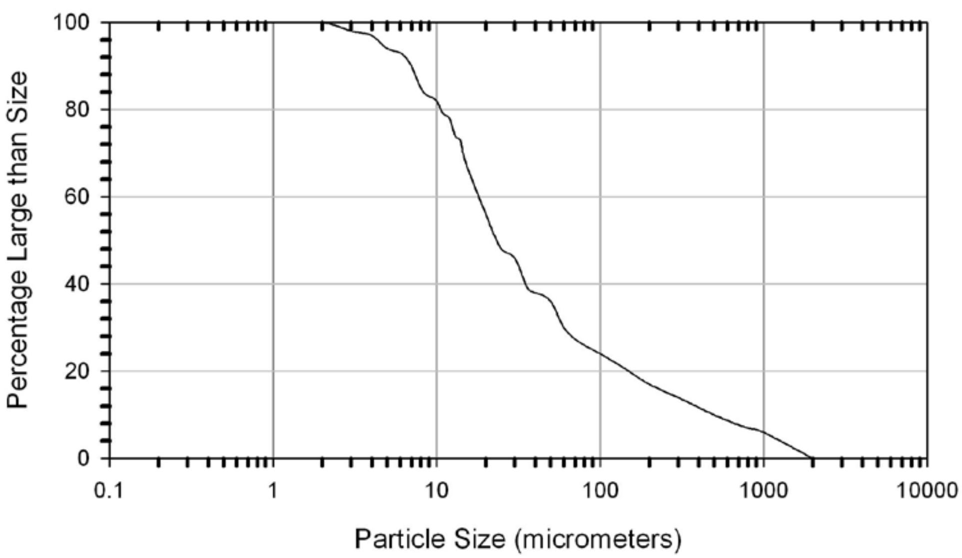


Figure 3.8: Observed particulate matter size distribution in runoff water at Tuscaloosa test site.

3.5 Expected Pollutant Capture Performance of the Upflow Filter

The UpFlow^{TM} filter has numerous pollutant removal mechanisms available, including: coarse screening of litter and debris, sedimentation in inlet, and chemical sorption in primary and secondary media. In all cases, overflows and bypasses are provided in the UpFlow^{TM} filter to safely pass flows in excess of the capacities of each process.

The objective for the design of the UpFlow^{TM} filter is to select a media that provides a high flow rate having the smallest effective aperture. Tradeoffs will be made between the level of treatment provided to the treated flows and the amount of annual flows being treated. The modified WinSLAMM (Pitt and Voorhees 2003) will enable final design curves to be made. Currently, we can consider the previously presented performance curve (Figure 3.7) in conjunction with the particle size distribution (Figure 3.8). Table 3.3 summarizes this information. This shows that as the flow rate increases, the performance decreases, as expected. The highest level of suspended solids capture will occur with a combination of small flow rates and a large sump area. Obviously though, as the flow rate decreases, the amount of runoff that will bypass the filter media will increase, reducing the overall effective performance level. As shown on the particle size distribution plot, the median size at the Tuscaloosa, AL, test area was measured to be about 22 µm. In order to capture at least 50% of the suspended solids, relying on basic settling theory only, will require capturing that sized particle, at least, which would only occur at flow rates of about 3 gal/min, or less. According to Table 3.1, these flows, or less, would only occur during rains at this site that are less than about 0.15 inches in depth, occurring over a period of about 10 hours. Table 3.3 shows the rains associated with a range of expected removals, assuming theoretical settling. The lowest level of control is about 15% that would occur with 4 inch rains, or larger. Smaller drainage areas for each UpFlow™ filter would be more typical, with much better minimal levels of control. As an example, if the drainage area was about 0.25 acres of pavement, the lowest level of sediment control would be about 25% for these very large rains. It is also possible to increase the performance of the UpFlow™ filter by increasing the area of the catchbasin sump.

Table 3.1: Runoff Volumes (ft^3) and Average Flow Rate for Different Sized Rains at Test Area

Rain Total (inches)	**Pitched Roof**	**Flat Roofs**	**Paved Parking**	**Land Use Totals**	**Average Flow Rate (gal/min)**	**Rv**
0.01	0	0	2	2	0.08	0.07
0.05	10	1	44	55	0.8	0.33
0.1	38	11	110	159	2	0.49
0.25	119	61	343	523	5	0.64
0.5	256	142	787	1185	10	0.73
0.75	391	224	1278	1894	13	0.77
1	529	306	1824	2658	19	0.81
1.5	800	477	3010	4288	31	0.87
2	1078	654	4207	5940	42	0.91
2.5	1348	832	5389	7568	54	0.93
3	1617	1020	6556	9193	65	0.94
4	2156	1380	8878	12414	88	0.95

Table 3.2: Calculated Suspended Solids Produced at Test Site (lbs) and Amount Trapped in Sump

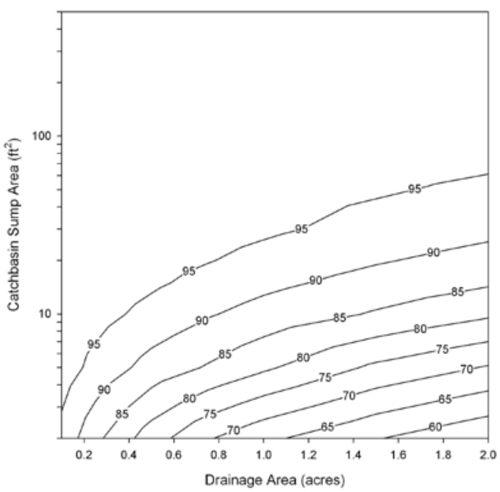
Rain Total (inches)	Pitched Roof	Flat Roofs	Paved Parking	Land Use Totals	Total After Inlet Sump	Amount Trapped in Inlet Sump	% SS reduction
0.01	0	0	0.0043	0.0045	0.00018	0.0043	96
0.05	0.020	0.0015	0.36	0.38	0.10	0.28	74
0.1	0.078	0.023	0.89	1.0	0.4	0.6	61
0.25	0.24	0.13	2.8	3.2	1.7	1.5	45
0.5	0.53	0.29	6.4	7.2	4.3	2.9	40
0.75	0.81	0.46	10	12	8	4	35
1	1.1	0.63	15	17	11	6	32
1.5	1.6	0.98	24	27	19	8	29
2	2.2	1.3	34	38	28	10	26
2.5	2.8	1.7	44	48	36	12	24
3	3.3	2.1	53	59	45	14	23
4	4.4	2.8	72	79	62	17	21

Table 3.3: Expected Performance of UpFlow™ Filter for Different Flow Rates (9 ft^2 sump area), Considering Sump Trapping Only

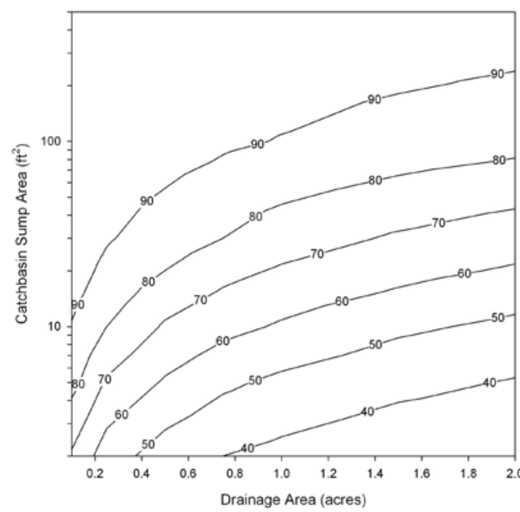
Flow rate (gal/min)	**Rain (inches) Producing this Average Runoff Rate**	**Critical Particle Size (µm)**	**Percentage SS Captured**
2.8	0.14	20	58
5.4	0.26	30	43
9	0.48	40	29
13	0.75	50	27
21	1.10	60	26
36	1.75	70	24
90	4.0	100	22
240	>4	200	18
390	>4	300	16
640	>4	400	15

Figure 3.9 contains a series of plots illustrating the sediment trapping capture capability of idealized catchbasins with sumps. These plots were prepared during sensitivity analyses using WinSLAMM and the particle size distribution measured at the Tuscaloosa test site. The values on these plots assume theoretical settling and no scour,

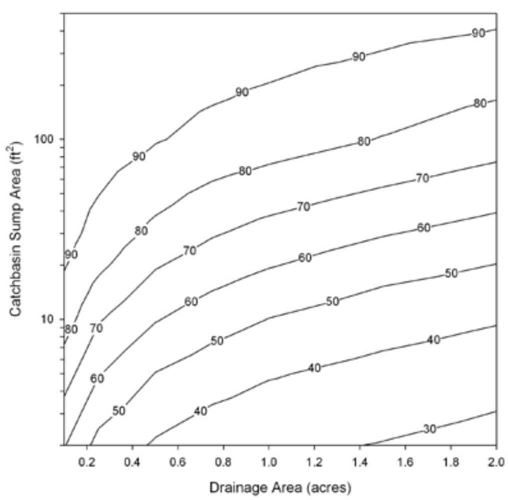
and are similar to values obtained during field monitoring of catchbasins. These require sumps and hoods to capture and retain the particles larger than the calculated critical particle sizes. Most catchbasins would be sized in the lower left hand corner of these plots, representing paved drainage areas of about 0.25 to 1 acre and sump areas of about 5 to 15 ft^2. In this range, the expected sediment capture efficiency would be about 25 to 35% for 1 inch rains and 35 to 45% for 0.5 inch rains. Increasing the sump surface area for a set drainage area would increase the theoretical sediment capture efficiency. However, it would be very difficult (and expensive) to achieve 80% removals of suspended solids with these devices that rely on plain settling alone.



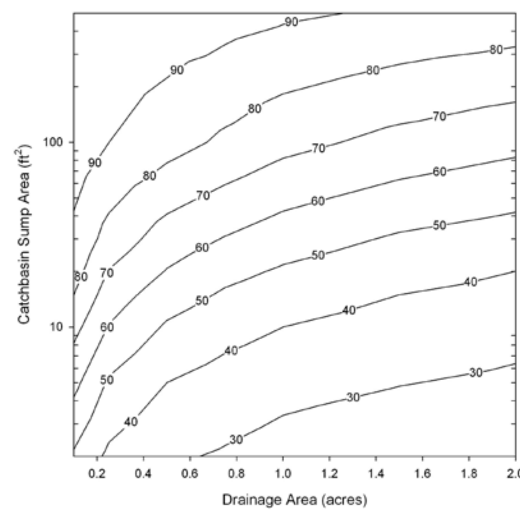
0.01 inch rain



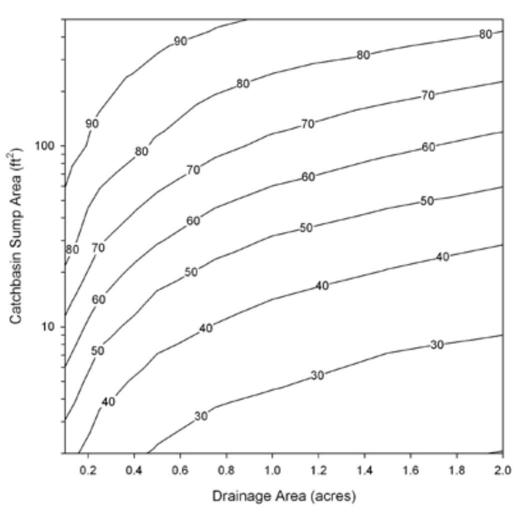
0.05 inch rain



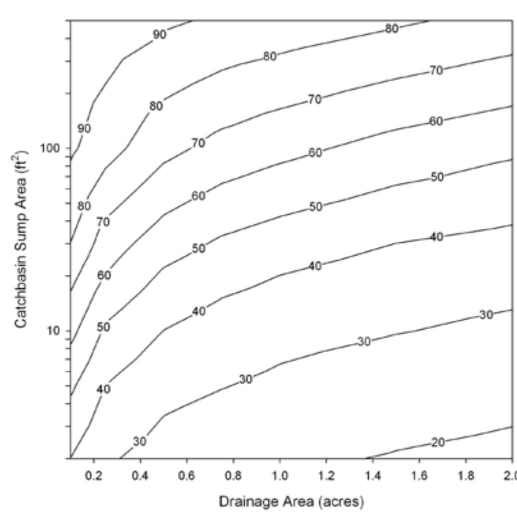
0.1 inch rain



0.25 inch rain

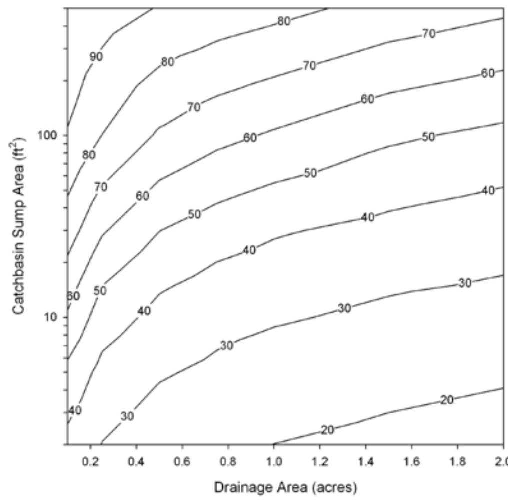


0.5 inch rain

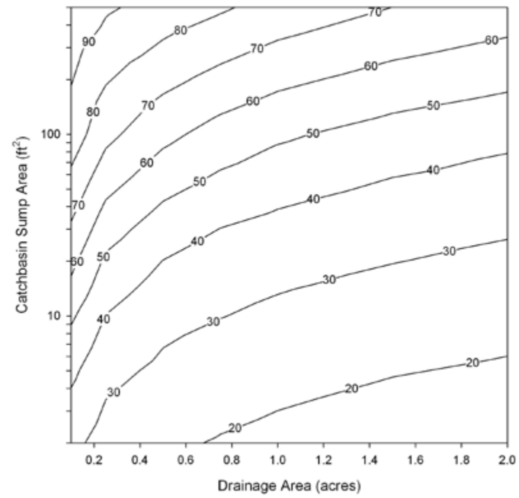


0.75 inch rain

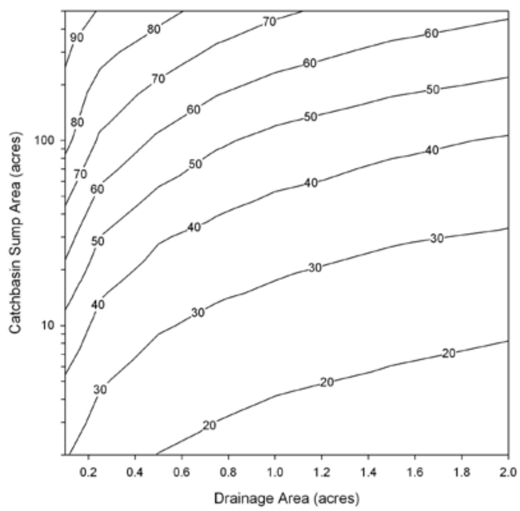
Figure 3.9a: Sediment capture (%) in catchbasin sumps for different sized paved drainage areas



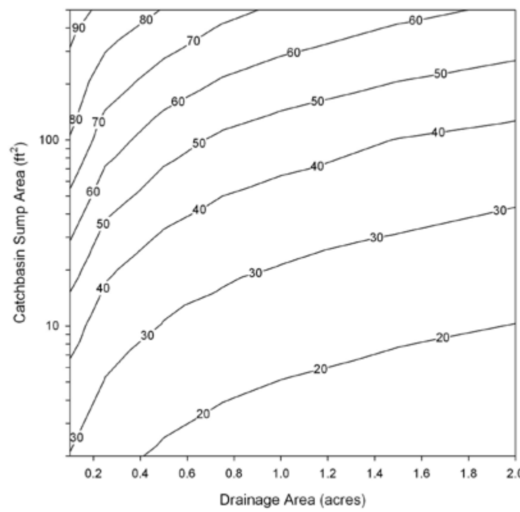
1.0 inch rain



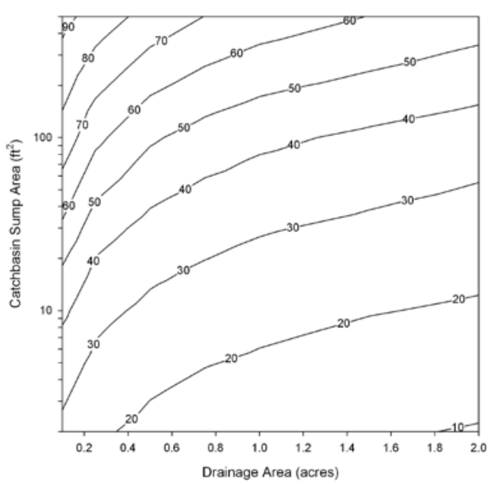
1.5 inch rain



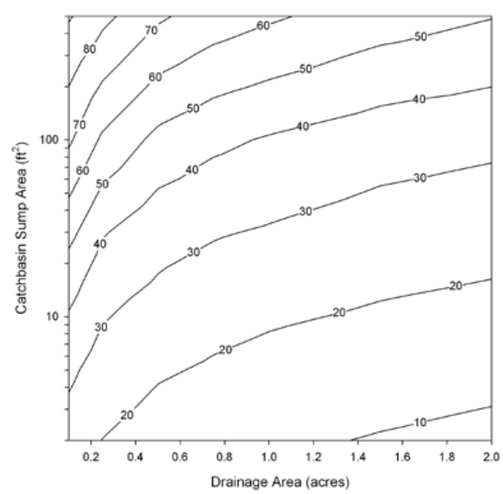
2.0 inch rain



2.5 inch rain



3 inch rain

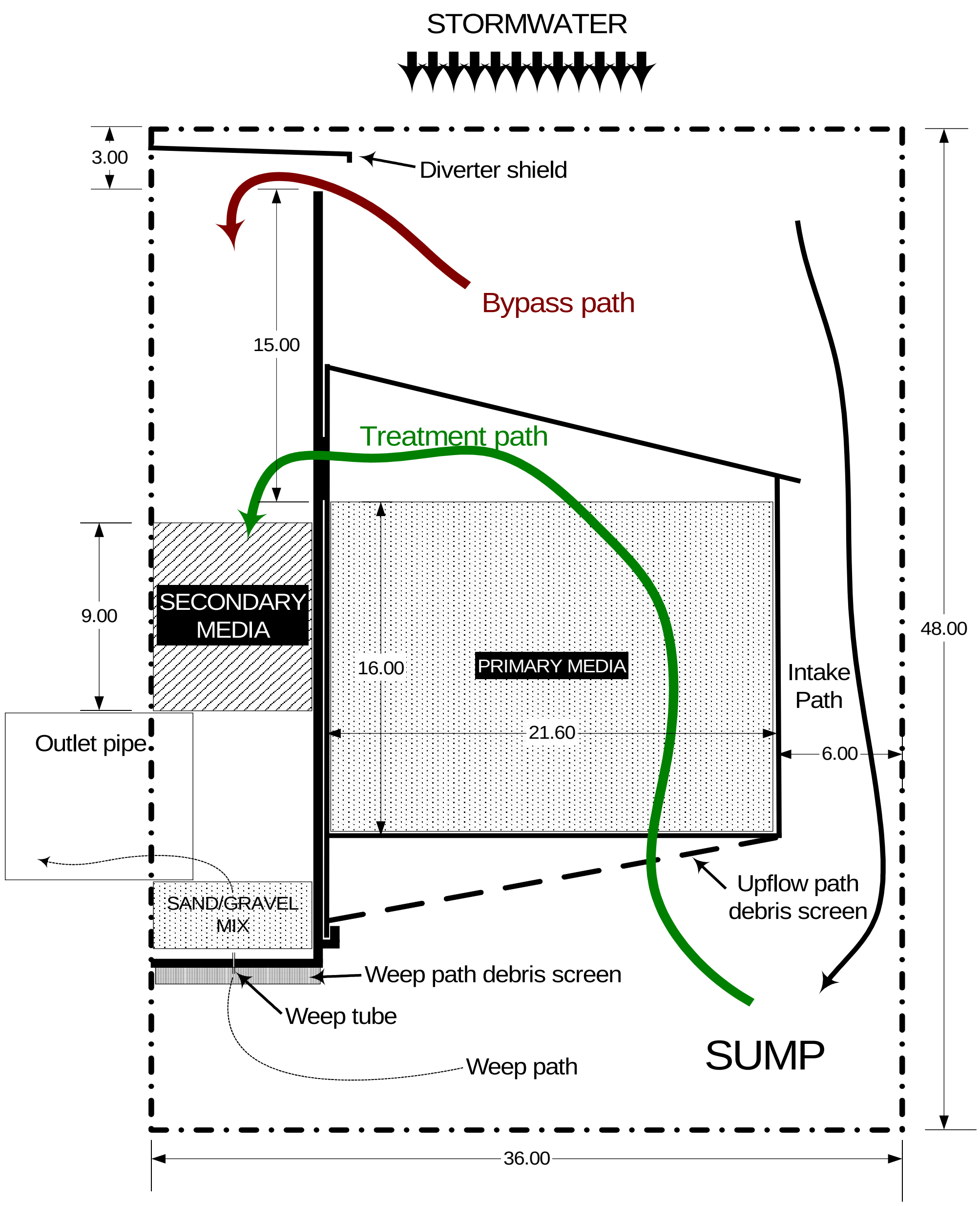


4 inch rain

Figure 3.9b: Sediment capture (%) in catchbasin sumps for different sized paved drainage areas

3.6 Design and Construction of Upflow Filter

The second generation UpFlow™ filter was constructed to fit in the modified inlet at the City Hall parking lot in Tuscaloosa, AL.



Catch Basin Upflow Filtration System

Figure 3.10: Side view of UpFlow™ filter and catchbasin system.

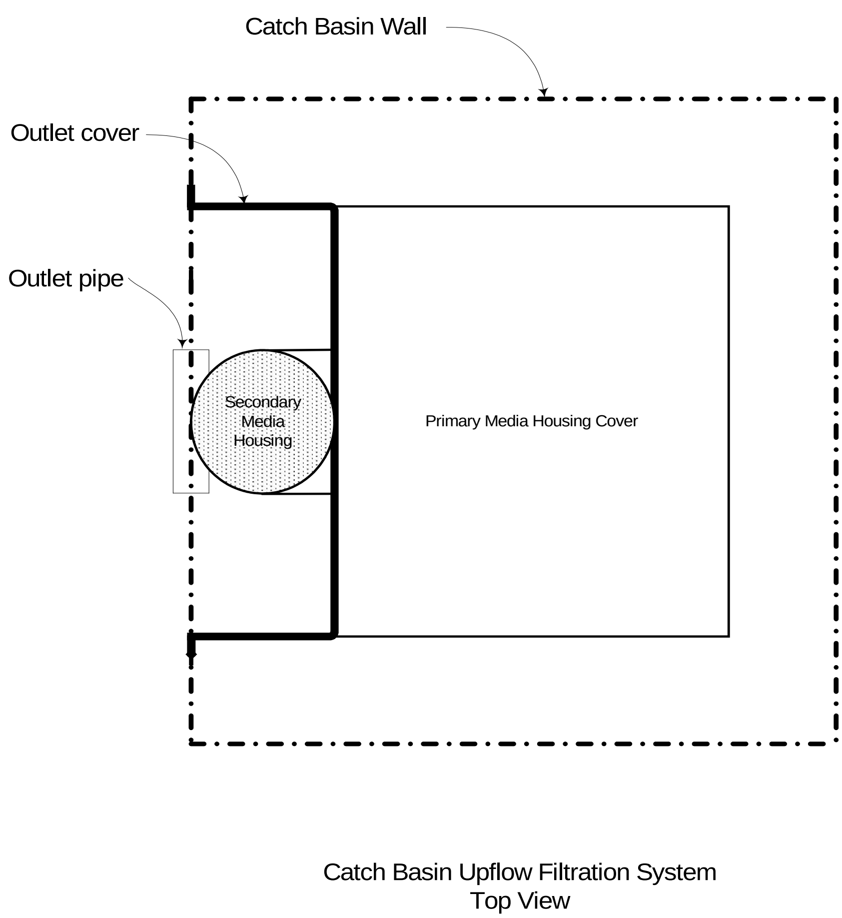


*Figure 3.11: Top view of **UpFlow™ filter** system.*

*Figure 3.12: Looking at bottom of **UpFlow™ filter**, main inlet with large holes (but without cover screen) and weep hole tubes and finer screen at right.*

*Figure 3.13: Looking down into **UpFlow™ filter**, with main upflow chamber to the right, the dividing wall with weir, the secondary filter chamber and overflow chamber.*

Figure 3.14: Side view of upflow chamber (upside down) with inclined floatable screen installed

ADDITIONAL PICTURES ON MEDIA BAGS FILLING, FOAM PLACING ETC

CHAPTER 4

SITE INVESTIGATION AND MONITORING EQUIPMENT INSTALLATION

4.1 Introduction

It is critical to thoroughly investigate a site that may be used to evaluate a control device, or to conduct any other monitoring, to ensure that it meets the project objectives. Flows and the pollutant concentrations that can be anticipated need to be known, for example. This chapter describes the preliminary investigations that were carried out at the test location at the City Hall parking lot, Tuscaloosa, AL. Also, this chapter describes the field equipment installations and the modifications that were made at the monitoring site.

4.2 Test Site Rainfall and Runoff Conditions

Figure 4.1 is a Tuscaloosa, AL, IDF (intensity-duration-frequency) curve that describes the characteristics of rare rain events at the test site, normally used for drainage design, prepared using the *Alabama Rainfall Atlas* software program developed by Dr. S. Rocky Durrans of the University of Alabama. Figure 4.2 is a plot showing sheetflow travel times for small paved areas and Figure 4.3 shows the travel velocities for sheetflows. For small paved and roofed areas that are best served with the UpFlow^{TM} filter, it is clear the times of concentration (Tc) will be less than 5 minutes. In this case, it is customary to use the rainfall intensity associated with a Tc of 5 minutes. For

Tuscaloosa, these rainfall intensities can be quite large, ranging from about 6 in/hr for rains having about a 50% chance of occurring in any one year (the so-called “2-year” storm), to about 10 in/hr for rains that may only occur with a 1% chance in any one year (the “100-year” storm). Except for the smallest events, these design storms are usually not suitable for water quality treatment, but the inlet and any inserts must be capable of accommodating the expected peak flow rates that may be expected for the designated critical design storm for the site.

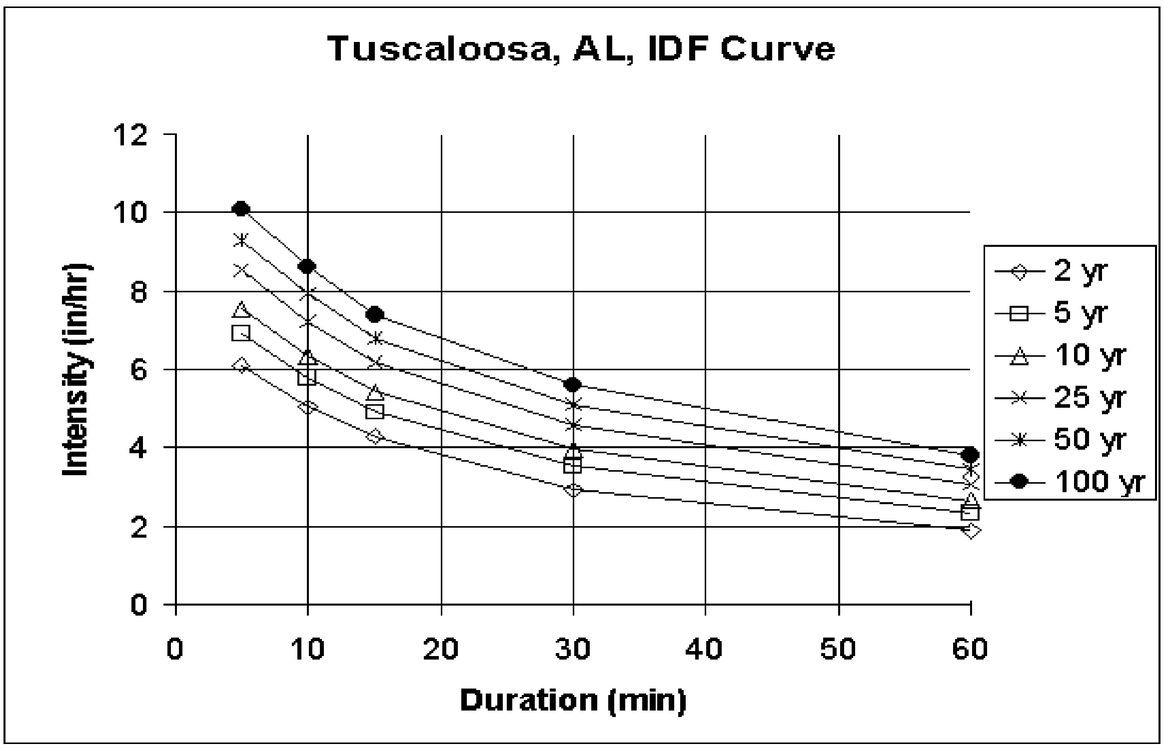


Figure 4.1: Tuscaloosa, AL, IDF curve (Alabama Rainfall Atlas)

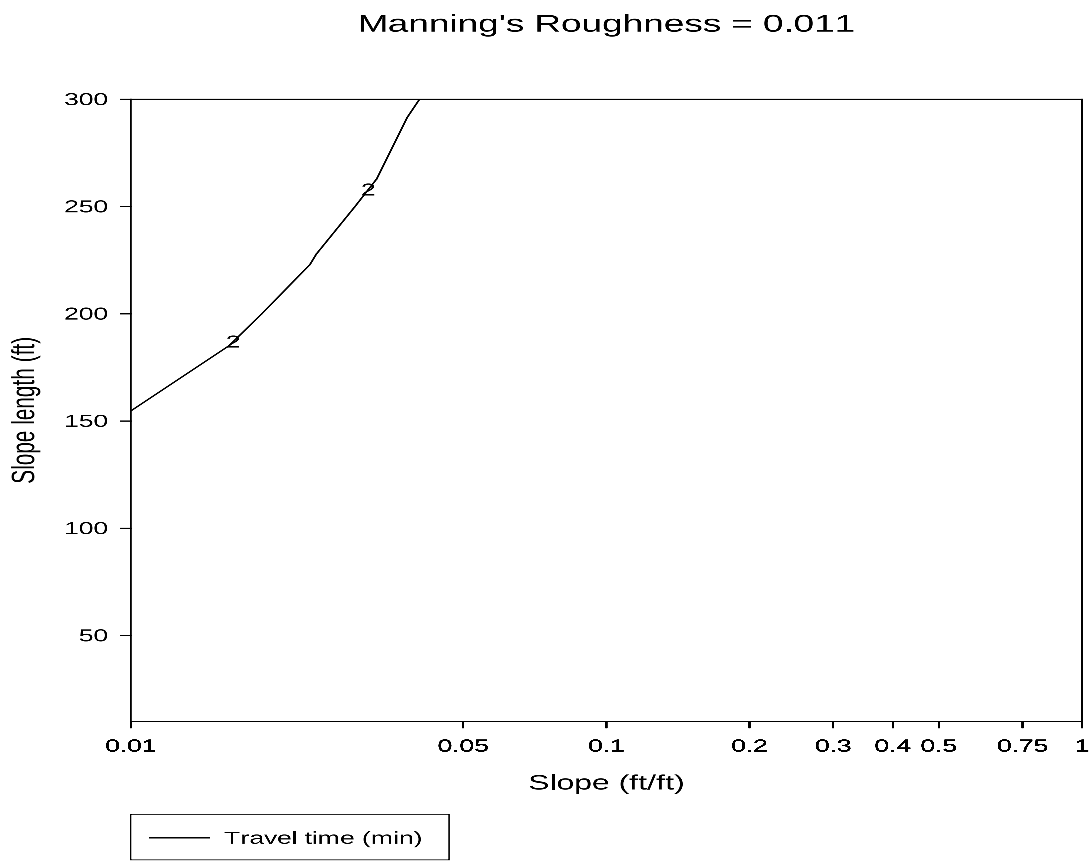


Figure 4.2: Sheetflow travel times for smooth surfaces (concrete, asphalt, gravel, or bare soil). The travel times to the right of the 2 minute contour line are between 1 and 2 minutes, while the travel times to the left of the 2 minute contour line are between 2 and 3 minutes.

Figure 4.4 is a plot showing estimated peak runoff rates for different rainfall intensities and paved drainage areas, while Figure 4.5 is a similar plot for average runoff rates for rains having different total rain depths. The peak runoff rates are estimated using the Rational formula, while the average runoff rates are estimated using volumetric runoff coefficients and typical rain durations for these sized rains in the region. The test site is quite large for the UpFlow™ filter, but that enabled us to obtain a wide range of flow observations during a smaller set of monitored events. The test site has about 0.9 acres of pavement and roof area and is expected to produce peak flow rates of 20 gal/min for short periods of rainfall intensities of about 0.1 in/hr, and peak 50 gal/min flow rates for short periods of rain intensities of about 0.2 in/hr. Average 20 gal/min flows would be expected during about 0.75 inch rains, and average 50 gal/min flows would be expected during about 2 inch rains. These rain conditions are expected to commonly occur at the prototype test site during the monitoring period. Peak flow rates (associated with short-term rain intensities of 6 to 10 in/hr) would be about 2,500 to 5,000 gal/min during the 2 to 100-year design storms.

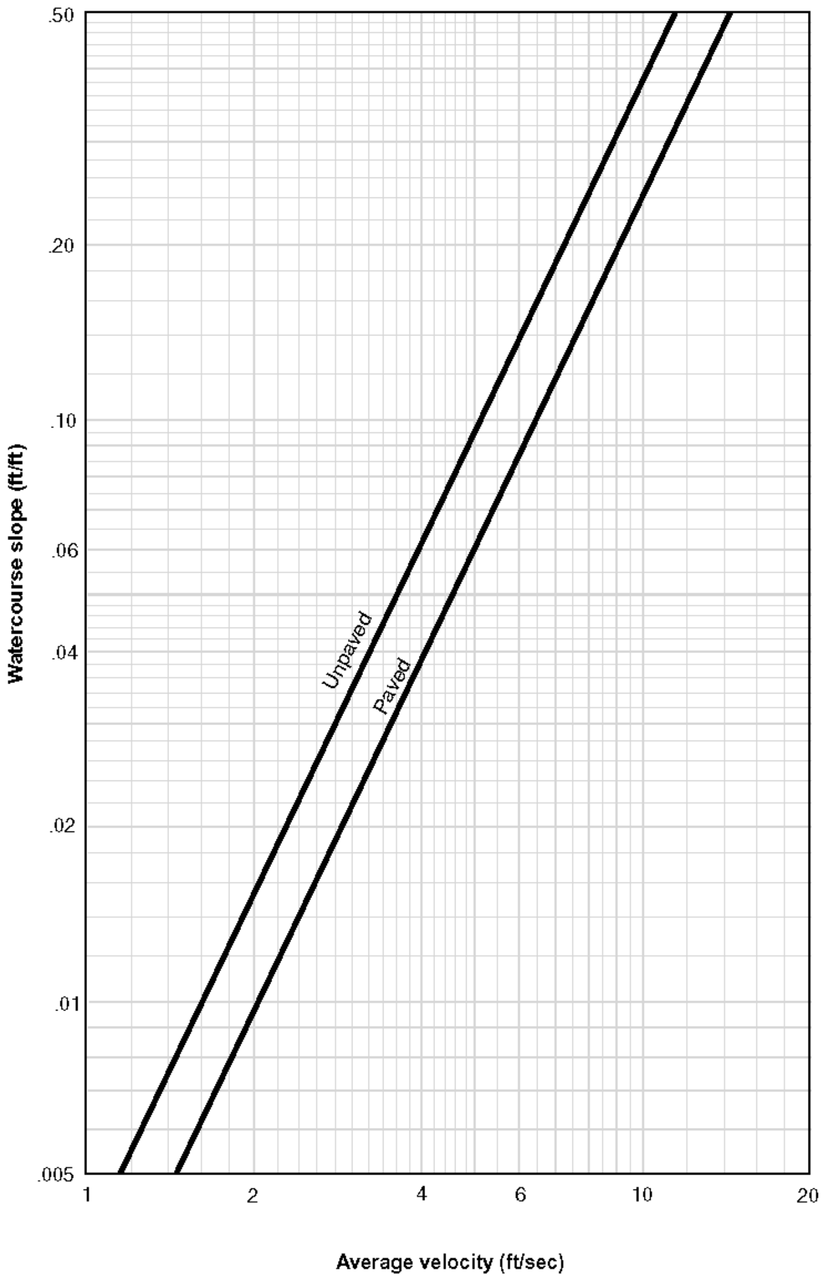


Figure 4.3. Shallow concentrated flow velocities (SCS 1986).

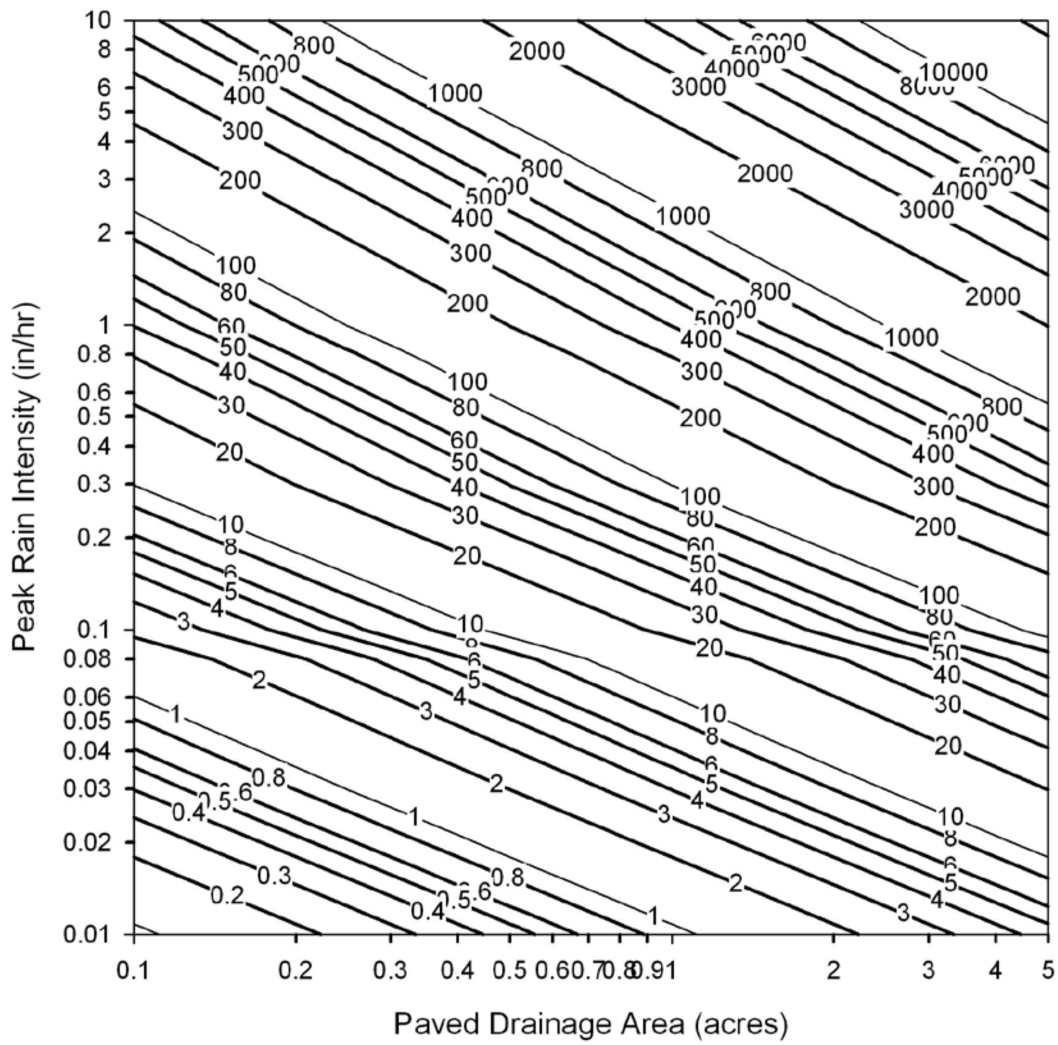


Figure 4.4: Calculated peak runoff rates (gallons/minute) for different peak rain intensities and drainage areas for paved areas.

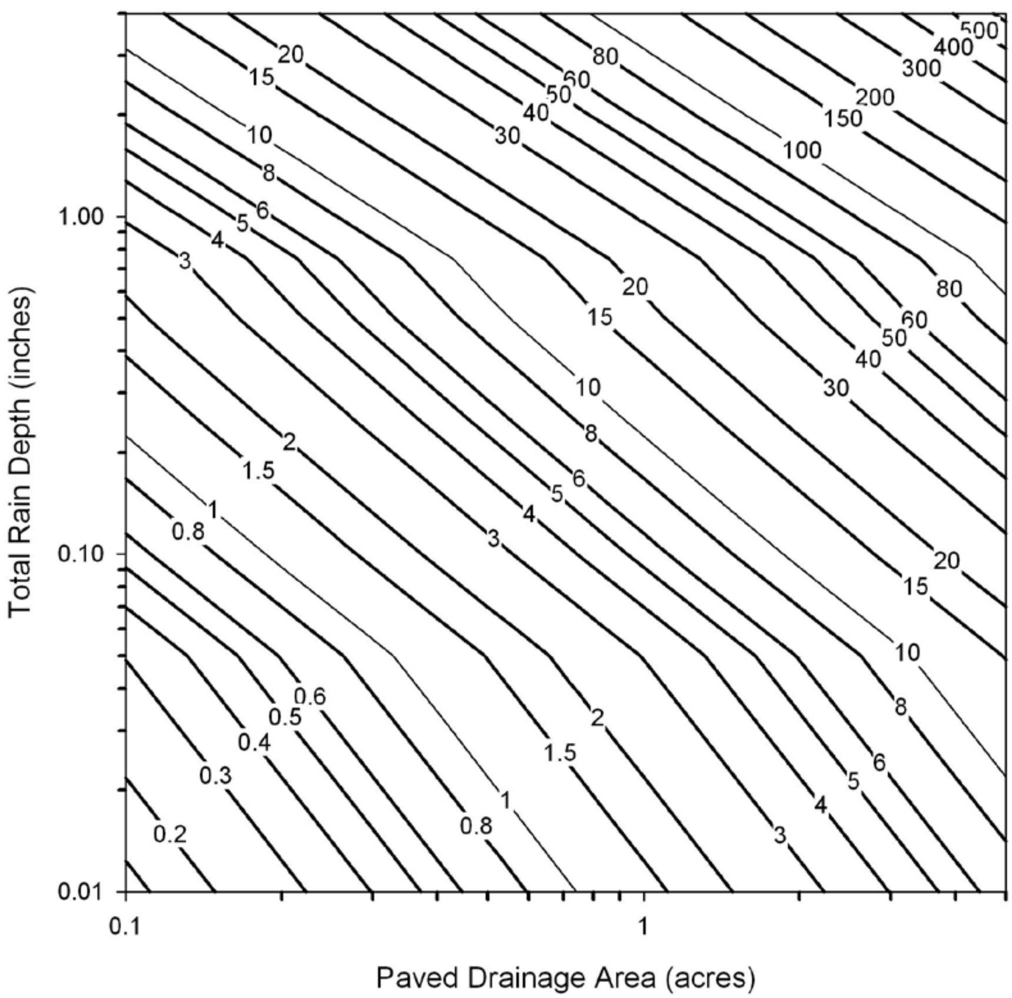


Figure 4.5: Average runoff rates (gallons/minute) for different total event rain depths and paved drainage areas.

Figure 4.6 is the preliminary particle size distribution measured at the test site. Runoff samples were taken manually at the inlet using a dipper sampler to collect water that was cascading from the gutter into the inlet, ensuring that all particulates would be captured in the sample. The sample was sieved using a 1500 µm screen to remove any large material. This large material was then washed from the screen and analyzed. The rest of the sample was split using a Dekaport/USGS cone sample splitter and the separate split fractions were sieved using a 200 µm sieve and a 0.45 µm filter. The sample fraction between 0.45 and 200 µm was analyzed using a Coulter Counter Multisizer 3. The size information was then combined to produce this plot which is only for particulate matter and did not include any large debris (leaves or litter) larger than 1500 µm, or dissolved solids less than 0.45 µm. The median particle size of the particulate matter in the runoff was about 25 µm, and about 15% of the particulates were larger than 250µm (but smaller than 1500 µm). During the initial testing in late fall, a substantial amount of leaves (about 5 ft^3), and several large pieces of litter (soda cans, plastic bags, and Styrofoam cups) were accumulated in the sump. The mass of the large debris was relatively small compared to the amount of runoff that flowed through the system, and would have an insignificant effect on the particle size distribution. However, these important gross solids were effectively captured in the modified inlet.

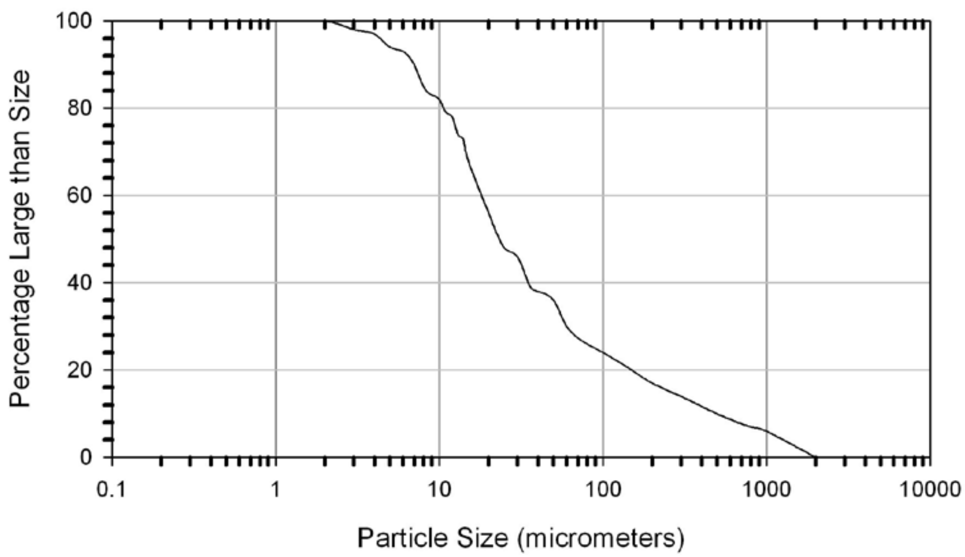


Figure 4.6: Observed particulate matter size distribution in runoff water at Tuscaloosa test site.

4.3 Installation of Monitoring Equipment and Upflow Filter

The City of Tuscaloosa, AL, graciously allowed us to test the UpFlow™ filter at a parking area adjacent to the new City Hall. In addition, they modified the inlet to meet our monitoring needs. This cooperation is greatly appreciated and critical, especially considering the problems in identifying a suitable inlet on campus. The following photographs show the inlet and modifications, along with the drainage area.

Figure 4.7: Inlet box with forms removed from baffle divider showing main outlet.

Figure 4.8: Area-velocity sensor installed in effluent pipe.

Figure 4.9: Installed prototype UpFlow^{TM} filter (empty) with sampling and monitoring lines

Figure 4.10: Prototype UpFlow™ filter with cover installed.

Figure 4.11: Completed modified inlet with new inlet grating.

Figure 4.12: Sampler shelter.

Figure 4.13: Samplers and flow meters in sampling shelter.

Figure 4.14: Tipping bucket and standard rain gauges installed at monitoring site.

Figure 4.15: YSI Sondes installed in sump (another is behind baffle wall for effluent)

4.4 Test Inlet at City Hall Parking Area, Tuscaloosa, AL

In August, 2004 we received permission from the mayor of Tuscaloosa and the support of the city’s stormwater group to use one of their inlets adjacent to their new city hall. After a quick search, we found a very good inlet. The city also modified the inlet to suit our needs. Originally, we had planned to dig out the bottom of the inlet and install a sump. However, the bottom was already 5 feet deep, so it was decided to simply use a baffle wall to divide the inlet into a suitable chamber. This resulted in a much better arrangement as we had much better access to the effluent side. Also, a full-size inlet grating was installed to allow access to the complete inlet area.

The City was very helpful in assisting us, including making modification in early September to the inlet to allow monitoring. The following are photographs of the test area (0.9 acres of parking, roofs, and adjacent storage areas), and the modifications to the inlet. The inlet now has a 3 inch thick concrete baffle to provide a 2-1/2 ft deep sump and a suitable mounting area for the prototype UpFlow™ filter.

The inlet box is 5 ft deep, 32 inches front to back, and 36 inches wide. The cast cover is only about 18 in front to back, but the city opened it up to full size. The city also installed a baffle wall to separate the inlet into two chambers.


15 inch outlet in bottom corner (small 4 inch outlet above, from landscaped area)

Inlet adjacent to diesel storage area for emergency generator.

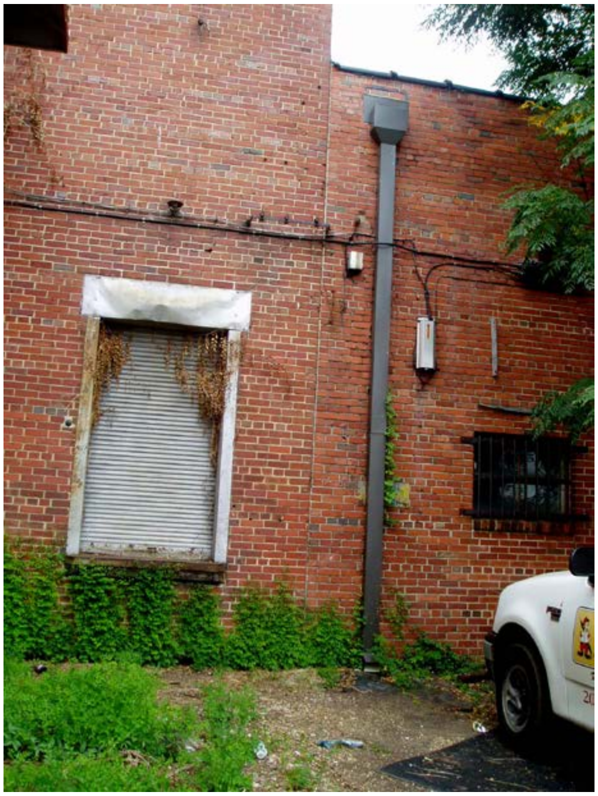

About ¼ acre of asphalt paved parking area draining to inlet.

Figure 4.16: Initial views of test inlet and vicinity before modifications.

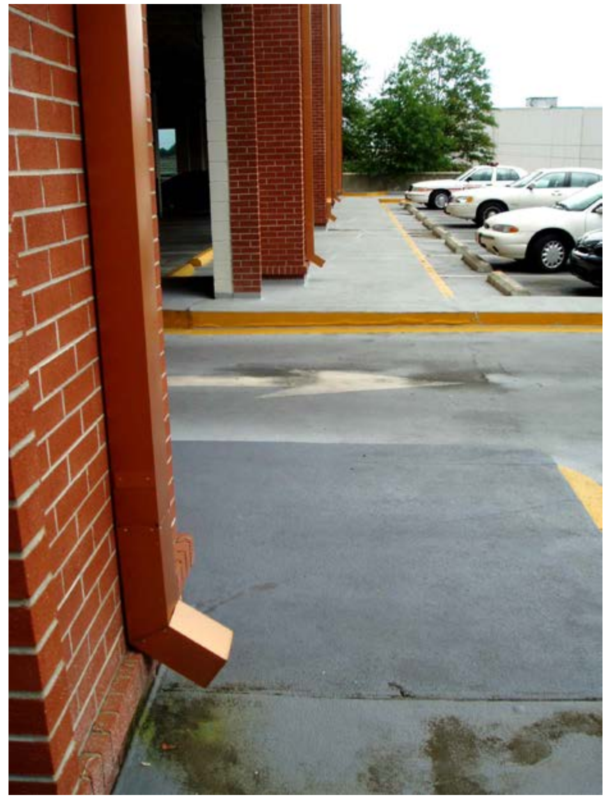
Debris boxes and outside AC condensers also in drainage area.

One area of building debris storage in small drainage area.

Figure 4.17: Views of test area drainage.



An older roof drain in test area.



New city hall roof drains to elevated concrete deck that drains to inlet (coated aluminum downspouts and roof, just looks like copper).

Elevated concrete parking area that also drains to test area.

Side view of elevated parking and city hall roof and drains to test area.

Figure 4.18: Views of test area drainage.

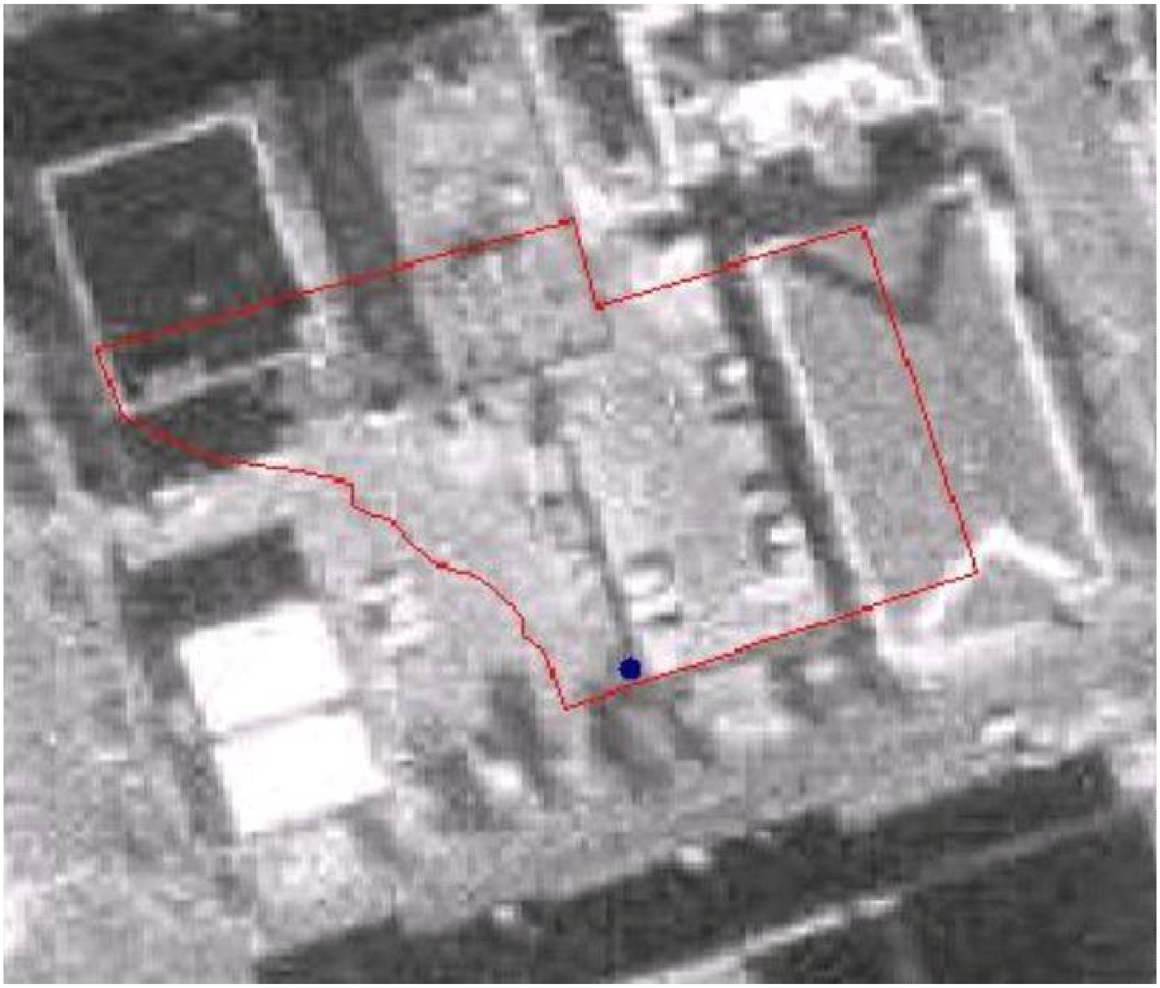


Figure 4.19: Drainage area on an overly-enlarged aerial photo.

4.5 Monitoring Results at Tuscaloosa City Hall Inlet Site, before UpFlow™ Filter Installation

From the 3rd week of October, 2004 until the 2nd week of November, 2004 the YSI 6600 Sondes were deployed in the inlet at the City Hall, Tuscaloosa location to determine their consistency and drift for an extended deployment. The sondes were calibrated in the laboratory before their deployment and were tested on the same standards after their deployment to measure any change before they were re-calibrated. Tables 31 and 32 show that the drifts of the parameters over this 20 day "turned on" plus 10 day "idle" period was very low, especially considering that the sondes were dry much of the time. The drifts were less than 3% during this period, well within a useable range.

Table 4.1: Drift in sonde -1 on November 18, 2004 after one month field deployment

	pH	**pH**	**D.O. %**	**DO (mg/L)**	**Depth (ft)**
Before new calibration (20th Oct 2004 prior calibration)	7.09	4.09	97.7	8.96	0.028
After new calibration 18th Nov 2004	7.00	4.00	100.1	9.18	0
% change	1.3%	2.3%	2.4%	2.4%	n/a

Table 4.2: Drift in sonde -2 on November 18, 2004 after one month field deployment

	pH	**pH**	**DO %**	**D.O. (mg/L)**	**Depth (ft)**
Before new calibration (20th Oct 2004 prior calibration)	7.11	4.11	98.7	9.05	0.025
After new calibration 18th Nov 2004	7.00	4.00	100.0	9.16	0
% change	1.6%	2.8%	1.3%	1.2%	n/a

A total of 1,920 data points (collected at 15minute intervals) were obtained. The various parameters that were logged were temperature, specific conductivity, DO, pH, ORP and turbidity. There were very large variations in the turbidity data indicating the runoff conditions at this site. The turbidity range has a maximum of about 1,100 NTU and the water periodically exceeded this value. During this 20 day period, two large storms occurred that were monitored, but most of the time the sondes were dry (Table 4.3 summarizes the rain characteristics).

Table 4.3: Summary of rain events during the initial sonde deployment

Event	Duration (hr : min)	Peak 5-min Intensity (in/hr)	Total Rainfall (in)
Event 1 (10/23/04)	3:15	4.17	0.84
Event 2 (11/3/04)	5:25	4.24	1.18

4.5.1 Storm sample analysis

The following table lists data for the first storm we sampled at the Tuscaloosa City Hall parking area test inlet on August 20 – 22, 2004, a very intense and large rain. Samples CH1 and CH2 are duplicates of the first flush that occurred within the first 30 minutes of this very intense rain. Samples CH3 and CH4 are duplicates collected two days later for the same event, after substantial runoff. CH1 was affected by ethylene glycol (radiator coolant) that had recently been blown from vehicle air conditioner heat exchangers on this very hot day. By the time the second sample (CH2) was collected a few minutes later, the color of the water was substantially reduced. The other notable effect is that the bacteria remained at very high levels throughout this event, in the obvious absence of sanitary sewage sources, while the other concentrations were greatly reduced (notably for TSS, conductivity, turbidity, COD, and phosphates).

Table 4.4: Chemical analysis data for the storm events during August 20 – 22, 2004

Sample #	**Sampling date**	**Total Coliform (MPN/100 mL)**	**E-coli (MPN/100 mL)**	**Enterococci (MPN/100 mL)**	**TSS (mg/L)**	**pH**
CH 1	20-Aug	>2419.2	1732.87	1046.24	136	6.72
CH 2	20-Aug	>2419.2	145	71.4	106	6.35
CH 3	22-Aug	>2419.2	>2419.2	1986.28	4	6.44
CH 4	22-Aug	>2419.2	>2419.2	>2419.2	3	6.35

Table 4.4: Chemical analysis data for the storm events during August 20 – 22, 2004 (cont)

Sample #	**Sampling date**	**Conductivity (μs/cm)**	**Turbidity (NTU)**	**Color (HACH color units)**	**COD (mg/L)**
CH 1	20-Aug	123	134	No match	184
CH 2	20-Aug	118	59	95	233
CH 3	22-Aug	8	3	5	39
CH 4	22-Aug	13	2	5	44

Table 4.4: Chemical analysis data for the storm events during August 20 – 22, 2004 (cont)

Sample #	**Sampling date**	**Phosphates (mg/L)**	**Ammonia (mg/L)**
CH 1	20-Aug	1.94	0.19
CH 2	20-Aug	1.35	0.07
CH 3	22-Aug	0.09	0.03
CH 4	22-Aug	0.15	0.01

CHAPTER 5

PEFORMANCE EVALUATION BY CONTROLLED TESTS

5.1 Introduction

Historically, the performance of filters has been measured during controlled laboratory experiments. Controlled tests enable precise measurements of filter behavior under repeatable conditions. Careful experiments enable us to model the behavior of filters during a wide variety of actual storm conditions. However, the major disadvantage of controlled testing is that they do not account for unusual and over-range conditions, such as bypass flows which may control filter performance during actual storm events. Also, actual stormwater is rarely used during controlled laboratory tests, with questionable transferability to real conditions. Idealized flow can be determined in the absence of clogging particulates, but pollutant retention measurements are difficult. The recent use of ground silicas available from the U.S. Silica Co., have enabled more accurate filter tests under controlled conditions. However, the filter removal capability for most pollutants requires the use of actual stormwater as a test solution (Clark 1999, Clark and Pitt 1999, Johnson, *et al*. 2003, etc.).

5.2 Tested Media and Flow Rate Estimation

Based on the literature review and from the SBIR phase-I studies, four media types were tested for performance in the UpFlow^{TM} filter. Granular activated carbon, bone char carbon, Mn coated zeolite and also a mixed media which is made of 45% bone char carbon, 45% Mn coated zeolite and 10% peat moss were evaluated during this research. Most of the work was directed towards the mixed media as it was expected to have higher pollutant removal rates with adequate flow capabilities (25 to 30 GPM). The prototype UpFlow^{TM} filter that was tested can hold two media bags, each about 1-1/2 ft^2 in area and 0.3 ft thick (therefore having a volume of 0.5ft^3 each).

Flow tests for each media type were conducted in the field with the cooperation of the Tuscaloosa Water Department by using a fire hose connected to a fire hydrant adjacent to the test site. The flows were measured using their calibrated meter, and also checked at the test rates by timing the filling of large containers at relatively low flows. The following photographs show the condition the filter during this period, along with some photos of the flow tests. The maximum flow through the UpFlow^{TM} filter, using the mixed media ranged from 29 to 31 gpm (about 20 gal/min/ft^2). The flow test results for all of the media are presented in Table 5.1. For the mixed media and the Mn coated zeolite, the flow rates were also determined with the bottom screen cleaned and with new media, and after extended use. Figure 5.1 shows various pictures in which 300 gpm of flow is handled by the empty UpFlow^{TM} filter without any bypass over flow. Table 5.2 shows the flow capacity and the required head loss for each tested media. Figures 5.2 to 5.5 show the flow vs head graphs for all the four media.

Effluent pouring onto sonde and sample intake

300 gpm showing overflow bypass

300 gpm into empty chamber with no overflow

31 gpm flow with Mixed media at capacity

Figure 5.1: Various pictures showing upflow filter and the catchbasin at different flow ranges.

Table 5.1: Flow capacity for different media

Media (2 Bags)	Flow Rate (GPM) for 1.5 ft^2 filter area	
Mn Coated Zeolite	14	Dirty media and screen
	21	Clean media and after cleaning the screen
Mixed	14.5	Dirty media and screen
	29 to 31	Clean media and after cleaning the screen
Granular Activated Carbon	47.5	Clean media and after cleaning the screen
Bone Char Carbon	46	Clean media and after cleaning the screen

Table 5.2: Flow Capacity Measurements with available head for each media

Flow Type	**Mixed Media**		**Activated Carbon**		**Bone Char Carbon**		**Mn Coated Zeolite**	
	head (in)	Flow (gpm)	head (in)	Flow (gpm)	head (in)	Flow (gpm)	head (in)	Flow (gpm)
Low	14.5	5.8	14.25	7.5	14.75	14.5	15.5	6.34
Medium	17	15.3	16.75	21	15.75	28	16.75	10.03
High	22.25	27	22.25	46	21.75	50.5	22.25	20.8

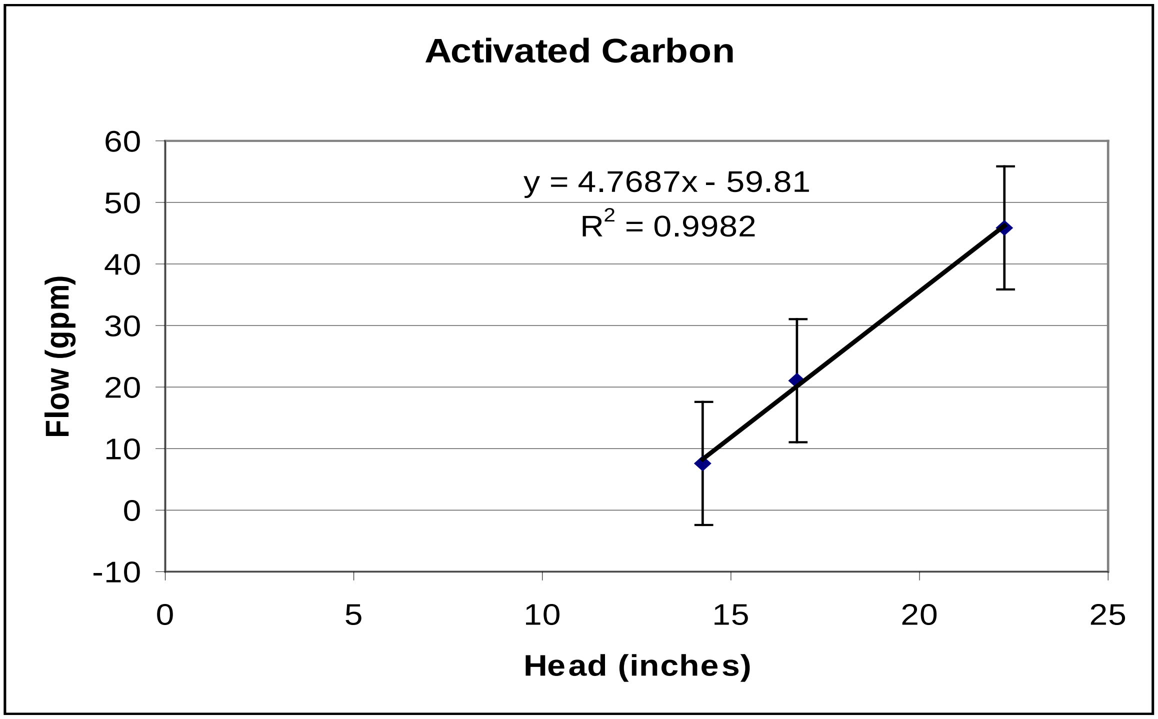


Figure 5.2: Flow Vs Head graph for Activated Carbon

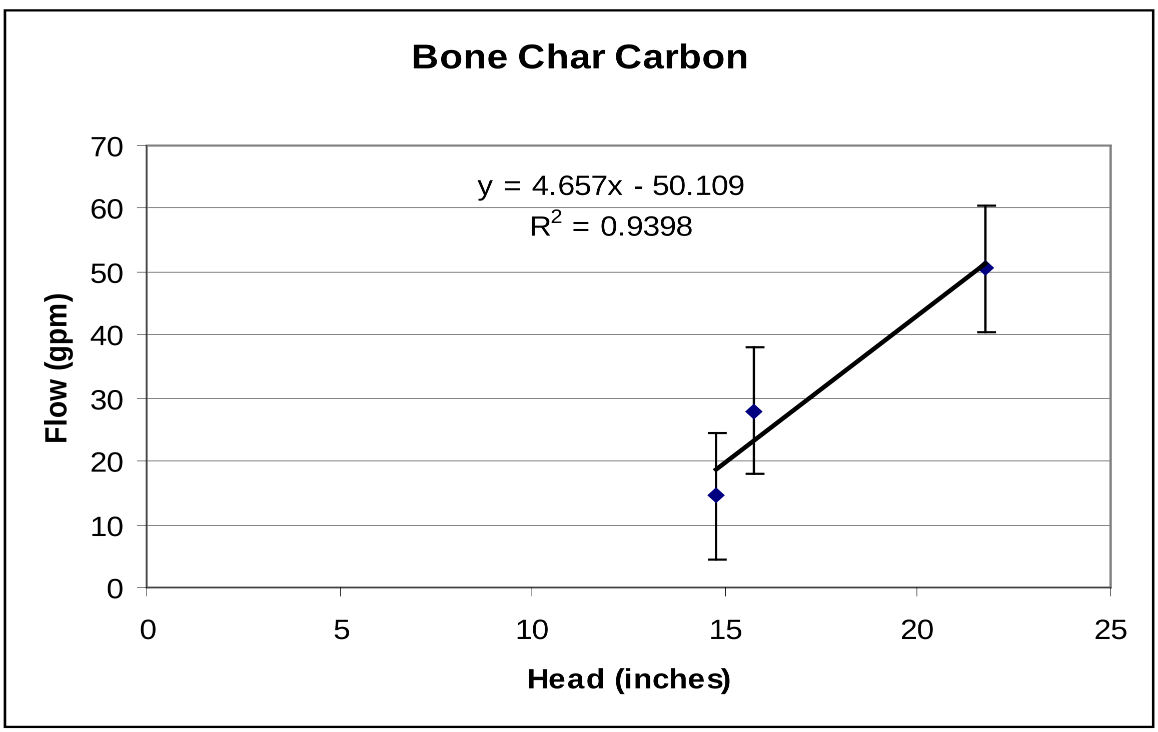


Figure 5.3: Flow Vs Head graph for Bone Char

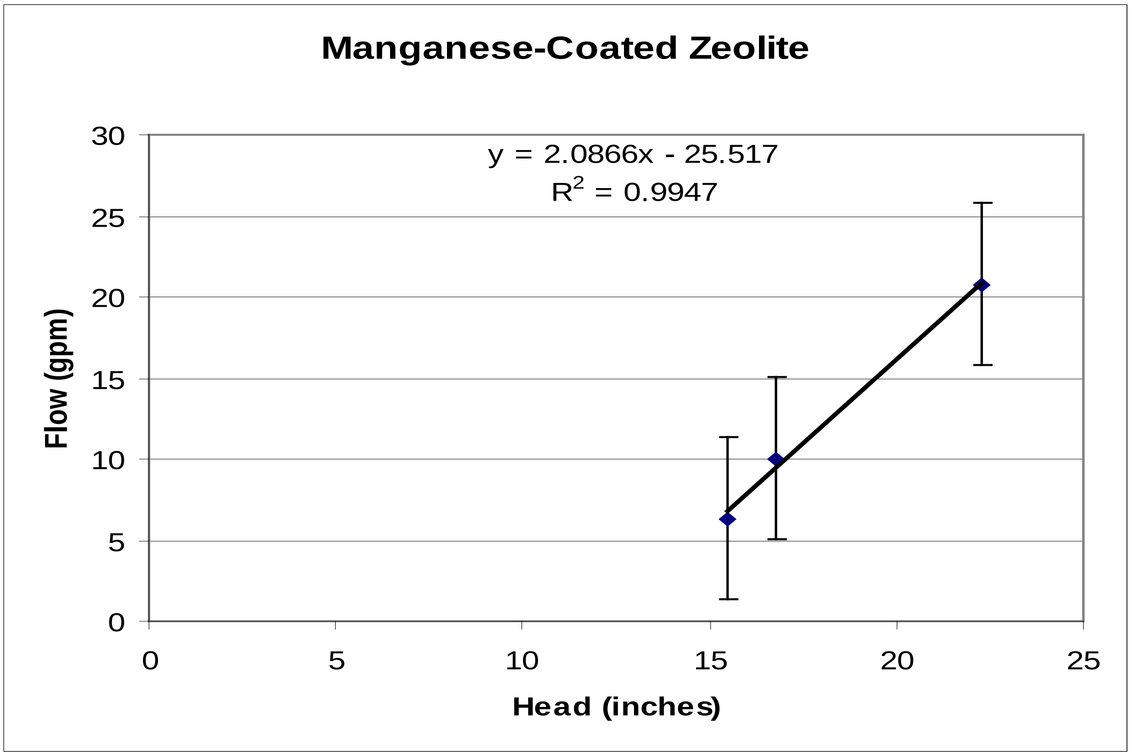


Figure 5.4: Flow Vs Head Graph for Mn coated zeolite

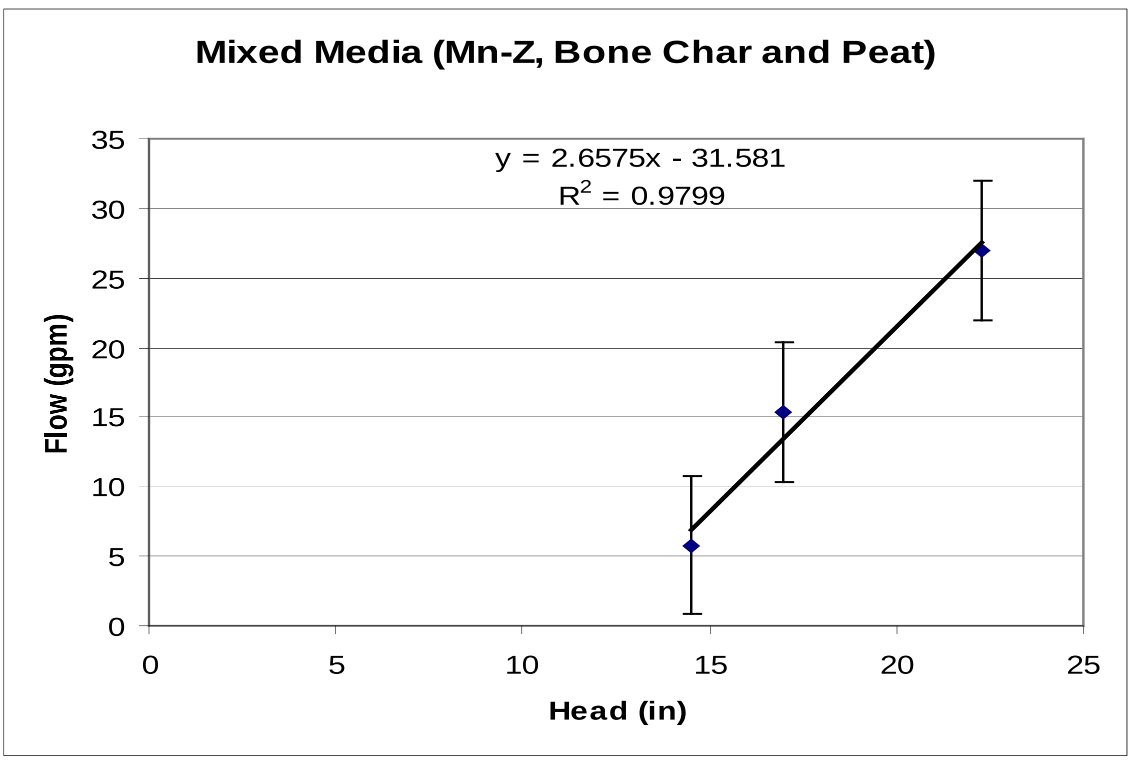


Figure 5.5: Flow Vs head graph for mixed media

5.3 Particle Size Distribution of tested media and test sediment

The particle size distribution for the tested media was determined using standard sieve analyses. Granular activated carbon and bone char carbon had a higher median particle size diameter at 2750 µm, while the manganese-coated zeolite and the fine sand were at the lower end, with median particle diameters of about 500 µm. The particle sizes for the coarse sand ranged from 125 µm and 2000 µm.

The test sediment in the stormwater stimulant was based on the following mixture: Sil-Co-Sil 250, Sil-Co-Sil 106 (both from the U.S. Silica Co.), coarse sand, and fine sand. The mixture was made by using equal weight fractions of each of the four components. The test sediment particle size ranged from 0.45 µm to 2000 µm. Two different batches of the test sediment were prepared and the particle size distributions for each the batches were determined.

Figures 5.6 to 5.12 show the particles size distributions of all the tested media and the two test sediment batches.

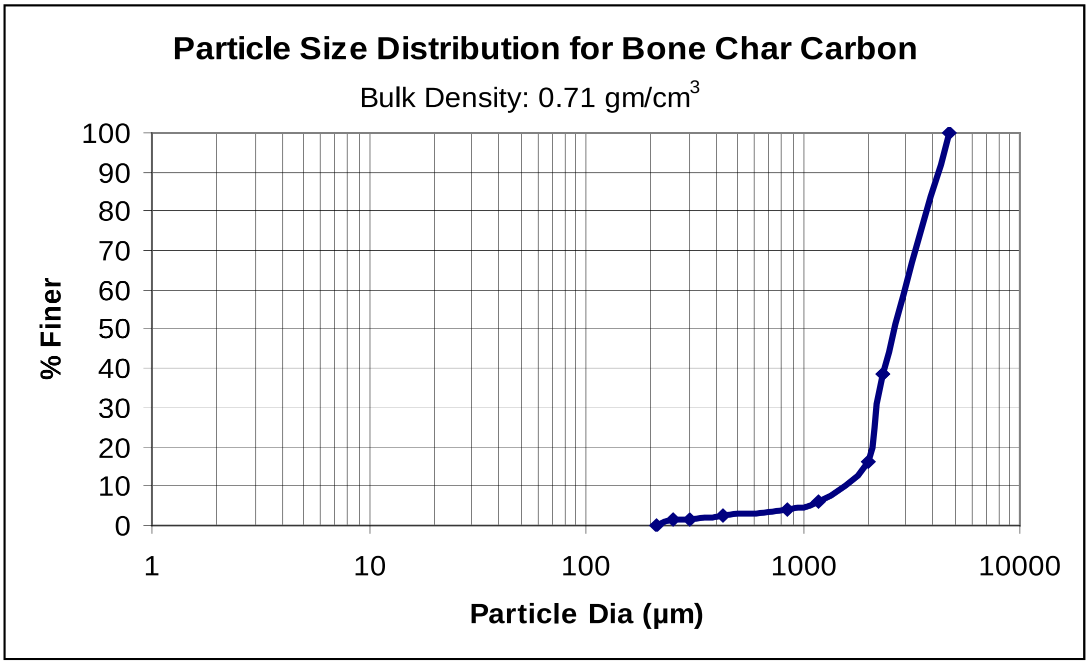


Figure 5.6: PSD curve for Bone Char Carbon

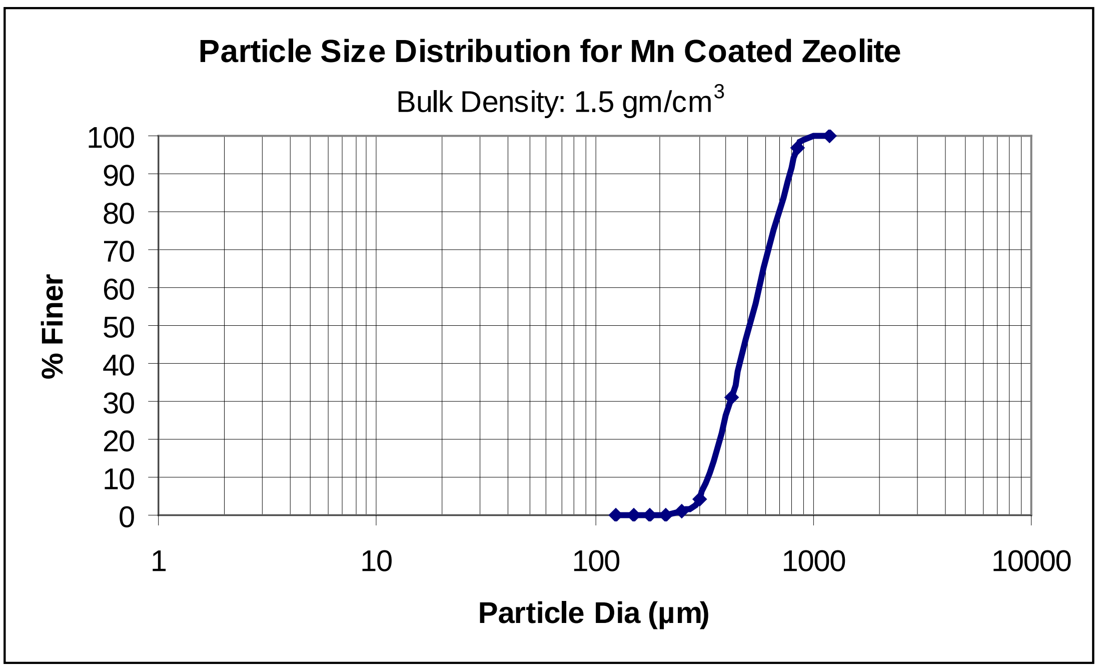


Figure 5.7: PSD curve for Manganese coated zeolite

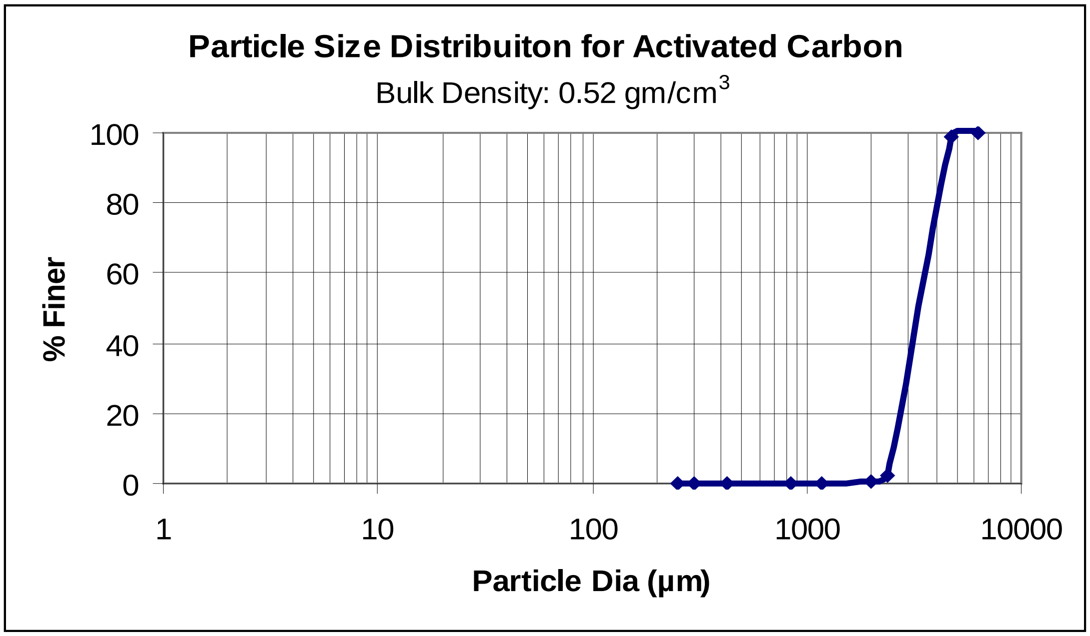


Figure 5.8: PSD curve for granular activated carbon

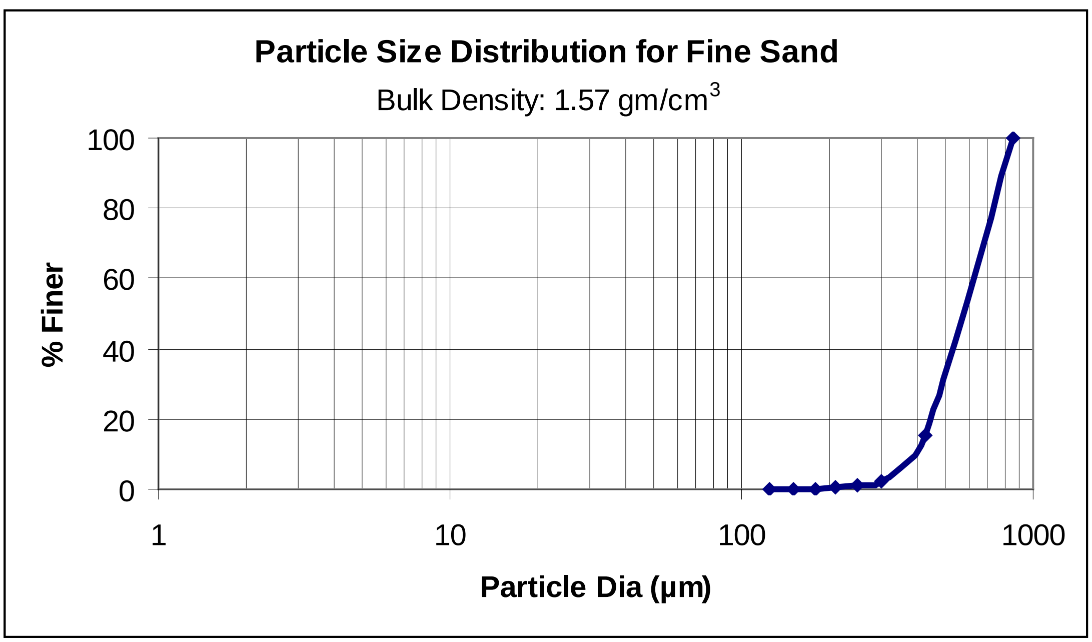


Figure 5.9: PSD curve for fine sand

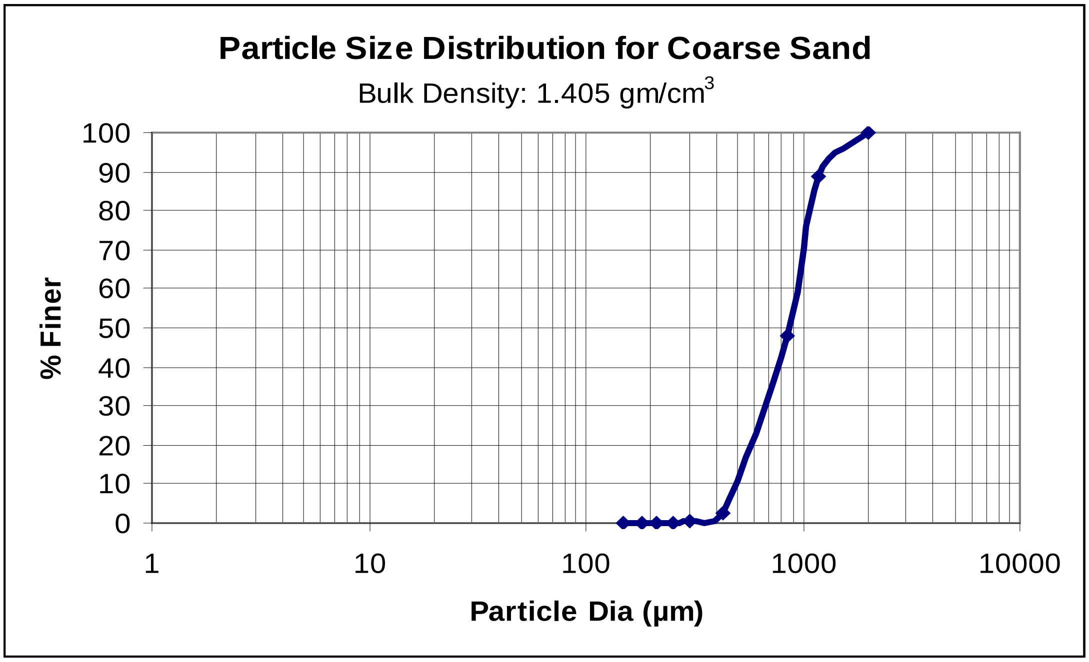


Figure 5.10: PSD curve for coarse sand

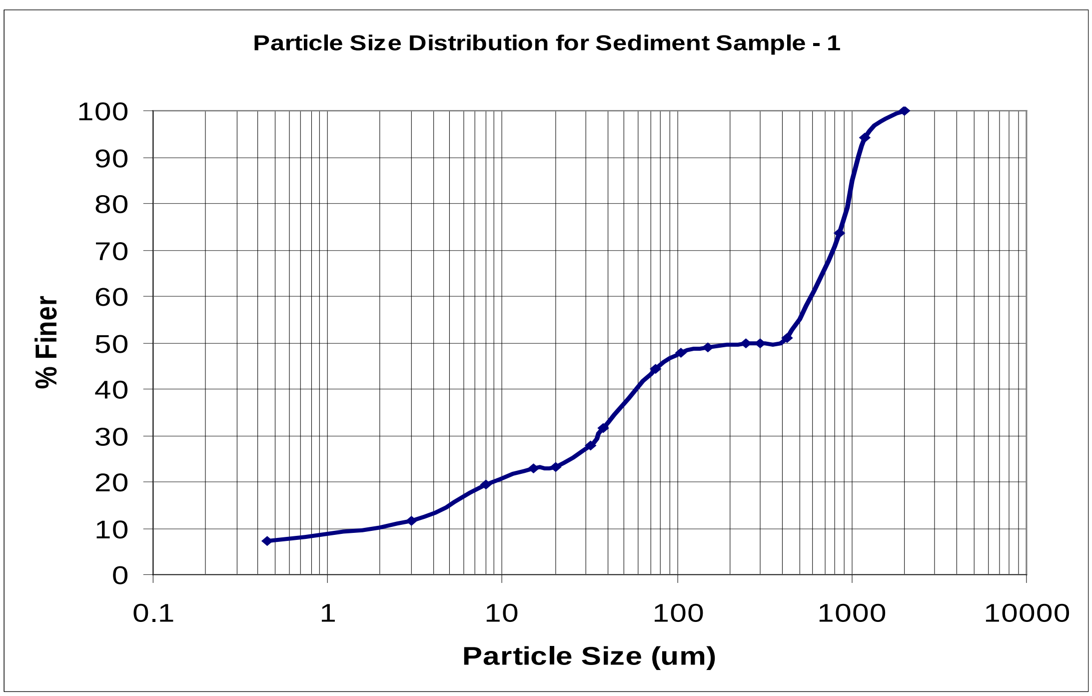


Figure 5.11: PSD curve for test sediment sample 1

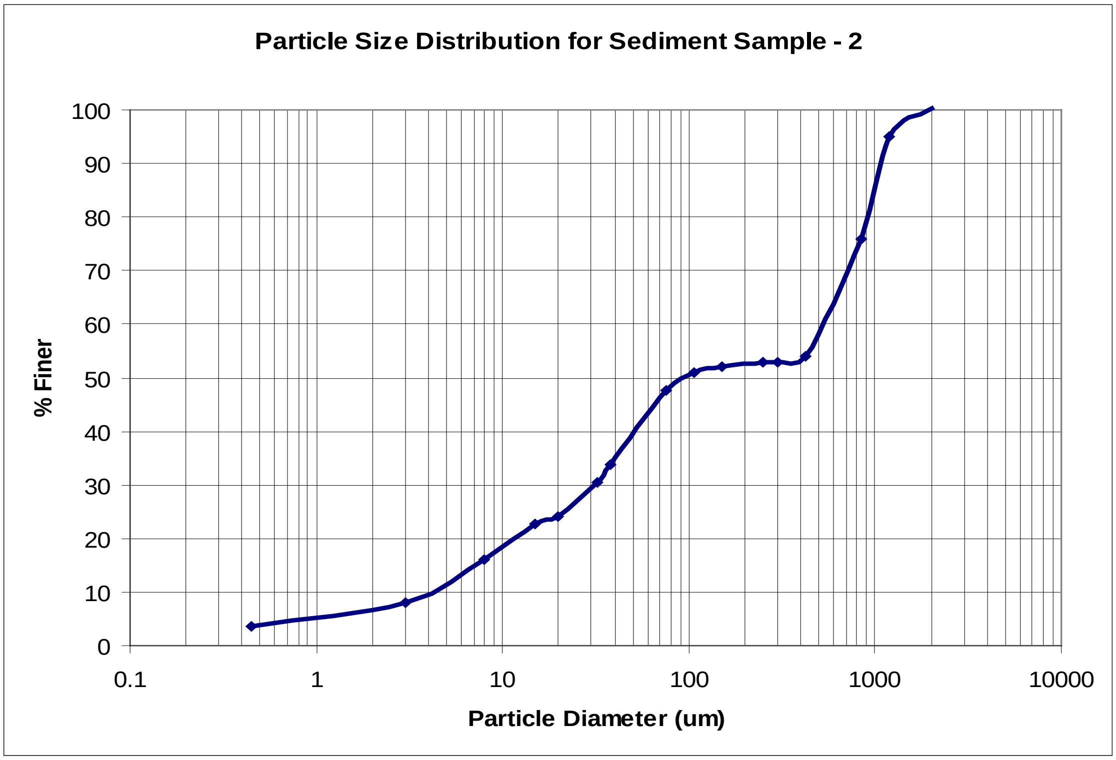


Figure 5.12: PSD curve for test sediment sample 2

5.4 Controlled Test Methodology

A known concentration of approximately 500mg/L particulate solids was used as the influent at three different flow rates representing the highest the filter media could tolerate (high), about half that flow (medium), and about one-forth the maximum flow (low). The solids mixture was made up of a specific combination of ground silica and sieved sand, covering the particle size range from about 0.45 to 2000 µm. The high flows were 46 gpm for the activated carbon media, and 51 gpm for the bone char media. Each experiment was conducted for 30 minutes, during which time measured aliquots of the dry sediment were carefully and constantly poured into the influent “clean” flow from the fire hydrant. An initial blank sample was collected from the upflow effluent location before any sediment was added to measure the background solids in the test water. A sample was collected using a dipper grab sampler every 1 minute and composited in a churn sample splitter for the 30-minute test period. Using the churn splitter, three samples of 1000 ml each were collected for each experiment for laboratory analyses. Samples of the added solids were also collected to verify the particle size distributions.

The sediment was pre-weighed in several 50 mL polyethylene bottles. The sediment was manually feed into the influent water over the whole period of each experiment, according to the desired particulate solids concentration for the specific flow rate for each test. Depth readings of the water levels were also taken during each experiment to determine the head loss for the UpFlow™ filter operation. Also, after completion of each experiment, flow and depth readings were taken to determine the final flow rate and available head to check if they changed during the experiment.

A fire hydrant located close to the test site was used as the influent water source. Before starting any experiment, the highest flow (as bypass just starts for the upflow filter) for the particular media being tested was determined and then the medium and low flow rates were set at about one-half and one-fourth of that highest flow rate. The total amount of sediment required for each experiment was calculated at the beginning of each experiment once the flow rates were determined.

For the mixed media, four different influent sediment concentrations were tested. The concentrations tested were 500 mg/L, 250 mg/L, 100 mg/L, and 50 mg/L. At each one of these four concentrations, three separate experiments were conducted at high, medium, and low flow rates. The highest flow tested for the mixed media was 29 gpm, very close to what is being measured as the overflow/bypass rate observed during actual storms. Figures 5.13 to 5.18 are pictures from the controlled tests.

Figure 5.13: Flow right at the overflow

Figure 5.14: Effluent sample collection

Figure 5.15: The sediment was added manually

Figure 5.16: Sediment wash off

Figure 5.17: Picture showing sediment addition

Figure 5.18: Sample splitting using churn splitter

5.5 Sample Handling and Analysis

A total of 21 separate controlled experiments were conducted resulting in the collection of 84 samples, including the blank samples for each experiment. Total solids, suspended solids, total dissolved solids (by difference), and particle size distribution (PSD) analyses were carried out for each sample and its duplicate. Therefore, the total number of samples analyzed during the controlled tests was 168. Before conducting the analyses, each sample was split into 10 equal volumes of 100 mL each using the Decaport/USGS cone splitter shown in figure 5.19. These split subsamples were analyzed for total solids, suspended solids, and PSD.

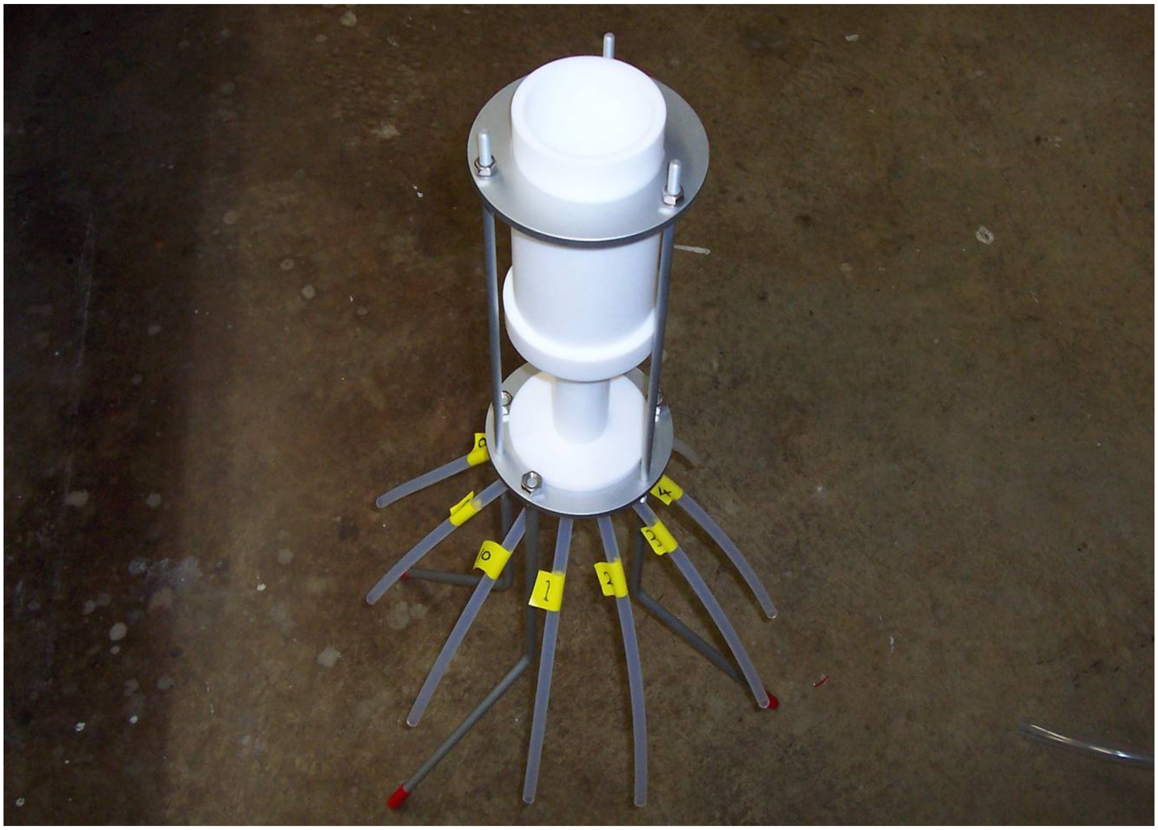


Figure 5.19: Decaport cone splitter

5.4.1 Analysis Results

The maximum flow rates ranged from 20 gal/min for the mixed media to about 50 gal/min for the coarser bone char. The effluent TSS concentrations were lower during lower influent tests compared to the higher concentration tests, indicating that irreducible concentrations were not strictly being observed. Generally, the effluent was better during the lower flow rate tests than for the higher flow tests, but the differences were small.

The percentage reductions for suspended solids for the mixed media tests and high influent concentrations (485 to 492 mg/L) were 84 to 94%, with effluent concentrations ranging from 31 to 79 mg/L for flows ranging from 15 to 30 gal/min. During the low concentration tests (54 to 76 mg/L), the reductions ranged from 68 to 86 mg/L, with effluent concentrations ranging from 11 to 19 mg/L. The coarser bone char and activated carbon media tests had slightly poorer solids removal rates (62 to 79% during the highest flow tests), but with much higher flow rates (46 to 50 gal/min). At flows similar to the mixed media (21 to 28 gal/min), these coarser materials provided similar removals (about 79 to 88% for suspended solids). The flow rates therefore seemed to be more important in determining particulate solids capture than the media type. However, dissolved constituent removals are expected to be enhanced by the mixed media (having the peat component). The results of these tests are presented in Tables 5.3 to 5.6.

Table 5.3: Suspended Solids for mixed media

Media	Flow	Effluent Blank 1/2	Effluent Blank 2/2	sample 1-1	sample 1-2	Sample 2-1	Sample 2-2	Sample 3-1	Sample 3-2	% SS reduction 1	% SS reduction 2	Avg % SS reduction
Mix + Mix 500	High	3	3	79	80	79	79	75	81	85	84.20	84.44
Mix + Mix 500	Mid	2	3	62	60	59	62	10	60	91	87.95	89.71
Mix + Mix 500	Low	10	11	32	32	29	32	30	30	96	95.67	95.78
Mix + Mix 250	High	8	6	51	53	41	50	28	55	86	81.23	83.84
Mix + Mix 250	Mid	7	10	30	32	46	43	45	27	87	89.55	88.25
Mix + Mix 250	Low	3	7	3	3	18	16	10	8	98	98.31	98.03
Mix + Mix 100	High	8	10	34	30	22	11	9	23	85	85.21	85.01
Mix + Mix 100	Mid	13	17	26	22	23	26	11	24	94	89.27	91.66
Mix + Mix 100	Low	8	4	2	6	7	2	6	14	100	98.54	99.27
Mix + Mix 50	High	18	2	2	3	17	16	19	9	94	100.00	96.95
Mix + Mix 50	Mid	8	14	17	17	41	19	7	13	78	88.97	83.46
Mix + Mix 50	Low	17	5	14	24	2	1	15	9	100	99.49	99.74

The influent suspended solids concentration is assumed to be zero. All units in mg/L

Table 5.4: Total Solids for Mixed Media

Media	Flow	Effluent Blank 1/2	Effluent Blank 2/2	sample 1-1	sample 1-2	Sample 2-1	Sample 2-2	Sample 3-1	Sample 3-2	% TS reduction 1	% TS reduction 2	Avg % TS reduction
Mix + Mix 500	High	91	95	175	170	178	175	168	170	83	83.86	83.65
Mix + Mix 500	Mid	93	90	159	163	167	154	114	162	89	85.88	87.23
Mix + Mix 500	Low	112	111	135	134	135	135	131	130	95	95.54	95.47
Mix + Mix 250	High	88	84	125	121	132	130	113	123	85	84.11	84.38
Mix + Mix 250	Mid	75	51	114	101	104	113	122	120	79	80.19	79.78
Mix + Mix 250	Low	82	77	82	93	99	101	98	99	94	92.33	93.32
Mix + Mix 100	High	76	73	95	101	91	106	104	109	73	63.02	68.22
Mix + Mix 100	Mid	61	84	102	104	104	91	111	92	60	72.39	66.43
Mix + Mix 100	Low	96	83	93	93	103	91	97	90	91	97.99	94.52
Mix + Mix 50	High	99	98	96	96	102	100	78	77	100	100.00	100.00
Mix + Mix 50	Mid	85	76	97	105	82	106	98	102	76	50.72	63.12
Mix + Mix 50	Low	90	96	0	99	92	81	88	89	100	100.00	100.00

Table 5.5: Suspended solids for activated carbon, bone char carbon, and Mn coated Zeolite

Media	Flow	Effluent Blank 1/2	Effluent Blank 2/2	sample 1-1	sample 1-2	Sample 2-1	Sample 2-2	Sample 3-1	Sample 3-2	% SS reduction 1	% SS reduction 2	Avg % SS reduction
AC+AC	High	5	8	90	98	98	103	59	89	81	78	79
AC+AC	Mid	5	4	54	58	51	51	46	49	88	87	88
AC+AC	Low	10	7	15	14	20	0	39	0	94	97	95
BC+BC	High	81	86	184	182	190	193	182	209	63	61	62
BC+BC	Mid	101	95	151	149	72	115	123	126	80	77	79
BC+BC	Low	66	66	111	113	115	115	112	116	76	75	75
Zeo+Zeo	High	6	5	14	10	20	27	77	81	92	92	92
Zeo+Zeo	Mid	81	77	84	21	49	58	52	52	89	92	91
Zeo+Zeo	Low	50	40	35	34	37	33	35	39	93	93	93

AC: Activated carbon; BC: Bone char carbon; Zeo: Mn coated zeolite

Table 5.6: Total solids for activated carbon, bone char carbon, and Mn coated zeolite

Media	Flow	Effluent Blank 1/2	Effluent Blank 2/2	sample 1-1	sample 1-2	Sample 2-1	Sample 2-2	Sample 3-1	Sample 3-2	% TS reduction 1	% TS reduction 2	Avg % TS reduction
AC+AC	High	77	81	151	161	158	170	158	161	69	67	68
AC+AC	Mid	77	76	129	124	127	132	125	119	74	75	74
AC+AC	Low	129	83	117	82	92	97	89	95	81	82	81
BC+BC	High	66	93	180	198	178	178	178	181	64	63	64
BC+BC	Mid	102	96	162	168	89	128	126	138	78	75	76
BC+BC	Low	53	71	26	37	55	129	50	99	90	81	86
Zeo+Zeo	High	98	78	111	132	120	138	176	160	76	75	75
Zeo+Zeo	Mid	178	102	184	66	147	123	149	134	74	83	78
Zeo+Zeo	High	156	100	133	134	134	129	121	164	78	76	77

AC: Activated carbon; BC: Bone char carbon; Zeo: Mn coated zeolite

5.6 Performance Plots

Figures 5.20 to 5.21 show the performance plots for the controlled flow Sil-Co-Sil challenge tests. These plots are for the mixed media (Mn-coated zeolite, bone char, and peat mixture) tests which provided maximum flow rates of about 25 gal/min/ft^2 (38 gal/min). During actual storms, treatment rates ranging from 35 to 50 gal/min are being observed for the prototype UpFlow^{TM} filter. Figures 5.22 to 5.27 show the performance plots for activated carbon, bone char carbon and Mn coated zeolite. These plots show excellent control of solids with the prototype UpFlow^{TM} filter for a wide range of flow and concentration conditions. Figures 5.28 to 5.31 show the performance plots of particle size distributions for the different media.

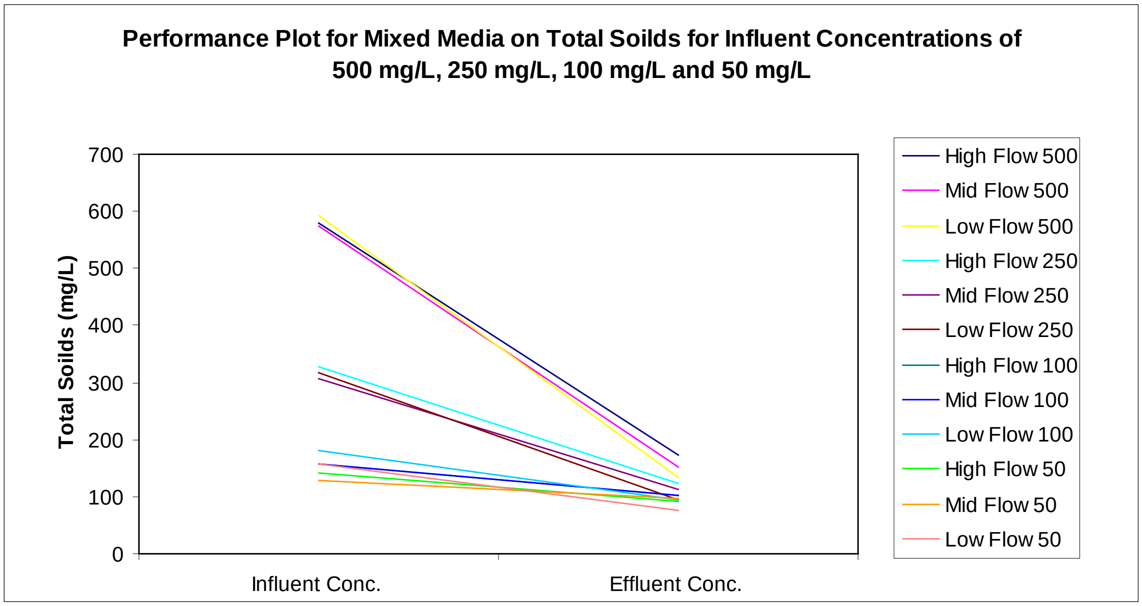


Figure 5.20: Performance plot for mixed media for total solids at influent concentrations of 500 mg/L, 250 mg/L, 100 mg/L and 50 mg/L.

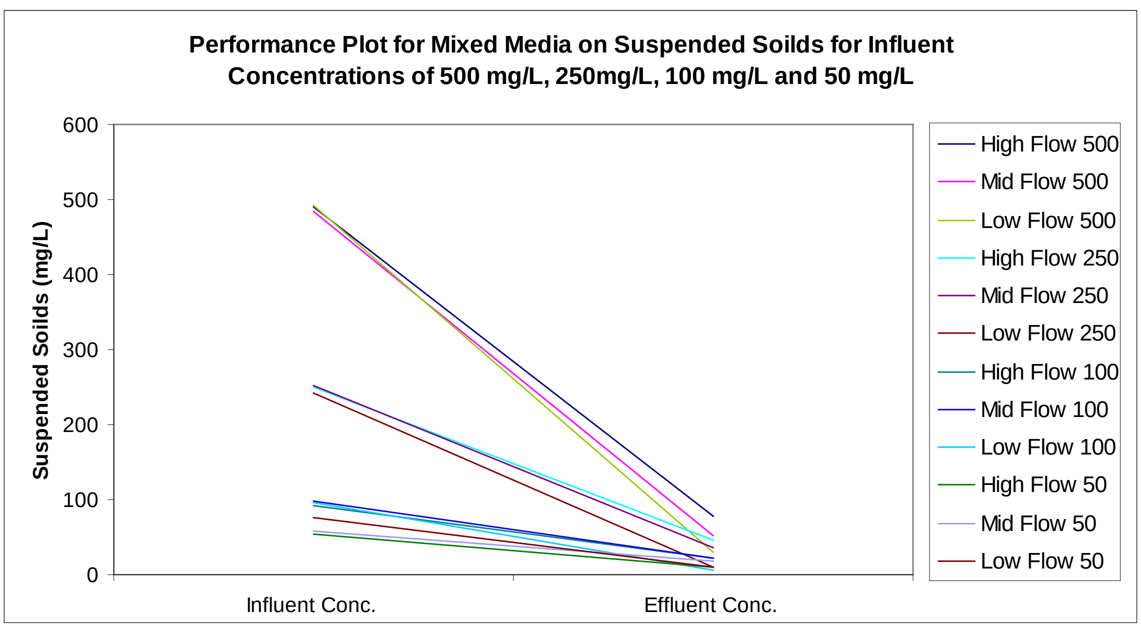


Figure 5.21: Performance plot for mixed media for suspended solids at influent concentrations of 500 mg/L, 250 mg/L, 100 mg/L and 50 mg/L.

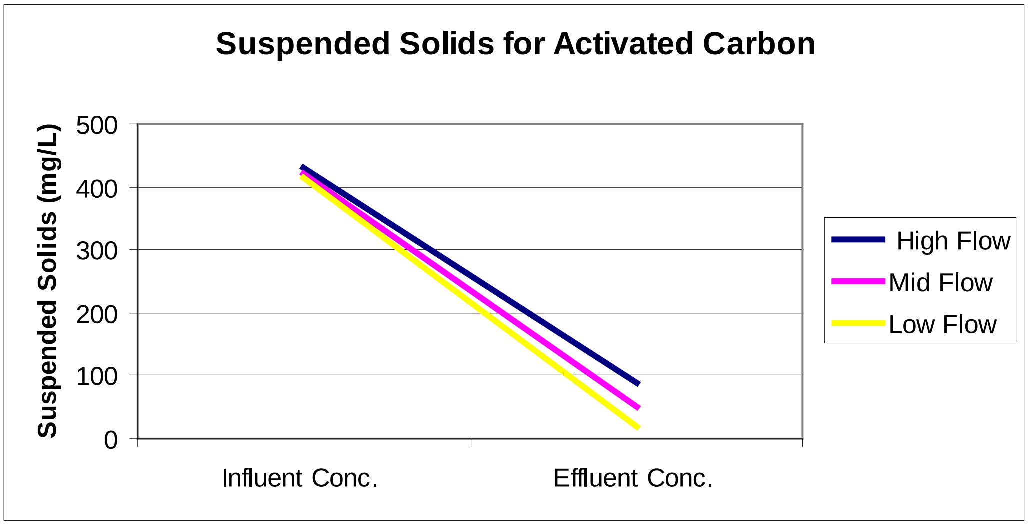


Figure 5.22: Performance of suspended solids for activated carbon

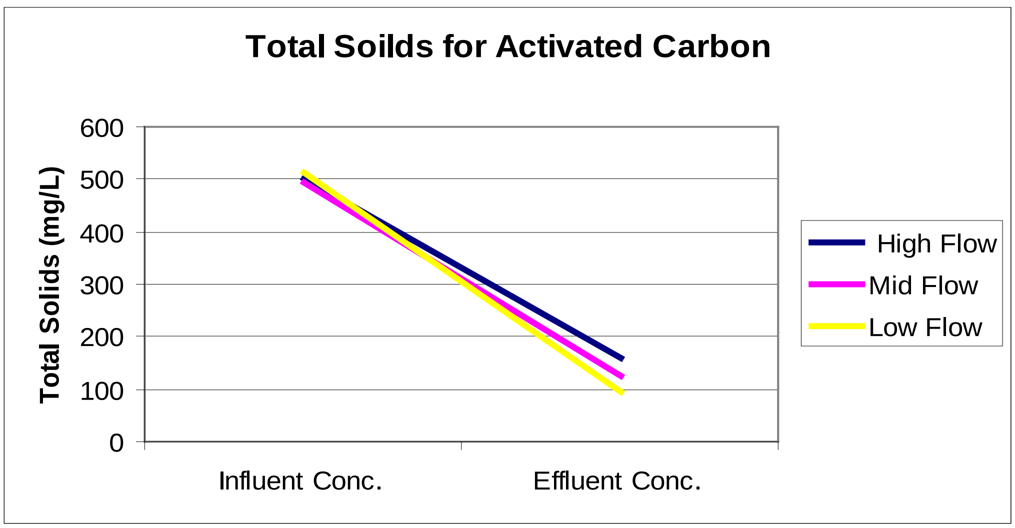


Figure 5.23: Performance of total solids for activated carbon

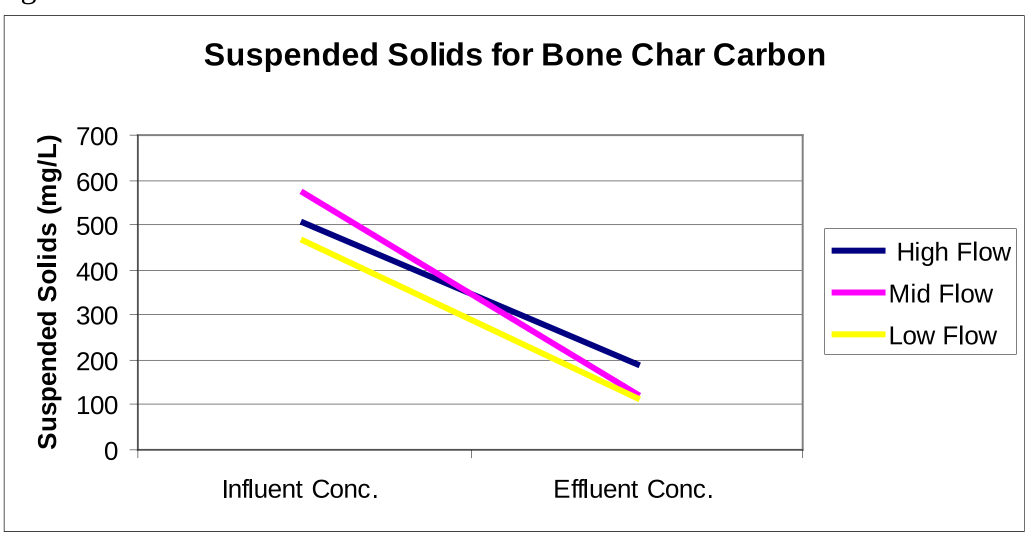


Figure 5.24: Performance of suspended solids for bone char carbon

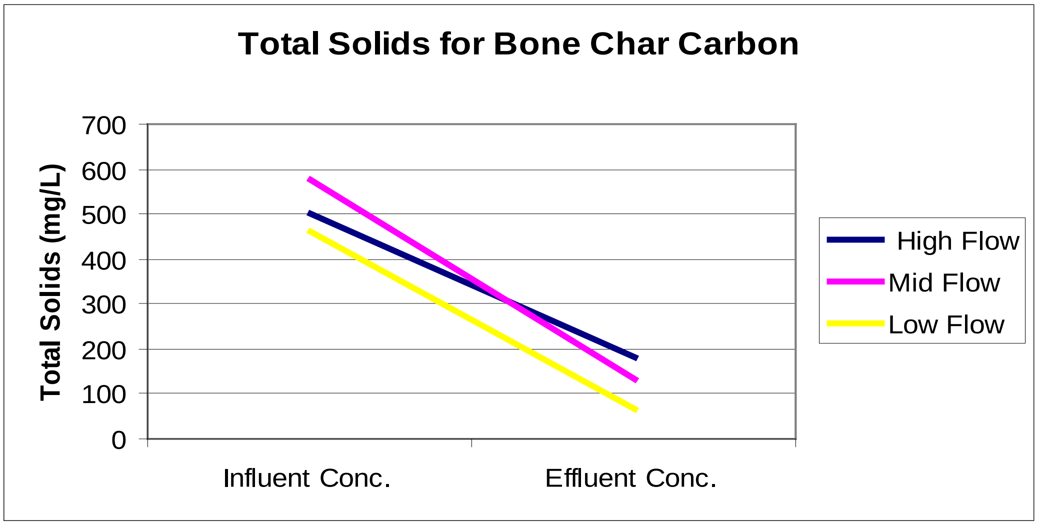


Figure 5.25: Performance of total solids for bone char carbon

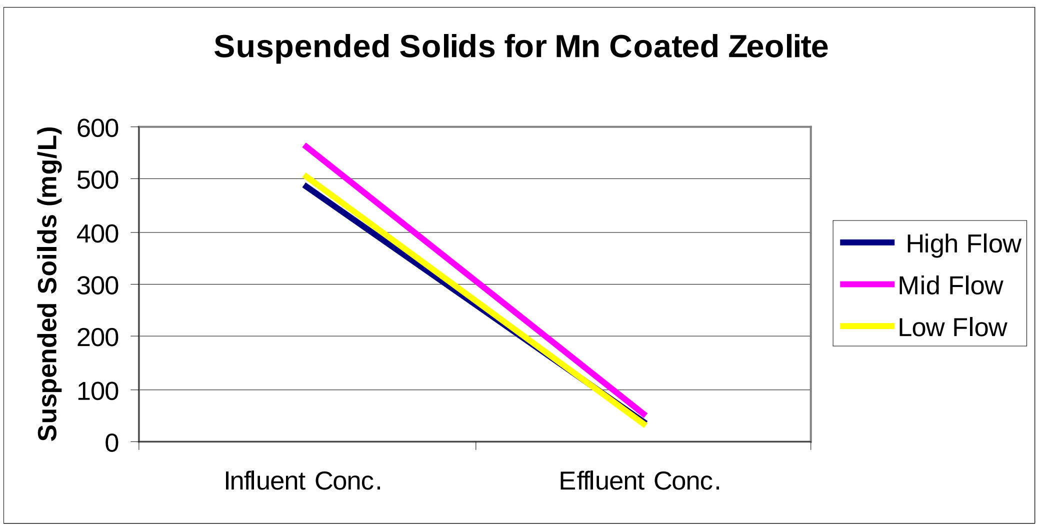


Figure 5.26: Performance of suspended solids for Mn coated zeolite

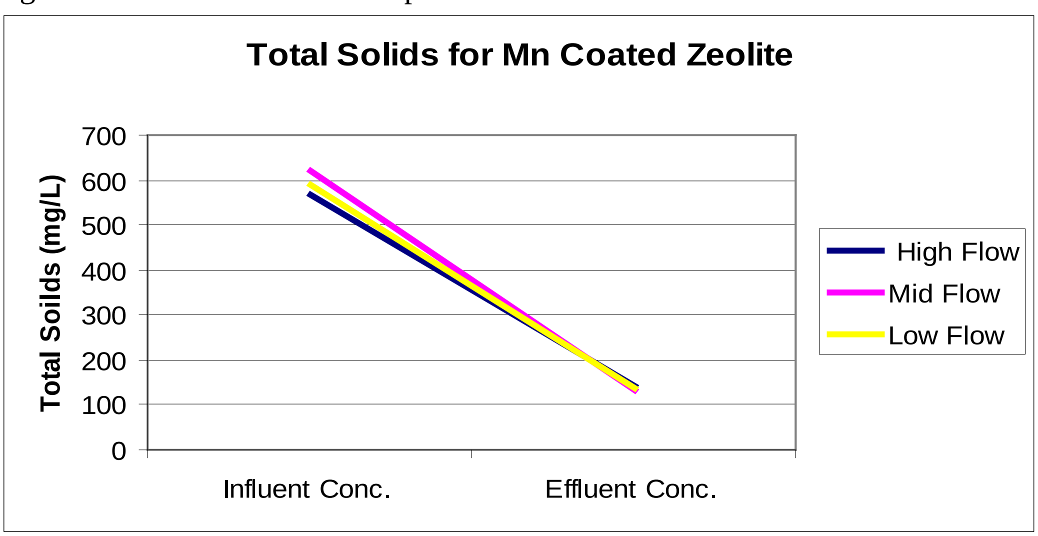


Figure 5.27: Performance of total solids for Mn coated zeolite

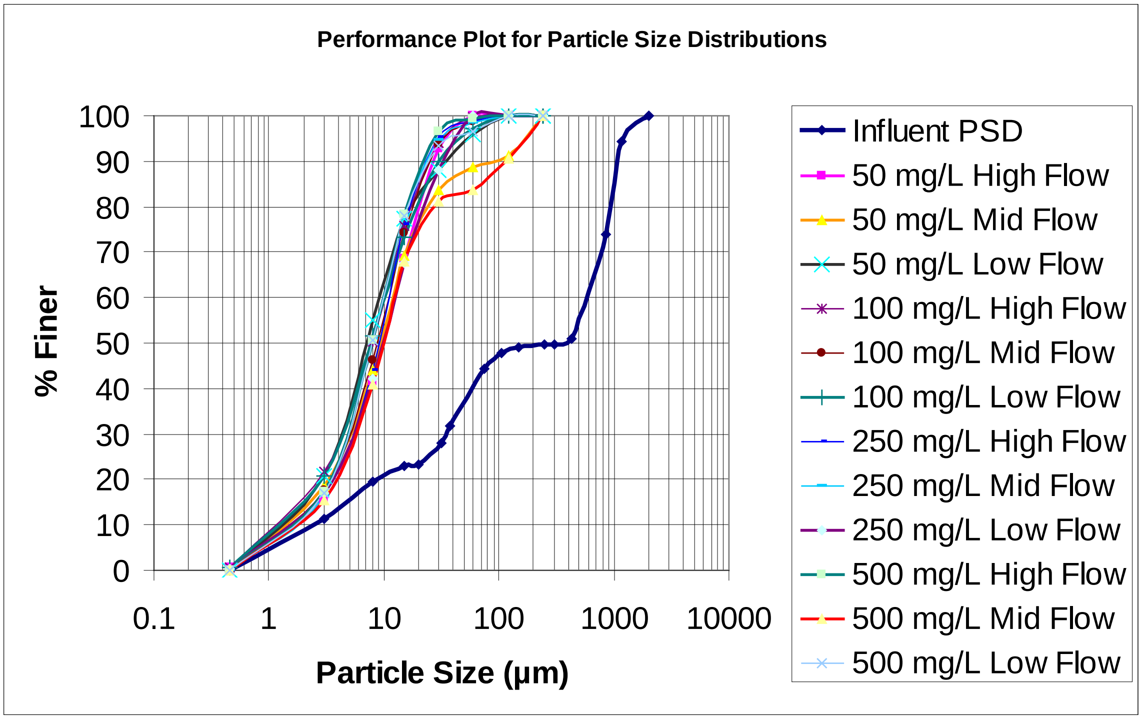


Figure 5.28: Performance plot of particle size distribution for mixed media

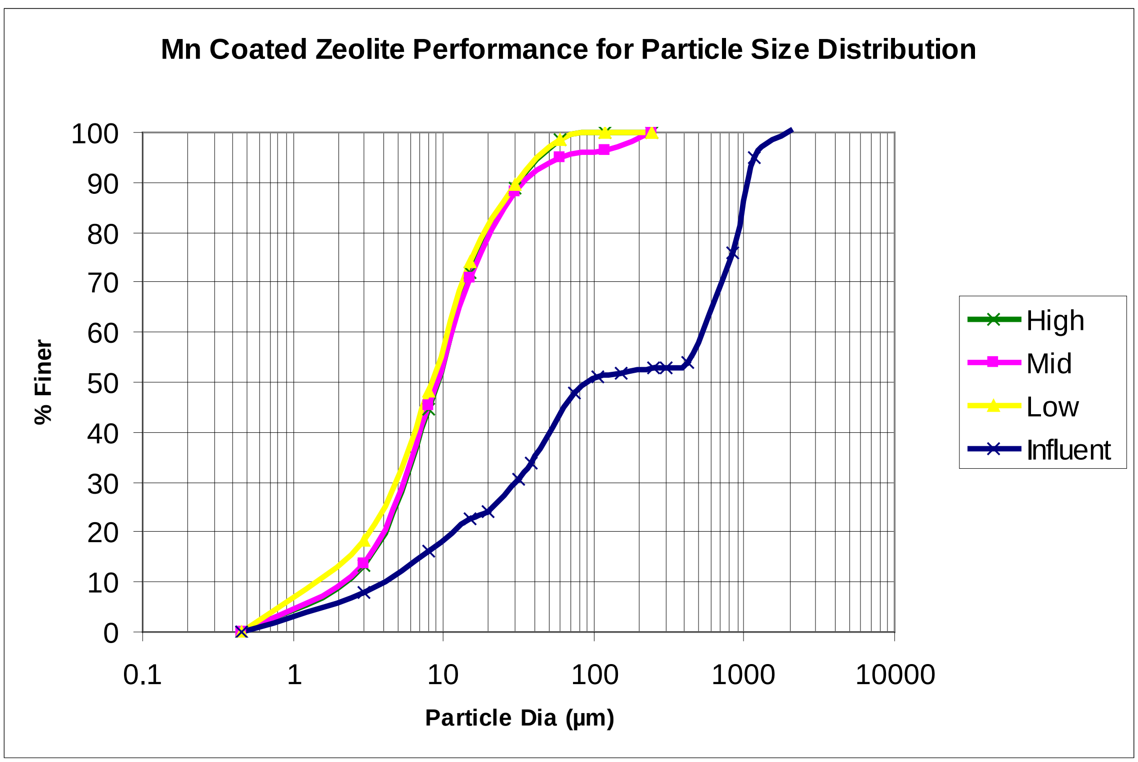


Figure 5.29: Performance plot of particle size distribution for Mn coated zeolite

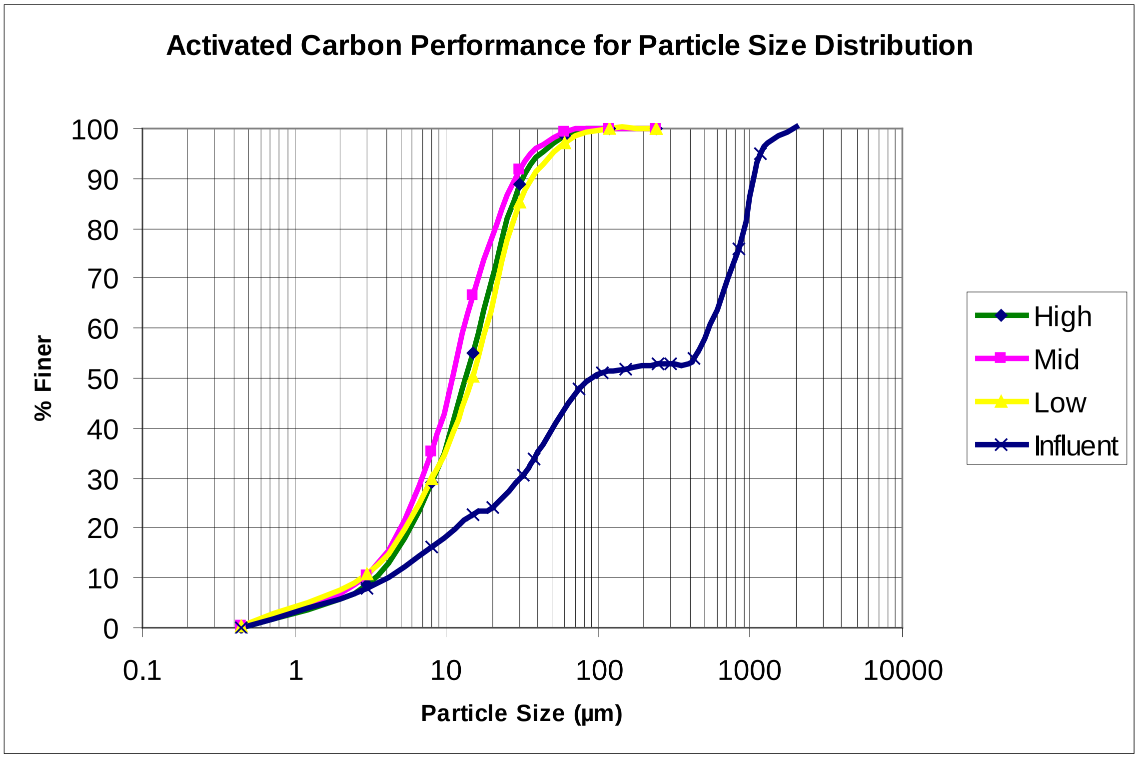


Figure 5.30: Performance plot of particle size distribution for activated carbon

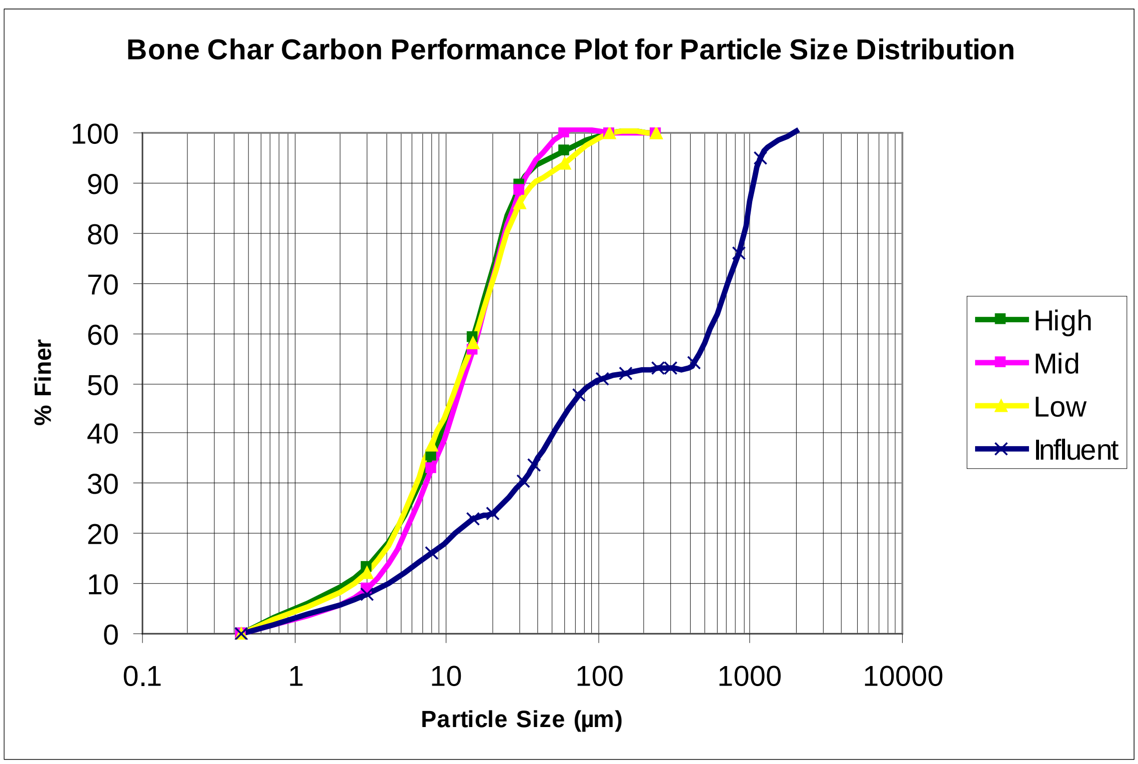


Figure 5.31: Performance plots of particle size distribution for bone char carbon

CHAPTER 6

UPFLOW™ FILTER PERFORMANCE TESTING DURING ACTUAL RAINFALL EVENTS

6.1 Introduction

The intent of these evaluation tests is to quantify the behavior of the UpFlow™ filter during actual rains and to verify the results from the controlled tests. The following thesis sections describe the field sampling methodology, sample selection, sample handling, sample analysis and the results from these monitoring activities.

6.2 Sampling Methodology

Sampling at the test site was conducted using two ISCO 6712 automatic samplers. The flow rates were determined using two ISCO 4250 area-velocity meters which also measured the stage both in the influent sump (the catchbasin sump) and in the effluent pipe. The rainfall intensity and amount was measured using a standard tipping bucket rain gauge. A small totalizing rain gauge was also used as a cross check. YSI 6600 water quality sondes were used to measure the real time water quality data (temperature, dissolved oxygen, pH, ORP, turbidity, conductivity, and water depth) of the influent and the effluent flows at 1minute intervals during storm flows and at 5 minute intervals during interevent periods. The sampling equipment is shown on Figures 6.1 and 6.2. Figure XX, in Chapter XX, are photographs of the installed samplers at the monitoring site. From the known site physical characteristics, measurable runoff is expected after 0.03 inches of rainfall. Once the rain gauge records a rainfall of 0.03 inches (3 tips of the tipping buckets) within a 30 minute period, a pulse is sent to the flow meter which is in turn connected to the automatic samplers to initiate the programmed sampling sequence.

The influent sample is collected from a half-pipe plastic tray (Figure 6.4, 6.7) that has been placed at the catchbasin entrance where the inflowing water cascades from the gutter. The influent real time water quality data is collected using the YSI sonde in the same tray. The tray is emptied after each event and the sonde is placed in a perforated plastic pipe harness to be suspended into the influent sump in order to monitor water quality in the sump between rain events. When an event is expected, the tray is cleaned out, and the sonde is moved from the catchbasin sump back into the tray. The effluent sonde is located at the same place as the effluent sample is collected, at the back side of the divider wall under the cascading water discharged from the UpFlow™ filter. The effluent flow is measured in the effluent pipe exiting from the bottom of the catchbasin. The UpFlow™ filter discharge location is 2.5 feet above the original catchbasin effluent pipe, the sump is therefore 2.5 ft deep.

Figure 6.1: From bottom left to right 1) ISCO 6712 automatic sampler 2) YSI 6600 Sonde 3) ISCO 4250 Flow Meter 4) ISCO 674 Tipping bucket rain gauge 5) YSI Sonde data logger

Figure 6.2: YSI 6600 sondes and data logger, Inset – Close look at the sensors

Figure 6.3: ISCO Propack, Propack holder and Nalgene bottle used to store samples

The setup in the sampler allows us to collect 24 one liter samples. Therefore, the sampler is programmed for sub sampling to enable frequent sampling during the runoff event. In the first one hour of the storm, a 500 mL subsample is collected every 5 minutes, filling a one L bottle every 10 minutes, or 6 bottles in the first hour. After the first hour of sampling, the sampler collects a 250 mL subsample every 15 minutes, filling up one bottle every one hour until the flow ceases, or all the bottles are filled. This sampling configuration allows us to sample continuously for 19 hours. For longer duration storms, additional sampler tubs outfitted with empty sample bottles are replaced with the filled sample tubs. The samplers are automatically disabled if the rainfall is less than 0.03 inches in a 30 minute period. It is common for the samplers to be disabled and enabled several times during a storm. Every time the samplers are re-initiated, they start where they had previously stopped. Both samplers operate together; they both start and stop at the same time. For example, if a sampler was stopped at the start of the 2nd subsample of the 4th bottle, the next subsample would start at the 2nd sub sample of the 4th bottle when the sampler is re-initiated.

Once the samplers stop at the end of the runoff event, sampling reports (Figure 6.5), and data from flow meters and the sondes (Figure 6.6) are collected. Also, all the samples are retrieved from the samplers and placed into clean Nalgene bottles, and the ISCO propacks (Figure 6.3) which are polyethylene one-time use bags, are replaced with new ones in the sampler bases. The samplers are then reset to automatically start at the next storm, along with the sondes. The flow meters need are reset as they continuously take readings.

There are two methods used for selecting appropriate samples for laboratory analyses. Because the monitoring site generates more runoff than can be treated by the prototype UpFlow™ filter, many samples are collected when large amounts of water are bypassing the filtration unit. Only paired samples representing periods when the water was treated are to be evaluated. The first method used to identify sampling periods when runoff was being treated is to use the stage data from the sump flow sensor. The height of the baffle wall and the overflow lip is 49”; hence any sample collected when the height of the water in the catchbasin is more than 49” deep is affected by a blend of treated water and bypassed water and was not analyzed. These samples are identified by overlapping the time data report from the sampler and the stage data. The second method used to identify partially treated water is to use the flow data from the effluent pipe flow sensor. Any flow causing a filter bypass can be directly identified based on the stage-flow relationships of the catchbasin and filter combination. Both of these methods were normally used to identify the samples for analysis. Periodically, higher flows than expected were treated by the UpFlow™ filter. This was possibly due to bed expansion of the filter media and also due to the inherent elasticity of the material used to construct the UpFlow™ filter. Normally, the overflow rates were close to the originally determined treatment capaciy. Once the appropriate samples were selected for analyses, the bacteria testing (*E. coli* and total coliforms) was conducted. In most cases, the bacterial testing started within 6 hrs after the storm ended.

Figure 6.4: Picture showing the influent sample tray and the sonde.

```
SAMPLER ID# 1071576858  18:19  7-JUL-05
  Hardware: A1       Software: 2.10
********** SAMPLING RESULTS **********
   SITE: FBAFFLE001
PROGRAM: CITYHALL02
Program Started at 11:54 WE 29-JUN-05
PART 'A' Nominal Sample Volume =  500 ml
PART 'B' Nominal Sample Volume =  250 ml
                                  COUNT
                                     TO
SAMPLE  BOTTLE TIME  SOURCE ERROR LIQUID
------- ------ ----       -- --  ------
               11:54  'A' DISABLED
               11:54  'B' DISABLED
-------------  TU 05-JUL-05 ------------
               13:43  'A' ENABLED
  1,2     1    13:44  'A' T            0
  2,2     1    13:49  'A' T            0
  1,2     2    13:54  'A' T            0
  2,2     2    13:59  'A' T            0
  1,2     3    14:04  'A' T            0
  2,2     3    14:09  'A' T            0
  1,2     4    14:14  'A' T            0
               14:18  'A' DISABLED
               17:14  MANUAL PAUSE
               17:16  MANUAL RESUME
-------------  WE 06-JUL-05 ------------
               06:16  'A' ENABLED
  2,2     4    06:19  'A' T            0
               06:22  'A' DISABLED
               06:24  'A' ENABLED
  1,2     5    06:29  'A' T            0
  2,2     5    06:34  'A' T            0
  1,2     6    06:39  'A' T            0
  2,2     6    06:44  'A' T            0
               06:44  'A' DONE 06-JUL
               06:45  'B' ENABLED
  1,4     7    07:00  'B' T            0
  2,4     7    07:15  'B' T            0
  3,4     7    07:30  'B' T            0
  4,4     7    07:45  'B' T            0
  1,4     8    08:00  'B' T            0
               08:01  'B' DISABLED
               10:46  'B' ENABLED
  2,4     8    11:01  'B' T            0
               11:10  'B' DISABLED
               11:12  'B' ENABLED
               11:16  'B' DISABLED
               11:21  'B' ENABLED
```

Figure 6.5: Example sampler report

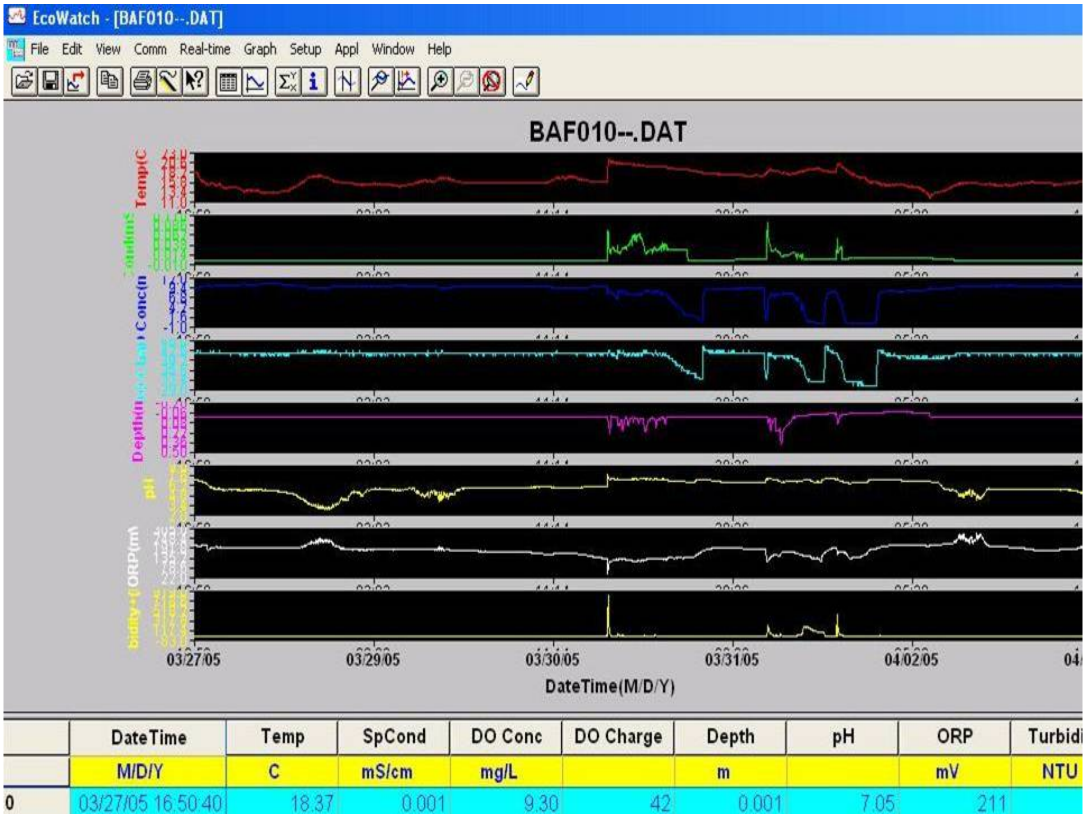


Figure 6.6: Screen shot of sonde data analysis screen

Figure 6.7: Sonde on the effluent side

Table 6.1 Sample volumes required for physical and chemical analyses

S.No	Analytical Parameter	Volume Needed (mL)	Holding Time
1	Total solids	100	n/a
2	Suspended solids	100	n/a
3	E-Coli and Total Coliforms	10 for 1 in 10 dilution 20 for 1 in 5 dilution	6-10 hrs
4	COD	1	28days with acid preservation
5	Ammonia	2	48hrs
6	Nitrates	25	28 days, cool 4^{o} C
7	Phosphorus	5+5	28 days, cool 4^{o} C
8	Microtox toxicity screening	40	7 days, cool 4^{o} C
9	Heavy metals	45	6 months once digested
10	Particle size distribution	50	n/a
	Total Volume Required	**400**	

Once the appropriate samples were selected for analyses, the samples were divided using a Dekaport/USGS cone splitter (Rickly Hydrological Company). As shown in table 6.1, a minimum sample volume of 400 mL was required to conduct the analyses. All the constituents shown in Table 6.1 are measured for both corresponding influent and effluent samples.

Every storm evaluated had a hyetograph (rainfall pattern) and hydrograph (runoff pattern) prepared with the treatment flow capacity marked for that particular event. An example is shown in Figure 6.8.

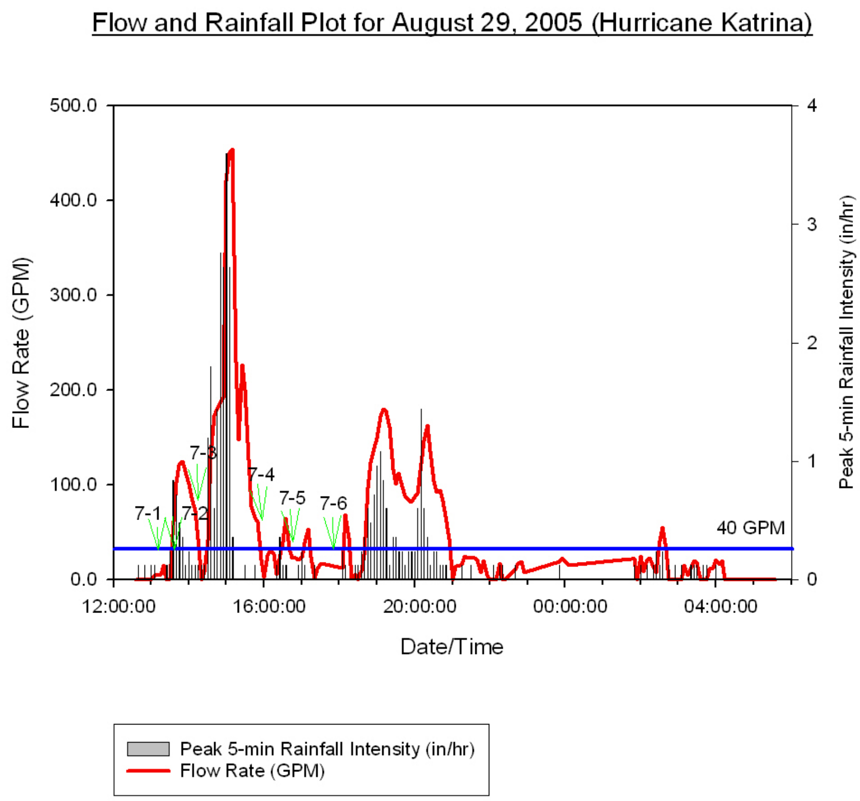


Figure 6.8: Hydrograph and Hyetograph for Hurricane Katrina

6.3 Sampling Results

As expected, the UpFlow™ filter is most effective in reducing pollutants mostly associated with particulate matter and less effective for dissolved constituents. Table 6.2 summarizes the overall performance of the UpFlow™ filter for the 24 sets of samples evaluated. The data summary does not include data where both influent and effluent were below the detection limits, such as happened for some dissolved heavy metals.

Table 6.2: Summary of UpFlow™ filter actual storm event monitoring results

	Average influent concentration (all mg/L, except for bacteria that are #/100 mL, turbidity that is NTU, and metals that are µg/L) (and COV)	Average effluent concentration (all mg/L, except for bacteria that are #/100 mL, turbidity that is NTU, and metals that are µg/L) (and COV)	Calculated percentage removal based on average influent and average effluent concentrations (median of individual sample pair reductions)	Probability that influent ≠ effluent (nonparametric sign test) (significant reduction at 95% level?)
Turbidity (NTU)	43 (2.4)	15 (1.3)	65 (45)	>99% (significant reduction)
Suspended solids	64 (2.9)	19 (1.6)	70 (58)	>99% (significant reduction)
Total solids	137 (1.7)	90 (1.3)	34 (17)	>99% (significant reduction)
COD	111 (1.6)	81 (1.4)	27 (18)	>99% (significant reduction)
Phosphorus	0.94 (1.1)	0.77 (1.4)	18 (13)	98% (significant reduction)
Nitrates	0.7 (1.2)	0.7 (1.3)	0 (0)	93% (not significant reduction)
Ammonia	0.44 (1.5)	0.24 (1.30)	45 (24)	97% (significant reduction)
E. coli	4,750 (0.8)	3,290 (0.8)	31 (21)	>99% (significant reduction)
Total coliforms	12,400 (1.0)	6,560 (0.7)	47 (37)	>99% (significant reduction)
Total Zinc (µg/L)	169 (1.2)	130 (1.3)	23 (23)	>99% (significant reduction)
Dissolved Zinc (µg/L)	103 (0.5)	116 (1.3)	-13 (17)	3.7% (not significant reduction)
Total Copper (µg/L)	13 (1.5)	8.7 (1.2)	33 (26)	64.1% (not significant reduction)
Dissolved	5.7 (0.6)	5.7 (1.0)	0 (35)	97.9% significant

Copper (µg/L)				reduction)
Total Cadmium (µg/L)	1.7 (2.0)	2.6 (3.2)	-53 (-20)	0% (not significant reduction)
Dissolved Cadmium (µg/L)	7.6 (3.5)	2.2 (2.1)	71 (9)	0% (not significant reduction)
Total Lead (µg/L)	15.5 (1.9)	5.5 (1.9)	65 (50)	90.8% (significant reduction)
Dissolved Lead (µg/L)	11.3 (2.7)	2.8 (2.2)	75 (58)	97.8% (significant reduction)
<0.45 µm	0.087 (3.1)	0.69 (4.6)	-690 (60)	90.4% (not significant reduction)
0.45 to 3 µm	4.4 (1.7)	1.7 (1.6)	61 (65)	98.9% (significant reduction)
3 to 12 µm	13.4 (3.3)	3.9 (1.5)	71 (67)	90.7% (not significant reduction)
12 to 30 µm	28.7 (3.6)	6.1 (2.2)	79 (65)	>99% (significant reduction)
30 to 60 µm	12.0 (2.1)	4.5 (1.9)	63 (72)	>99% (significant reduction)
60 to 120 µm	3.1 (1.7)	1.5 (2.0)	52 (47)	97.4% (significant reduction)

Table 6.3 summarizes all the storm events that were sampled. Tables 6.4 through 6.26 present the results for each constituent that was evaluated. These tables only represent simple summaries of the performance of the UpFlow™ Filter, while Appendices B and C show the performance plots, including scatterplots and regression analyses for each constituent. In most cases, significant regression relationships between influent and effluent concentrations were identified, showing strong correlations of performance with influent concentrations. The percentage reduction values shown in the tables in this section are overall averages, while the actual percentage reductions increase substantially as the influent concentrations increase.

Table 6.3: Summary of all the storm events

Storm Number	Date	Total Rain Depth (inches)	Beginning and Ending time of Rain	Rainfall Duration (hrs)	Average Rain Intensity (in/hr)	Peak 5-min Rain Intensity (in/hr)	Instantaneous Peak Runoff Rate (cfs)	Number of Sample Pairs Evaluated per Event
1	26-Mar-05	0.74	7.15pm – 11.25pm	4.17	0.18	1.08	0.27	1
2	1-Apr-05	0.11	3.35pm – 3.55pm	0.3	0.37	0.96	0.28	2
3	26-Apr-05	1.04	4am-6.30am and 9am-12.25pm	5.91	0.17	1.56	0.26	2
4	30-Apr-05	0.64	4.15am – 5.20am	1.08	0.59	3.72	0.31	2
5	5-Jul-05	0.2	2.45pm – 3.30pm	0.75	0.26	0.48	0.13	2
6	6-Jul-05	0.51	7.00am -2.45pm	7.75	0.06	0.24	0.12	5
7	29-Aug-05	3.2	12.35pm – 4.00am (29th aug - 30th aug)	15.42	0.2	3.6	1.01	6
8	16-Sep-05	0.12	1.45pm-2.45pm	1	0.12	0.24	0.17	2
9	25-Sep-05	1.47	5.15pm - 12.00am (25th Sep - 26th Sep)	6.75	0.218	3.96	1	1
10	6-Oct-05	0.11	8.15pm - 11.15pm	3	0.037	0.24	0.037	1
								Total Sample Pairs: 24

Table 6.4: Turbidity Summary for all storm events

Observed Turbidity Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)	% reduc
1-1	15.2	6.76	56
2-1	85.6	42.4	50
2-2	78.6	42.3	46
3-1	8.53	6.95	19
3-2	2.97	1.64	45
4-1	4.93	2.8	43
4-2	2.93	2.16	26
5-1	22.5	8.94	60
5-2	17.5	9.98	43
6-1	8.08	7.33	9
6-2	6.51	5.92	9
6-3	5.35	2.65	50
6-4	7.51	8.15	-9
6-5	3.54	6.45	-82
7-1	44.2	43.7	1
7-2	19.3	10.6	45
7-3	16.5	12.5	24
7-4	5.27	5.21	1
7-5	11.1	5.02	55
7-6	5.15	2.67	48
8-1	502	85.8	83
8-2	35.8	19.9	44
9-1	25.7	2.28	91
10-1	85.7	19	78
min	2.930	1.640	-82
max	502.000	85.800	91
average	42.520	15.046	35
median	13.150	7.140	45
st dev	101.198	19.790	36
COV	2.4	1.3	1

Probability that influent = effluent (nonparametric sign test): 0.00 (>99% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.5: Suspended Solids Summary for storm events

Observed Suspended Solids Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)	% reduc
1-1	17	4	76
2-1	53	36	32
2-2	50	37	26
3-1	6	0	100
3-2	3	1	67
4-1	1	0	100
4-2	1	0	100
5-1	80	37	54
5-2	15	17	-13
6-1	5	6	-20
6-2	11	8	27
6-3	15	13	13
6-4	17	3	82
6-5	21	3	86
7-1	83	36	57
7-2	43	30	30
7-3	29	33	-14
7-4	23	6	74
7-5	5	2	60
7-6	4	4	0
8-1	913	150	84
8-2	41	18	56
9-1	29	2	93
10-1	72	17	76
min	1.000	0.000	-20
max	913.000	150.000	100
average	64.042	19.292	52
median	19.000	7.000	58
st dev	182.538	31.035	38
COV	2.9	1.6	1

Probability that influent = effluent (nonparametric sign test): 0.00 (>99% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.6: Total Solids summary for storm events

Observed Total Solids Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)	% reduc

1-1	69	44	36
2-1	88	75	15
2-2	93	71	24
3-1	41	36	12
3-2	142	38	73
4-1	33	26	21
4-2	25	33	-32
5-1	217	203	6
5-2	124	104	16
6-1	120	111	8
6-2	88	77	13
6-3	60	57	5
6-4	78	43	45
6-5	88	39	56
7-1	178	173	3
7-2	80	65	19
7-3	67	64	4
7-4	29	27	7
7-5	74	37	50
7-6	32	24	25
8-1	1192	580	51
8-2	122	153	-25
9-1	112	72	36
10-1	86	19	78
min	25.000	19.000	-32
max	1192.000	580.000	78
average	134.917	90.458	23
median	87.000	60.500	17
st dev	229.886	114.947	27
COV	1.7	1.3	1

Probability that influent = effluent (nonparametric sign test): 0.00 (>99% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.7: COD summary for storm events

Observed COD Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)	% reduc
1-1	89	37	58
2-1	141	45	68
2-2	137	75	45
3-1	38	23	39
3-2	42	22	48
4-1	20	16	20

4-2	14	8	43
5-1	159	141	11
5-2	75	79	-5
6-1	79	75	5
6-2	62	63	-2
6-3	37	36	3
6-4	27	21	22
6-5	26	22	15
7-1	137	96	30
7-2	61	44	28
7-3	50	32	36
7-4	22	22	0
7-5	32	37	-16
7-6	27	25	7
8-1	891	540	39
8-2	103	112	-9
9-1	76	68	11
10-1	312	310	1
min	14.000	8.000	-16
max	891.000	540.000	68
average	110.708	81.208	21
median	61.500	40.500	18
st dev	178.743	116.009	23
COV	1.6	1.4	1

Probability that influent = effluent (nonparametric sign test): 0.00 (>99% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.8: Phosphorus summary for storm events

Observed Phosphorus Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)	% reduc
1-1	0.35	0.42	-20
2-1	0.64	0.6	6
2-2	0.25	0.25	0
3-1	0.58	0.2	66
3-2	0.15	0.03	80
4-1	0.08	0.07	13
4-2	1.06	1.14	-8
5-1	0.76	0.63	17
5-2	0.31	0.39	-26
6-1	0.27	0.34	-26
6-2	0.28	0.15	46

6-3	0.39	0.15	62
6-4	0.16	0.22	-38
6-5	1.88	1.62	14
7-1	0.69	0.5	28
7-2	1.32	0.66	50
7-3	0.29	0.28	3
7-4	0.76	0.45	41
7-5	0.51	0.18	65
7-6	3.14	3.14	0
8-1	0.74	0.58	22
8-2	3.12	4.54	-46
9-1	1.04	0.96	8
10-1	3.81	0.9	76
min	0.080	0.030	-46
max	3.810	4.540	80
average	0.941	0.767	18
median	0.610	0.435	13
st dev	1.027	1.038	36
COV	1.1	1.4	2

Probability that influent = effluent (nonparametric sign test): 0.02 (98% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.9: Nitrates summary for storm events

Observed Nitrates Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)	% reduc
1-1	0.5	0.5	0
2-1	0.4	0.4	0
2-2	0.5	0.4	20
3-1	0.3	0.5	-67
3-2	0.4	0.5	-25
4-1	0.3	0.2	33
4-2	0.2	0.2	0
5-1	2.7	3.8	-41
5-2	2.5	1.6	36
6-1	0.5	0.8	-60
6-2	0.4	0.3	25
6-3	0.3	0.1	67
6-4	0.1	0.1	0
6-5	0.1	0.1	0
7-1	0.5	0.4	20
7-2	0.1	0.1	0
7-3	0.1	0.2	-100
7-4	0	0	n/a
7-5	0.1	0.5	-400
7-6	0.4	0.3	25
8-1	2.4	2.9	-21
8-2	2	2	0
9-1	0.7	0.5	29
10-1	1.2	0.4	67
min	0.000	0.000	-400
max	2.700	3.800	67
average	0.696	0.700	-17
median	0.400	0.400	0
st dev	0.824	0.945	93
COV	1.2	1.3	-5

Probability that influent = effluent (nonparametric sign test): 0.07 (93% confident that influent ≠ effluent), therefore not statistically significant at least at the α 0.05 level.

Table 6.10: Ammonia summary for storm events

Observed Ammonia Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)	% reduc
1-1	0.21	0.91	-333

2-1	0.23	0.21	9
2-2	0.27	0.39	-44
3-1	0.16	0.1	38
3-2	0.5	0.01	98
4-1	0.04	0.02	50
4-2	0.01	0.03	-200
5-1	0.19	0.31	-63
5-2	0.31	0.03	90
6-1	0.17	0.13	24
6-2	0.14	0.11	21
6-3	0.07	0.05	29
6-4	0.05	0.07	-40
6-5	0.03	0.04	-33
7-1	1.47	1.14	22
7-2	0.58	0.42	28
7-3	0.49	0.31	37
7-4	0.47	0.08	83
7-5	0.53	0.1	81
7-6	0	0.02	n/a
8-1	2.9	0.22	92
8-2	0.99	0.87	12
9-1	0.2	0.02	90
10-1	0.01	0.26	-2500
min	0.000	0.010	-2500
max	2.900	1.140	98
average	0.418	0.244	-105
median	0.205	0.105	24
st dev	0.630	0.310	531
COV	1.5	1.3	-5

Probability that influent = effluent (nonparametric sign test): 0.03 (97% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.11: Comparison of E. coli for storm events

Observed E. coli Counts

Sample Number	Influent (#/100 mL)	Effluent (#/100 mL)	% reduc
1-1	1466	1218	17
2-1	1202	1182	2
2-2	1390	1076	23
3-1	2656	2277	14
3-2	1508	808	46
4-1	3877	1937	50
4-2	2750	2430	12
5-1	3540	1890	47
5-2	3458	3122	10
6-1	1236	1457	-18
6-2	1187	2419	-104
6-3	1935	1011	48
6-4	1725	1918	-11
6-5	1515	1455	4
7-1	10560	7180	32
7-2	9300	7460	20
7-3	10240	6540	36
7-4	9740	5560	43
7-5	7580	2180	71
7-6	9140	2420	74
8-1	9814	8166	17
8-2	9442	10198	-8
9-1	4084	3024	26
10-1	2745	1952	29
min	1187.000	808.000	-104
max	10560.000	10198.000	74
average	4670.417	3286.667	20
median	3104.000	2228.500	21
st dev	3604.267	2666.438	35
COV	0.8	0.8	2

Probability that influent = effluent (nonparametric sign test): 0.00 (>99% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.12: Comparison of Total Coliform for storm events

Observed Total Coliforms Counts

Sample Number	Influent (#/100 mL)	Effluent (#/100 mL)	% reduc
1-1	48384	20924.8	57
2-1	7754	10950	-41
2-2	22397	12262	45
3-1	7701	4786	38
3-2	9208	2209	76
4-1	2656	2277	14
4-2	3877	1937	50
5-1	14540	9768	33
5-2	6628	5944	10
6-1	4352	3448	21
6-2	6131	4701	23
6-3	5794	4106	29
6-4	4160	2382	43
6-5	6867	4386	36
7-1	7270	1220	83
7-2	14136	11620	18
7-3	17328.7	6910	60
7-4	5172	5468	-6
7-5	12996.5	5172	60
7-6	7270	4106	44
8-1	48394	10670	78
8-2	19212	13820	28
9-1	7622	5026	34
10-1	7770	3440	56
min	2656.000	1220.000	-41
max	48394.000	20924.800	83
average	12400.842	6563.867	37
median	7661.500	4906.000	37
st dev	12192.521	4741.179	28
COV	1.0	0.7	1

Probability that influent = effluent (nonparametric sign test): 0.00 (>99% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.13: Comparison of Total Zinc for storm events

Observed Total Zinc Concentrations

Sample Number	Influent (ug/L)	Effluent (ug/L)	% reduc

1-1	118	81	31
2-1	123	95	23
5-1	267	221	17
5-2	168	147	13
6-1	101	85	16
6-2	86	70	19
6-3	83	73	12
6-4	95	60	37
6-5	100	35	65
7-1	124	109	12
7-2	125	73	42
7-3	62	44	29
7-4	27	17	37
7-5	30	72	-140
7-6	64	46	28
8-1	990	785	21
8-2	277	298	-8
9-1	139	38	73
10-1	237	115	51
min	27.000	17.000	-140
max	990.000	785.000	73
average	169.263	129.684	20
median	118.000	73.000	23
st dev	210.900	172.435	43
COV	1.2	1.3	2

Probability that influent = effluent (nonparametric sign test): 0.0007 (>99% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.14: Comparison of dissolved zinc for storm events

Observed Dissolved Zinc Concentrations

Sample Number	**Influent (ug/L)**	**Effluent (ug/L)**	**% reduc**
1-1	84.8	73.9	13
5-1	227.4	177.8	22
5-2	137.9	132	4
6-1	97.3	68.3	30
6-2	109.8	50.9	54
6-3	80.3	84.6	-5
6-4	87.5	60.6	31
6-5	73	54.8	25
7-1	118.2	110.8	6
7-2	127	72.8	43
7-3	61.8	54.3	12
7-4	53.9	40.7	24
7-5	49.4	74.8	-51
7-6	44.2	15.8	64
8-1	135.4	679.6	-402

8-2	156.5	196.57	-26
9-1	32	45.1	-41
10-1	183.7	100.7	45
min	32.000	15.800	-402
max	227.400	679.600	64
average	103.339	116.337	-8
median	92.400	73.350	17
st dev	51.716	148.059	103
COV	0.5	1.3	-12

Probability that influent = effluent (nonparametric sign test): 0.963 (4.7% confident that influent ≠ effluent), therefore not statistically significant at least at the α 0.05 level.

Table 6.15: Comparison of Total Copper for storm events

Observed Total Copper Concentrations

Sample Number	Influent (ug/L)	Effluent (ug/L)	% reduc
1-1	4.6	2.7	41
2-1	9.7	5.2	46
5-1	25.5	18.9	26
5-2	13	16.5	-27
6-1	18	10.2	43
6-2	6.8	7.9	-16
6-3	3.4	2.9	15
6-4	4.4	2.4	45
6-5	19	0.9	95
7-1	6	9.4	-57
7-2	0.7	4	-471
7-3	0.003	3.1	-103233
7-4	0.003	2.1	-69900
7-5	0.003	2.7	-89900
7-6	0.4	0.1	75
8-1	85.8	48.2	44
8-2	26.3	11.5	56
9-1	11.2	7	38
10-1	11	9.7	12
min	0.003	0.100	-103233
max	85.800	48.200	95
average	12.937	8.705	-13846
median	6.800	5.200	26
st dev	19.512	10.877	33319
COV	1.5	1.2	-2

Probability that influent = effluent (nonparametric sign test): 0.3593 (64.1% confident that influent ≠ effluent), therefore not statistically significant at least at the α 0.05 level.

Table 6.16: Comparison of dissolved copper for storm events

Observed Dissolved Copper Concentrations

Sample Number	Influent (ug/L)	Effluent (ug/L)	% reduc
1-1	3.43	2.35	31
5-1	9.73	14.07	-45
5-2	12.5	7.27	42
6-1	10.18	9.83	3
6-2	6.49	3.9	40
6-3	5.2	3.32	36
6-4	4.72	3	36
6-5	2.67	0.98	63
7-1	4.08	9.44	-131
7-2	9.85	1.77	82
7-3	1.73	0.0034	100
7-6	1.32	0.0034	100
8-1	0.15	22.07	-14613
8-2	6.24	5.76	8
9-1	4.63	3.02	35
10-1	7.55	5.2	31
min	0.150	0.003	-14613
max	12.500	22.070	100
average	5.654	5.749	-886
median	4.960	3.610	35
st dev	3.551	5.825	3661
COV	0.6	1.0	-4

Probability that influent = effluent (nonparametric sign test): 0.0213 (97.9% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.17: Comparison of Total Cadmium for storm events

Observed Total Cadmium Concentrations

Sample Number	Influent (ug/L)	Effluent (ug/L)	% reduc
1-1	1.23	0.76	38
2-1	0.54	0.39	28
5-1	10	0.039	100
5-2	0.0008	29.92	-3739900
6-5	7.82	0.0008	100
7-1	0.0008	0.156	-19400
7-2	0.0008	0.24	-29900
7-5	0.0008	0.058	-7150
7-6	0.17	0.44	-159
8-1	1.15	0.95	17

8-2	0.058	0.2	-245
9-1	0.26	0.008	97
10-1	0.3	0.36	-20
min	0.001	0.001	-3739900
max	10.000	29.920	100
average	1.656	2.579	-292030
median	0.260	0.240	-20
st dev	3.276	8.220	1035998
COV	2.0	3.2	-4

Probability that influent = effluent (nonparametric sign test): 1 (0% confident that influent ≠ effluent), therefore not statistically significant at least at the α 0.05 level.

Table 6.18: Comparison of dissolved cadmium for storm events

Observed Dissolved Cadmium Concentrations

Sample Number	Influent (ug/L)	Effluent (ug/L)	% reduc
1-1	1.06	0.96	9
5-1	1.88	0.49	74
5-2	0.0023	0.0008	65
6-2	0.26	0.0008	100
6-3	0.0008	0.54	-67400
6-4	1.03	0.53	49
6-5	0.0008	0.65	-81150
7-1	109.1	2.23	98
7-2	0.86	1.06	-23
7-3	1.64	20.32	-1139
7-4	5.52	1.88	66
7-5	0.11	3.83	-3382
7-6	1.9	0.99	48
8-1	0.82	0.88	-7
8-2	0.067	0.16	-139
9-1	1.93	3.4	-76
10-1	2.46	0.13	95
min	0.001	0.001	-81150
max	109.100	20.320	100
average	7.567	2.238	-8983
median	1.030	0.880	9
st dev	26.200	4.793	24710
COV	3.5	2.1	-3

Probability that influent = effluent (nonparametric sign test): 1 (0% confident that influent ≠ effluent), therefore not statistically significant at least at the α 0.05 level.

Table 6.19: Comparison of Total Lead for storm events

Observed Total Lead Concentrations

Sample Number	Influent (ug/L)	Effluent (ug/L)	% reduc
1-1	31.68	16.17	49
2-1	11.44	7.26	37
5-1	24.16	0.0062	100
7-1	1.5	1.51	-1
7-2	2.64	0.65	75
7-3	1.19	0.345	71
7-4	0.58	2.6	-348
7-5	0.012	0.68	-5567
7-6	1.6	0.91	43
8-1	107.4	36.29	66
8-2	8.67	4.35	50
9-1	7.9	0.47	94
10-1	3	0.8	73
min	0.012	0.006	-5567
max	107.400	36.290	100
average	15.521	5.542	-404
median	3.000	0.910	50
st dev	29.271	10.257	1555
COV	1.9	1.9	-4

Probability that influent = effluent (nonparametric sign test): 0.0923 (90.8% confident that influent ≠ effluent), therefore not statistically significant at least at the α 0.05 level.

Table 6.20: Comparison of dissolved lead for storm events

Observed Dissolved Lead Concentrations

Sample Number	Influent (ug/L)	Effluent (ug/L)	% reduc
1-1	20.3	19.5	4
5-1	112	0.0062	100
5-2	0.23	0.0062	97
7-1	0.0062	1.13	-18126
7-2	0.3	0.3	0
7-3	2.83	0.32	89
7-4	0.43	0.18	58
7-5	0.53	0.36	32
7-6	1.06	0.26	75
8-1	1.22	13.2	-982
8-2	6.23	1.05	83
9-1	0.74	0.0062	99
10-1	0.58	0.28	52
min	0.006	0.006	-18126
max	112.000	19.500	100
average	11.266	2.815	-1417
median	0.740	0.300	58
st dev	30.764	6.153	5029
COV	2.7	2.2	-4

Probability that influent = effluent (nonparametric sign test): 0.0225 (97.8% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.21: Comparison of 0 to 0.45 μm particles for storm events

Observed 0 to 0.45 μm Particle Size Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)	% reduc
1-1	0	0	n/a
2-1	0	15.67	n/a
2-2	0	0	na/
3-1	0.0072	0	100
3-2	0.0036	0.0036	0
4-1	0.0014	0	100
4-2	0.0071	0	100
5-1	0.064	0.33	-416
5-2	0.055	0.03	45
6-1	0.003	0.0102	-240
6-2	0.055	0.0008	99
6-3	0.0345	0.0299	13
6-4	0.1207	0.0129	89
6-5	0.0252	0.015	40
7-1	0.1245	0.0216	83
7-2	0.0387	0.096	-148
7-3	0.00638	0.1122	-1659
7-4	0.0667	0.0066	90
7-5	0.0045	0.0014	69
7-6	0.024	0.0136	43
8-1	0.0821	0.033	60
8-2	0.036	0.0079	78
9-1	0.0017	0.0028	-65
10-1	1.325	0.2057	84
min	0.000	0.000	-1659
max	1.325	15.670	100
average	0.087	0.692	-68
median	0.025	0.012	60
st dev	0.266	3.191	387
COV	3.1	4.6	-6

Probability that influent = effluent (nonparametric sign test): 0.0963 (90.4% confident that influent ≠ effluent), therefore not statistically significant at least at the α 0.05 level.

Table 6.22: Comparison of 0.45 to 3 μm particles for storm events

Observed 0.45 to 3 μm Particle Size Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)	% reduc
1-1	2.0451	0.9596	53
2-1	8.6655	4.7268	45
2-2	7.95	4.2217	47

3-1	0.2448	0	100
3-2	0.22401	0.1153	49
4-1	0.0501	0	100
4-2	0.0145	0	100
5-1	1.712	12.3728	-623
5-2	3.2805	0.5814	82
6-1	0.197	1.3092	-565
6-2	2.2759	0.1904	92
6-3	1.365	0.5993	56
6-4	3.1331	0.792	75
6-5	0.8652	0.3618	58
7-1	5.4448	1.836	66
7-2	1.788	2.62	-47
7-3	0.62	1.36	-119
7-4	4.8783	0.3114	94
7-5	0.3255	0.0632	81
7-6	0.3368	0.1232	63
8-1	18.8169	3.612	81
8-2	7.2988	0.6994	90
9-1	0.209	0.08	62
10-1	34.42	3.8114	89
min	0.015	0.000	-623
max	34.420	12.373	100
average	4.423	1.698	5
median	1.750	0.649	65
st dev	7.672	2.707	191
COV	1.7	1.6	35

Probability that influent = effluent (nonparametric sign test): 0.0106 (98.9% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.23: Comparison of 3 to 12 µm particles for storm events

Observed 3 to 12 µm Particle Size Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)	% reduc
1-1	5.96	1.12	81
2-1	3.62	1.74	52
2-2	3.105	3.45	-11
3-1	0.68	0	100
3-2	0.48	0.17	65
4-1	0.15	0	100
4-2	0.124	0	100
5-1	8.58	2.78	68
5-2	5.32	5.74	-8
6-1	0.551	2.03	-268
6-2	2.02	0.519	74
6-3	4.866	2.36	52
6-4	5.505	1	82

6-5	3.28	0.617	81
7-1	11.93	15.43	-29
7-2	6.65	8.67	-30
7-3	3.15	7.61	-142
7-4	5.41	1.33	75
7-5	1.27	0.25	80
7-6	0.58	0.25	57
8-1	221.44	23.71	89
8-2	11.67	3.93	66
9-1	1.39	0.085	94
10-1	13.92	10.25	26
min	0.124	0.000	-268
max	221.440	23.710	100
average	13.402	3.877	36
median	3.450	1.535	67
st dev	44.485	5.761	86
COV	3.3	1.5	2

Probability that influent = effluent (nonparametric sign test): 0.0931 (90.7% confident that influent ≠ effluent), therefore not statistically significant at least at the α 0.05 level.

Table 6.24: Comparison of 12 to 30 µm particles for storm events

Observed 12 to 30 µm Particle Size Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)	% reduc
1-1	3.36	1.04	69
2-1	2.91	9.23	-217
2-2	2.8	3.63	-30
3-1	2.23	0	100
3-2	0.8	0.14	83
4-1	0.344	0	100
4-2	0.252	0	100
5-1	36.86	6.83	81
5-2	3.97	8.45	-113
6-1	1.961	1.45	26
6-2	3.193	2.135	33
6-3	5.3025	3.804	28
6-4	4.39	0.423	90
6-5	6.95	1.045	85
7-1	8.17	2.92	64
7-2	24.37	10.11	59
7-3	12.26	14.33	-17
7-4	7.25	2.54	65
7-5	1.49	0.753	49
7-6	0.92	0.82	11
8-1	515.53	66.82	87
8-2	14.61	9.04	38
9-1	13.02	0.453	97

10-1	15.732	1.266	92
min	0.252	0.000	-217
max	515.530	66.820	100
average	28.695	6.135	41
median	4.180	1.793	65
st dev	104.054	13.528	75
COV	3.6	2.2	2

Probability that influent = effluent (nonparametric sign test): 0.0026 (99.7% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.25: Comparison of 30 to 60 μm particles for storm events

Observed 30 to 60 μm Particle Size Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)	% reduc
1-1	4.5883	0.848	82
2-1	25.0902	4.086	84
2-2	24.1	19.758	18
3-1	2.1594	0	100
3-2	0.879	0.1841	79
4-1	0.394	0	100
4-2	0.6026	0	100
5-1	16.16	1.8796	88
5-2	2.1075	2.0196	4
6-1	2.0645	1.0194	51
6-2	2.8721	3.6696	-28
6-3	2.7885	4.1717	-50
6-4	3.4952	0.3618	90
6-5	6.8712	0.9606	86
7-1	16.3095	6.2028	62
7-2	8.7935	5.889	33
7-3	9.9934	7.9233	21
7-4	5.3866	1.3428	75
7-5	1.205	0.7338	39
7-6	1.7272	1.7772	-3
8-1	128.0939	38.13	70
8-2	6.0024	3.8394	36
9-1	10.092	0.9374	91
10-1	5.3928	1.4297	73
min	0.394	0.000	-50
max	128.094	38.130	100
average	11.965	4.465	54
median	4.987	1.603	72
st dev	25.706	8.296	42
COV	2.1	1.9	1

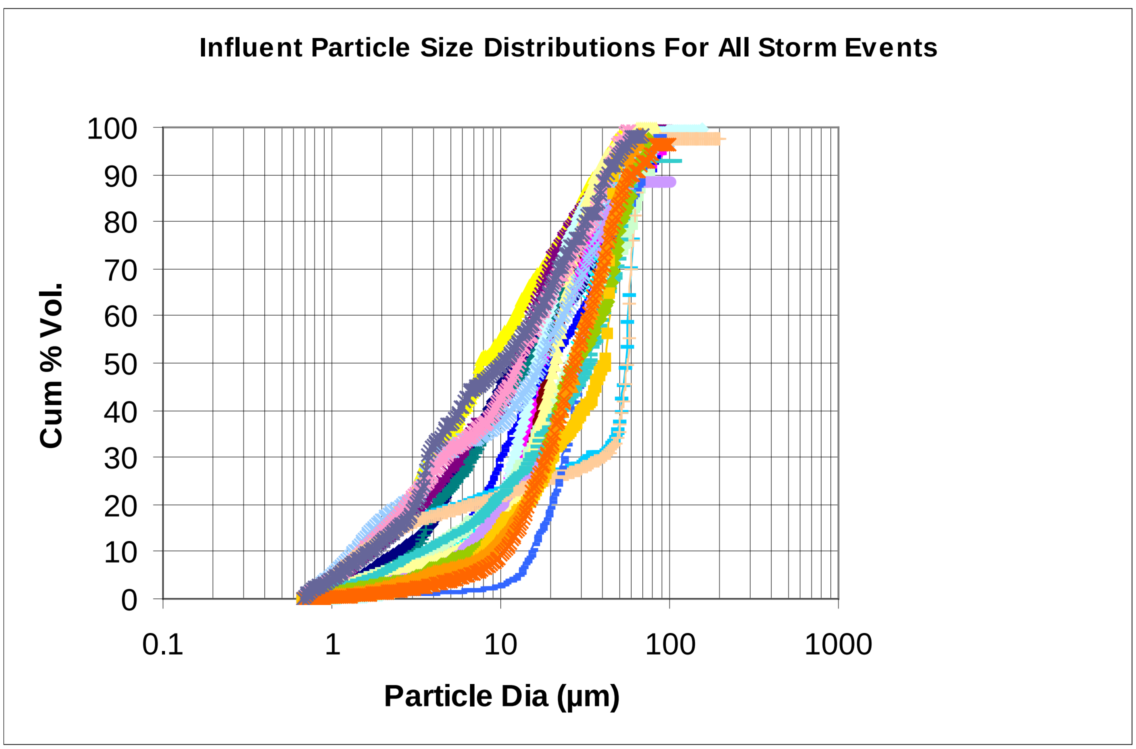
Probability that influent = effluent (nonparametric sign test): 0.0005 (99.95% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.

Table 6.26: Comparison of 60 to 120 µm particles for storm events

Observed 60 to 120 µm Particle Size Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)	% reduc
1-1	1.0523	0.028	97
2-1	12.7041	0	100
2-2	12.055	5.9422	51
3-1	0.6768	0	100
3-2	0.6195	0.3467	44
4-1	0.0543	0	100
4-2	0	0	n/a
5-1	8.288	0.7955	90
5-2	0.261	0.1734	34
6-1	0.2235	0.1698	24
6-2	0.5753	1.4848	-158
6-3	0.6435	2.0319	-216
6-4	0.3519	0.4038	-15
6-5	2.9988	0	100
7-1	1.1205	3.0312	-171
7-2	1.3631	2.613	-92
7-3	2.9609	1.65	44
7-4	0	0.4596	n/a
7-5	0.69	0.1936	72
7-6	0.4116	1.0548	-156
8-1	21.2729	14.115	34
8-2	1.3735	0.4716	66
9-1	4.2775	0.441	90
10-1	1.2024	0.0272	98
min	0.000	0.000	-216
max	21.273	14.115	100
average	3.132	1.476	15
median	0.871	0.422	47
st dev	5.269	3.022	103
COV	1.7	2.0	7

Probability that influent = effluent (nonparametric sign test): 0.026 (97.4% confident that influent ≠ effluent), therefore statistically significant at least at the α 0.05 level.



Influent Particle Size Distributions For All Storm Events
Cum % Vol.
100
90
80
70
60
50
40
30
20
10
0
0.1
1
10
100
1000
Particle Dia (µm)

Figure 6.27: Influent particle size distribution for all storm events

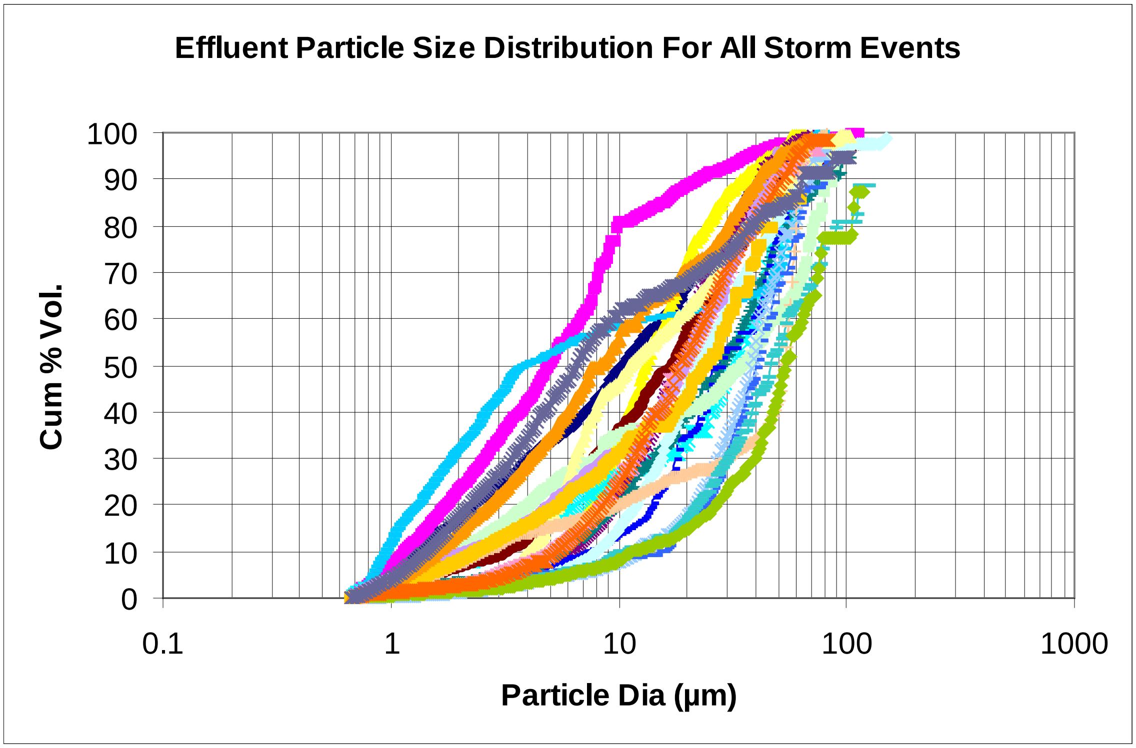


Figure 6.28: Effluent particle size distribution for all storm events

Appendix –A

Actual Storm Events Flow and Hydrographs

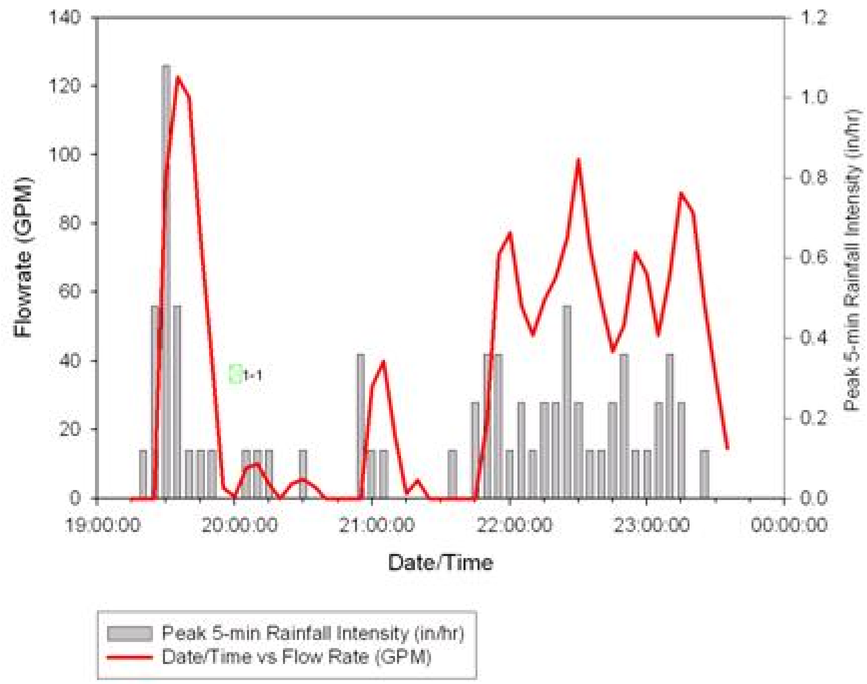


Figure 1: Flow and hydrograph for March 26, 2005 event

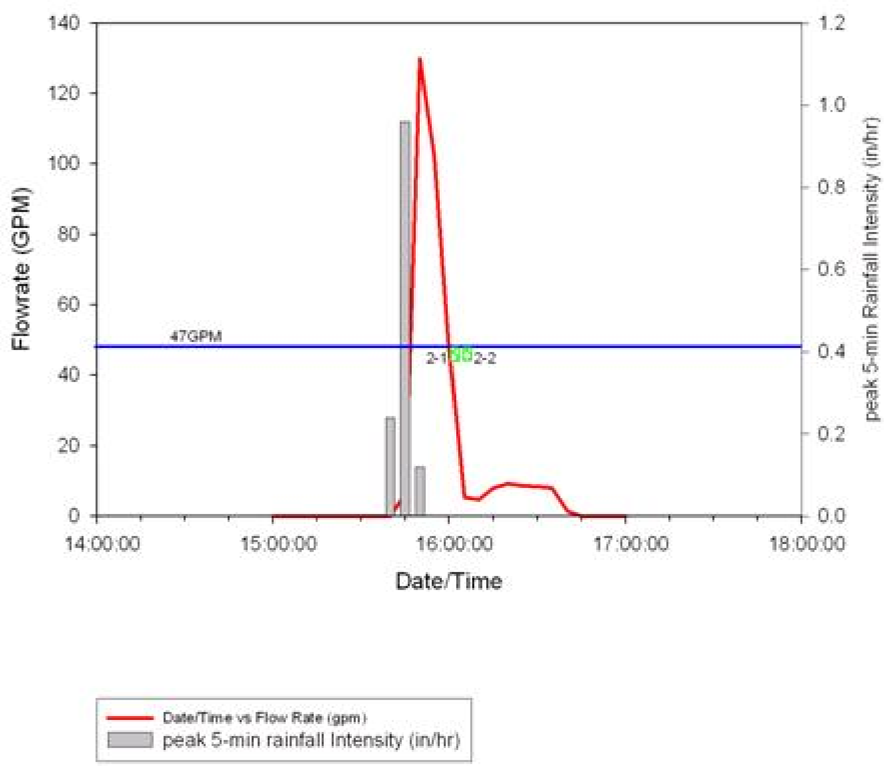


Figure 2: Flow and hydrograph for April 01, 2005 event

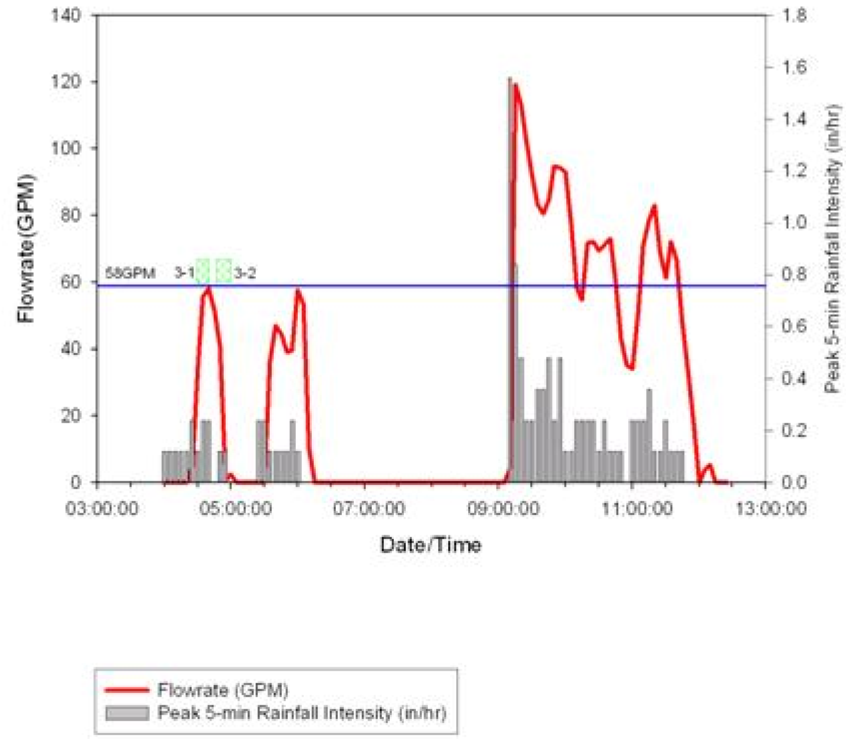


Figure 3: Flow and hydrograph for April 26, 2005 event

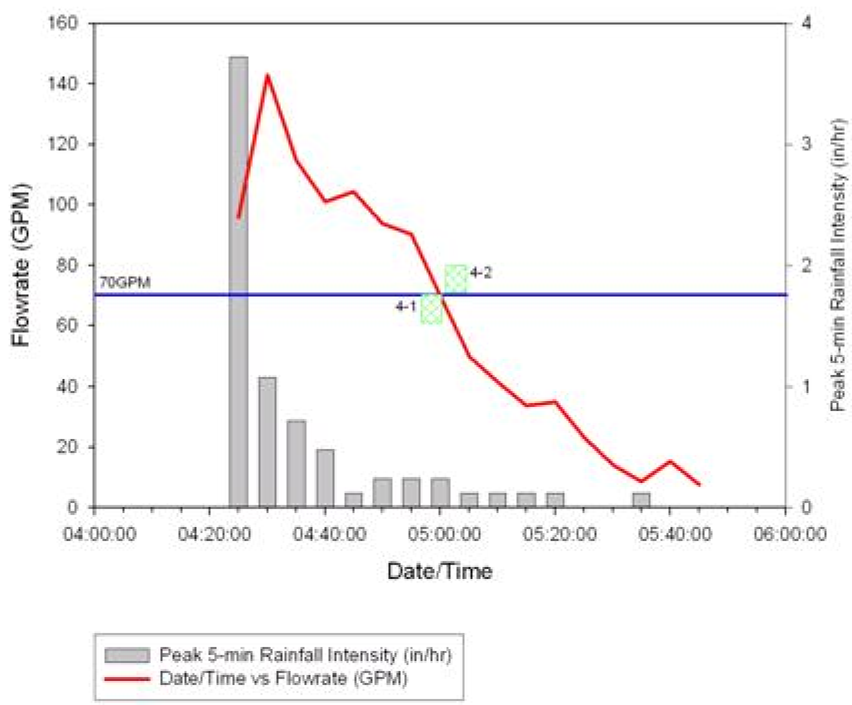


Figure 4: Flow and hydrograph for April 30, 2005 event

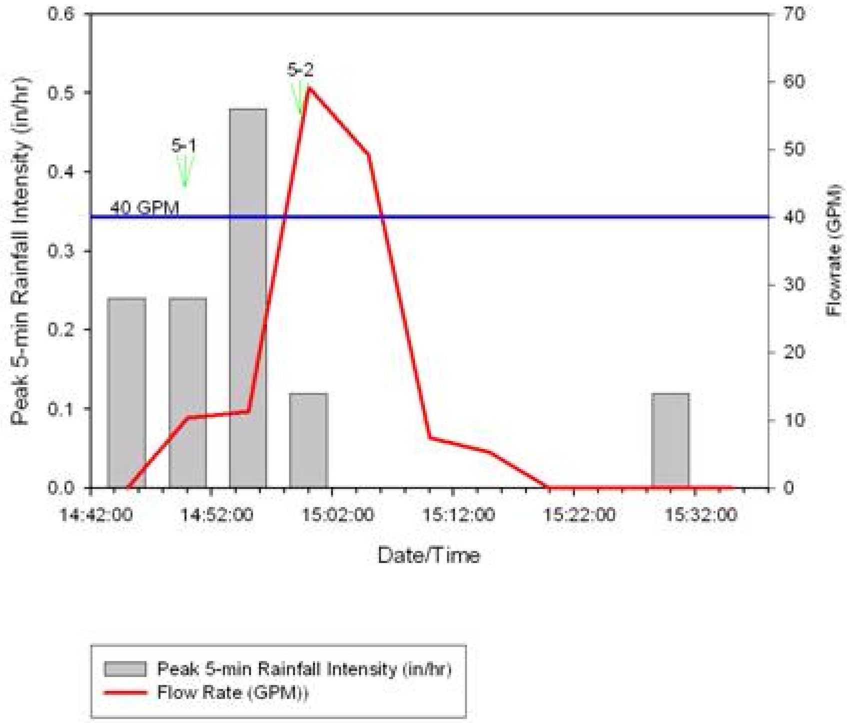


Figure 5: Flow and hydrograph for July 05, 2005 event

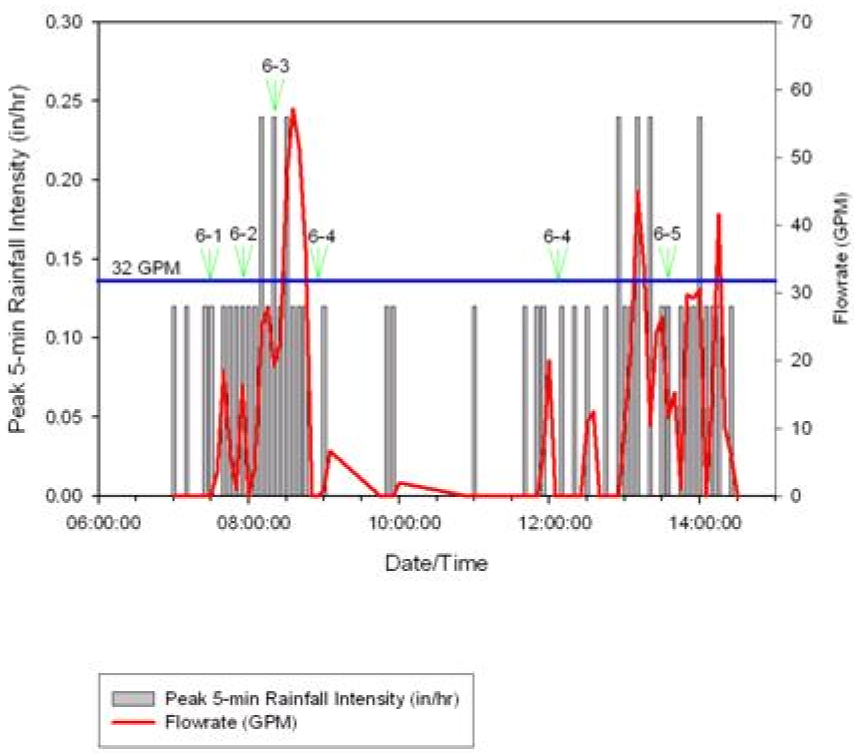


Figure 6: Flow and hydrograph for July 06, 2005 event

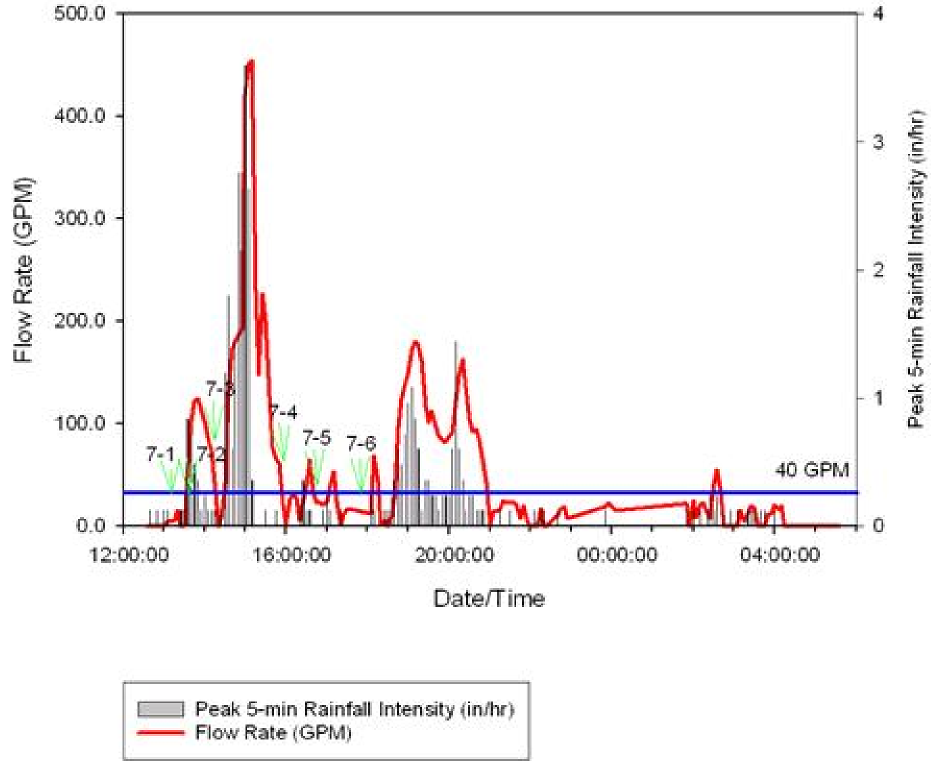


Figure 7: Flow and hydrograph for August 29, 2005 event

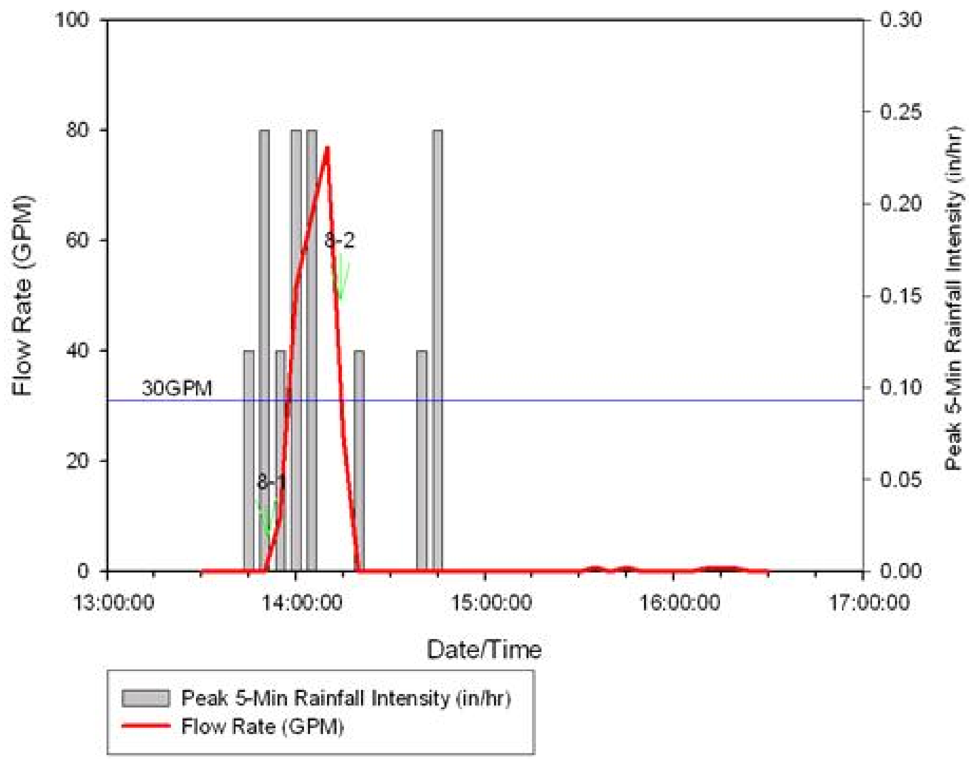


Figure 8: Flow and hydrograph for September 16, 2005 event

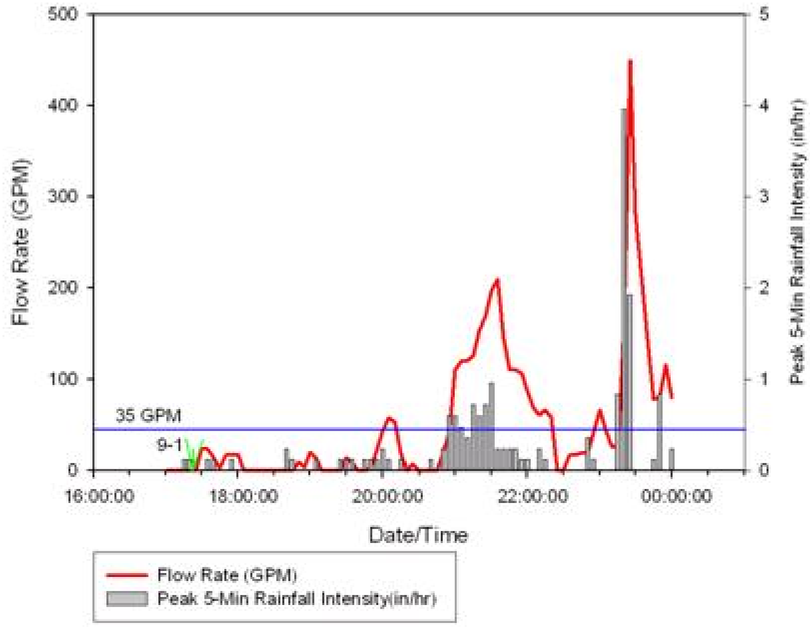


Figure 9: Flow and hydrograph for September 25, 2005 event

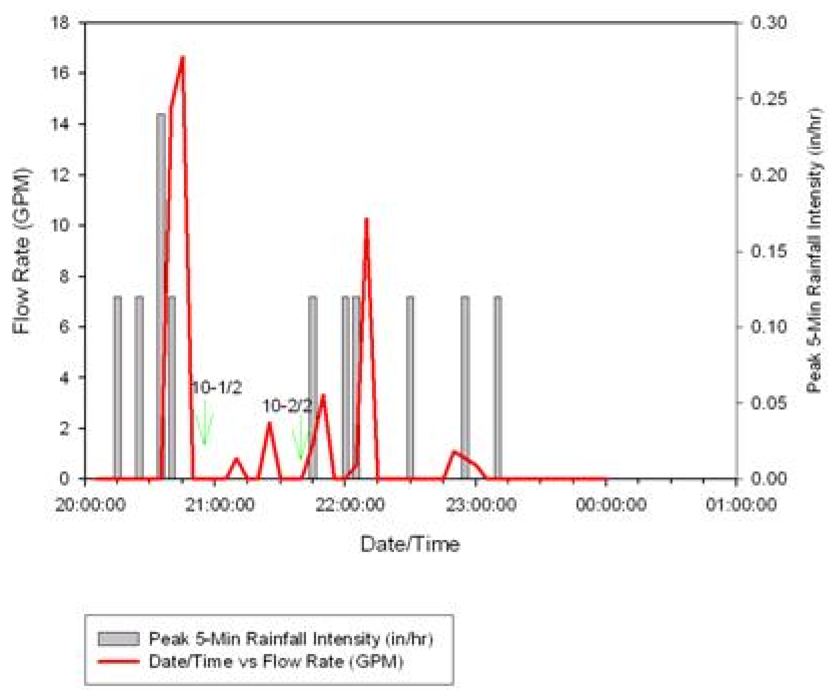


Figure 10: Flow and hydrograph for October 06, 2005 event

Date	Total Rain Depth Tipping Bucket (Inches)	Total Rain Depth Cumulative Rain Gauge (Inches)	% Difference
26-Mar-05	0.74	0.75	1.33
1-Apr-05	0.11	0.12	8.33
26-Apr-05	1.04	1.1	5.45
30-Apr-05	0.64	0.7	8.57
5-Jul-05	0.20	0.25	20.00
6-Jul-05	0.51	0.6	15.00
29-Aug-05	3.20	3.45	7.25
16-Sep-05	0.12	0.12	0.00
25-Sep-05	1.47	1.5	2.00
6-Oct-05	0.11	0.11	0.00

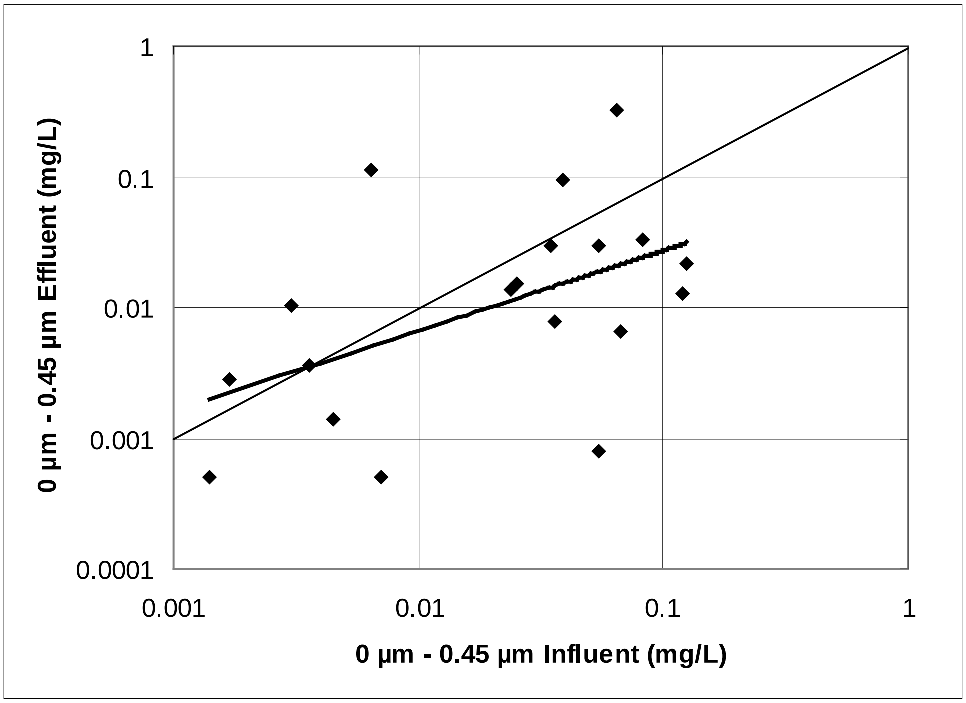
Appendix B

Actual Storm Event Particle Size Range Performance, Scatter, Box and Probability Plots

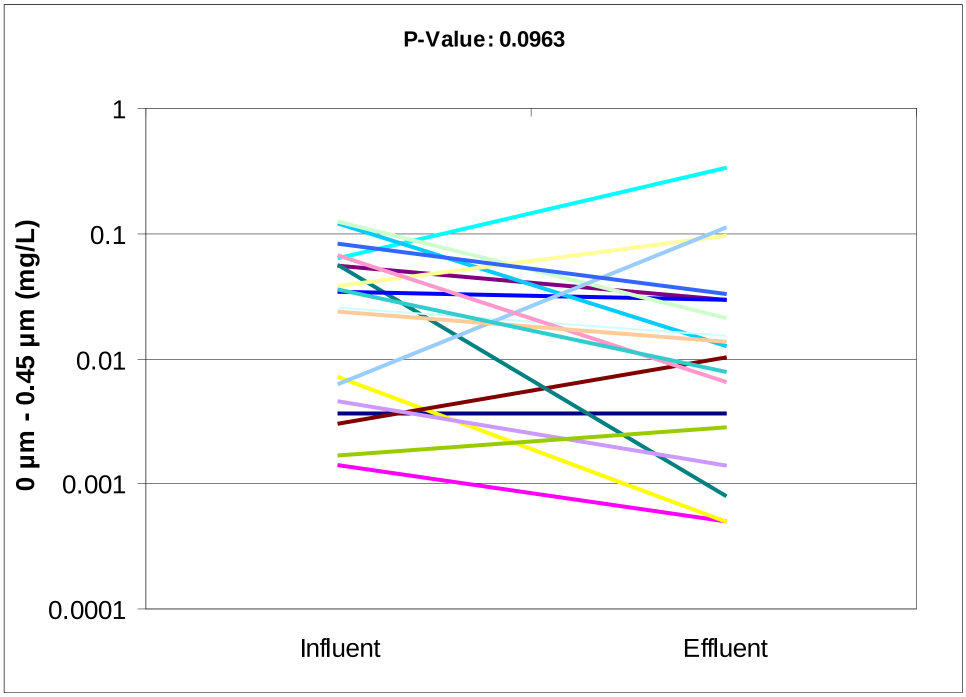
0 to 0.45 μm Particle Size

Observed 0 to 0.45 μm Particle Size Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)
1-1	0	0
2-1	0	15.67
2-2	0	0
3-1	0.0072	0
3-2	0.0036	0.0036
4-1	0.0014	0
4-2	0.0071	0
5-1	0.064	0.33
5-2	0.055	0.03
6-1	0.003	0.0102
6-2	0.055	0.0008
6-3	0.0345	0.0299
6-4	0.1207	0.0129
6-5	0.0252	0.015
7-1	0.1245	0.0216
7-2	0.0387	0.096
7-3	0.00638	0.1122
7-4	0.0667	0.0066
7-5	0.0045	0.0014
7-6	0.024	0.0136
8-1	0.0821	0.033
8-2	0.036	0.0079
9-1	0.0017	0.0028
10-1	1.325	0.2057



Scatterplot of observed influent and effluent 0 to 0.45 µm particle size concentrations.



Paired influent and effluent 0 to 0.45 µm particle size concentrations.

Fitted Equation:

Effluent Turbidity, (0 to 0.45 µm particle size log mg/L) = 1.10 * (0 to 0.45 µm particle size log mg/L)

Regression Statistics

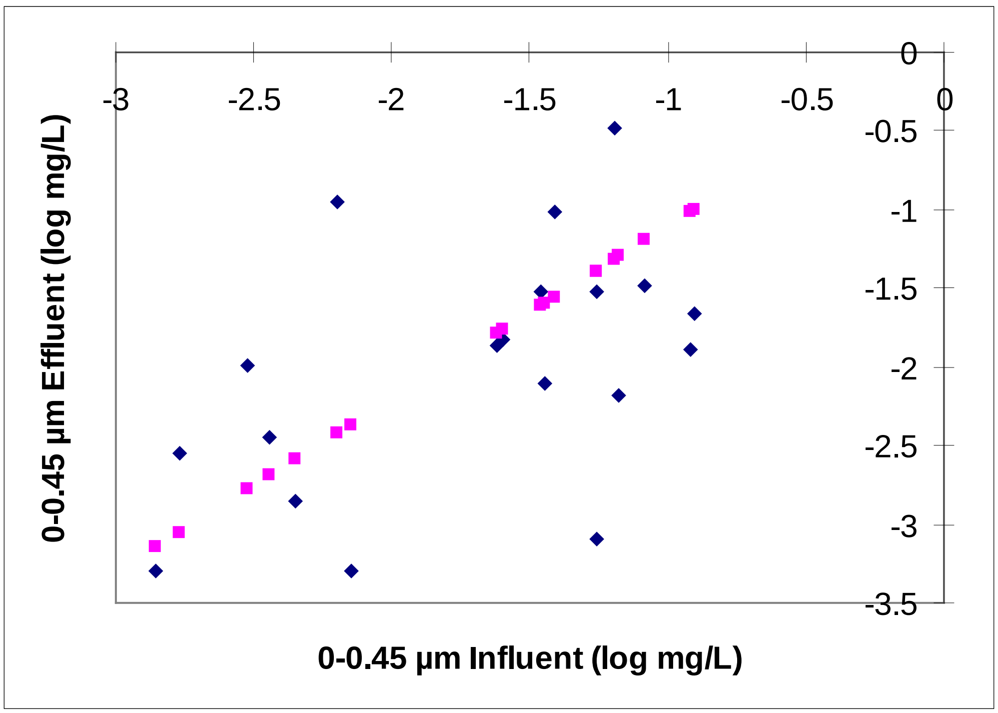
Multiple R	0.939
R Square	0.882
Adjusted R Square	0.826
Standard Error	0.758
Observations	19

ANOVA

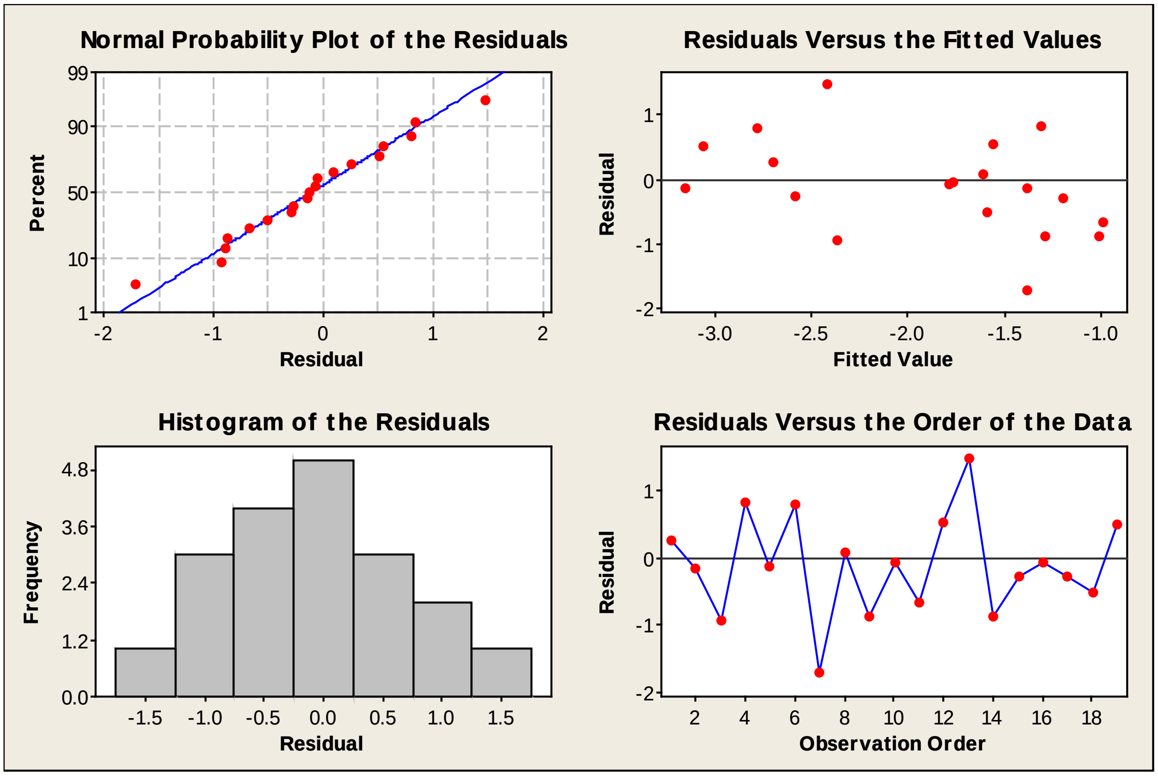
	df	SS	MS	F	Significance F
Regression	1	77.0	77.0	134	1.74E-09
Residual	18	10.38	0.575		
Total	19	87.3			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
0-0.45 µm Log Influent*	1.10	0.0953	11.6	9.00E-10	0.902	1.30

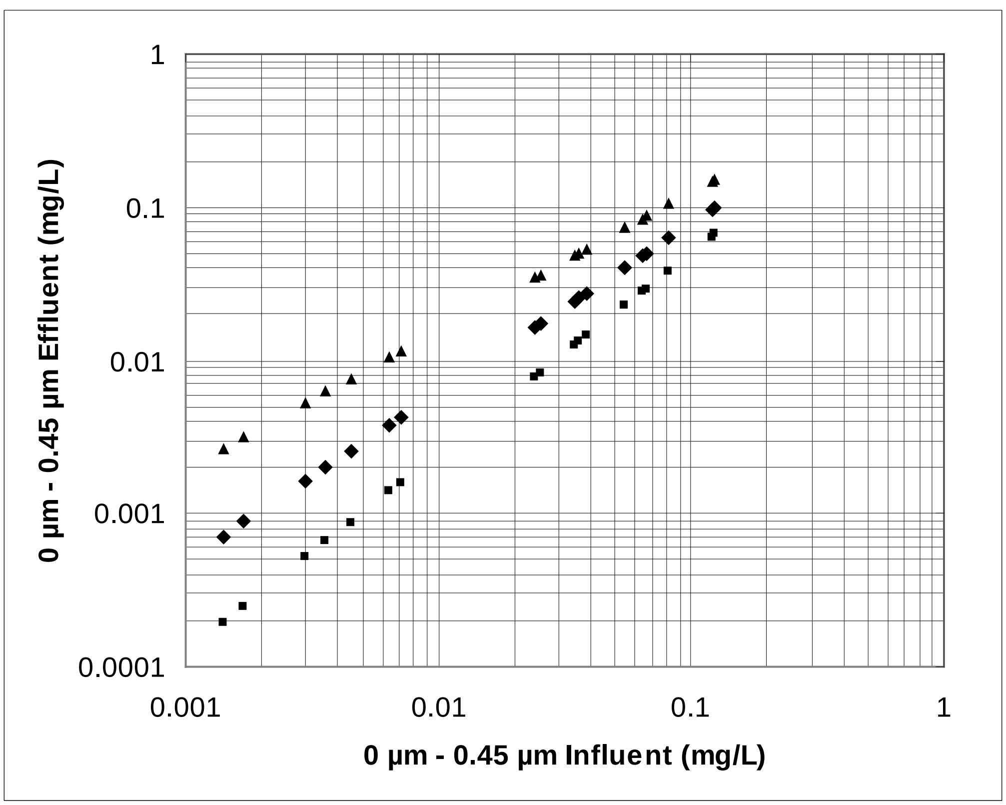
* the intercept term was determined to be not significant



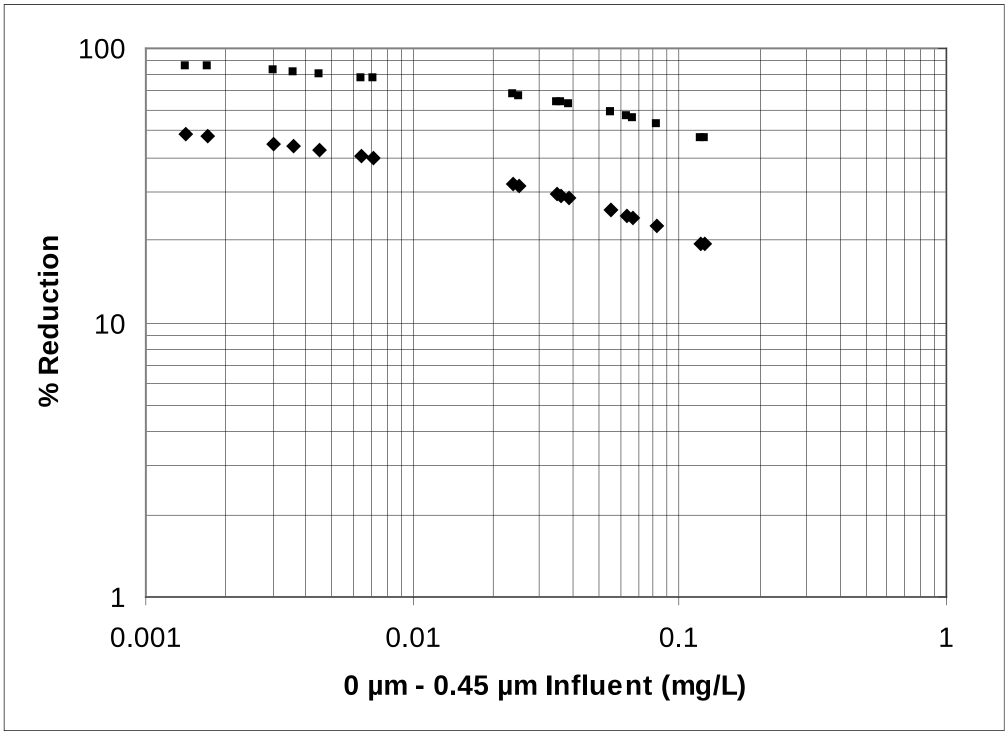
Fitted equation and data points for influent and effluent 0 to 0.45 µm particle size.



Residual analyses of fitted equation for 0 to 0.45 µm particle size influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

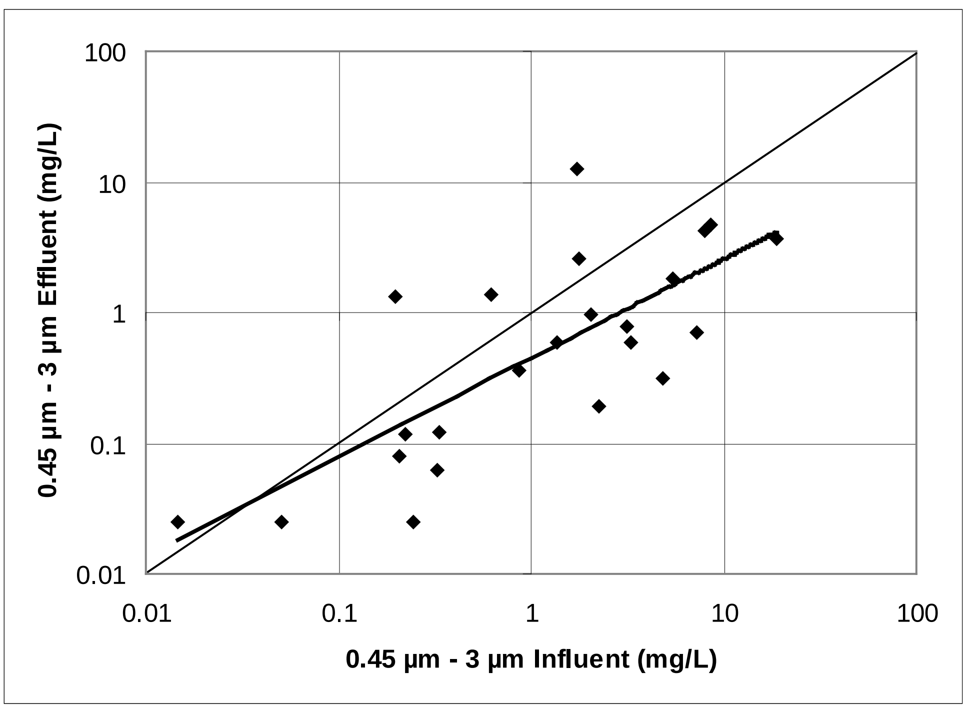


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

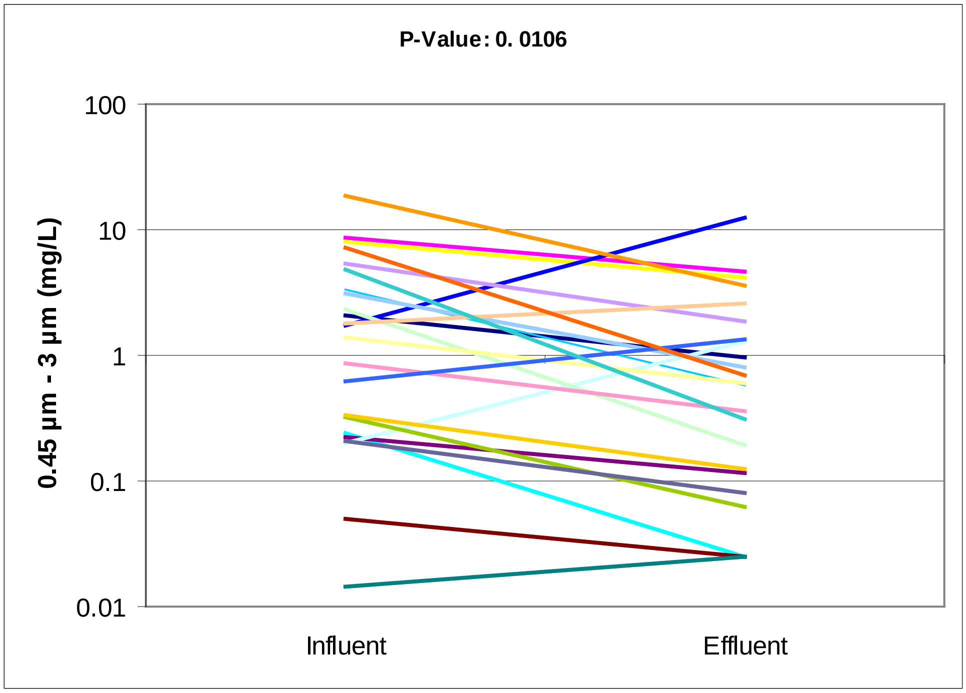
0.45 to 3 µm Particle Size

Observed 0.45 to 3 µm Particle Size Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)
1-1	2.0451	0.9596
2-1	8.6655	4.7268
2-2	7.95	4.2217
3-1	0.2448	0
3-2	0.22401	0.1153
4-1	0.0501	0
4-2	0.0145	0
5-1	1.712	12.3728
5-2	3.2805	0.5814
6-1	0.197	1.3092
6-2	2.2759	0.1904
6-3	1.365	0.5993
6-4	3.1331	0.792
6-5	0.8652	0.3618
7-1	5.4448	1.836
7-2	1.788	2.62
7-3	0.62	1.36
7-4	4.8783	0.3114
7-5	0.3255	0.0632
7-6	0.3368	0.1232
8-1	18.8169	3.612
8-2	7.2988	0.6994
9-1	0.209	0.08
10-1	34.42	3.8114



Scatterplot of observed influent and effluent 0.45 to 3 µm particle size concentrations.



Paired influent and effluent 0.45 to 3 µm particle size concentrations.

Fitted Equation:

Effluent Turbidity, (0.45 to 3 µm particle size log mg/L) = -0.347 + 0.757 * (0.45 to 3 µm particle size log mg/L)

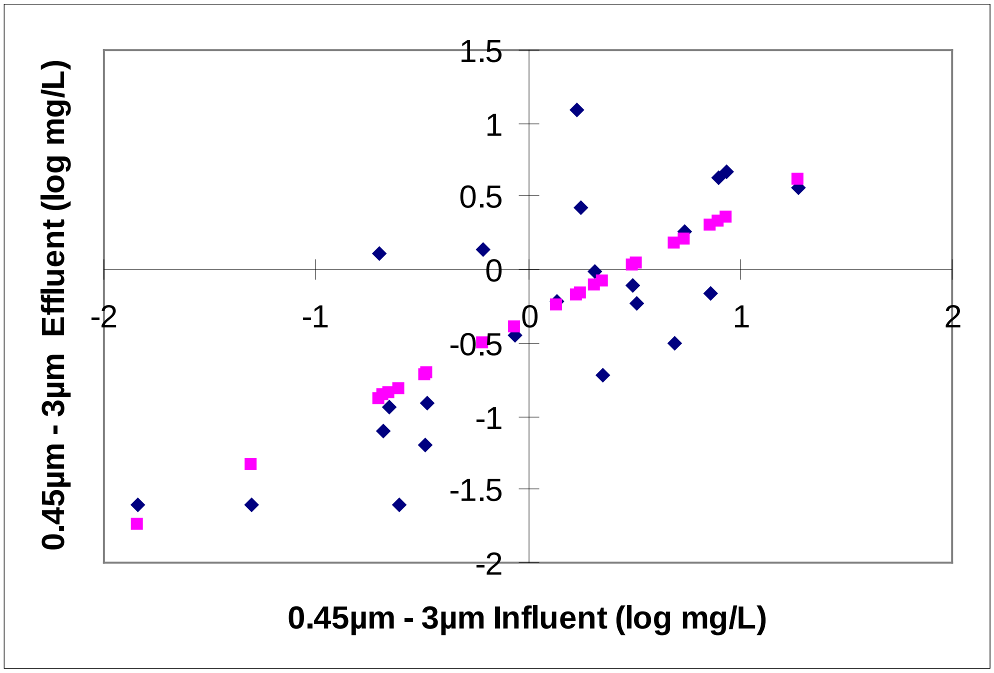
Regression Statistics

Multiple R	0.754
R Square	0.568
Adjusted R Square	0.547
Standard Error	0.524
Observations	23

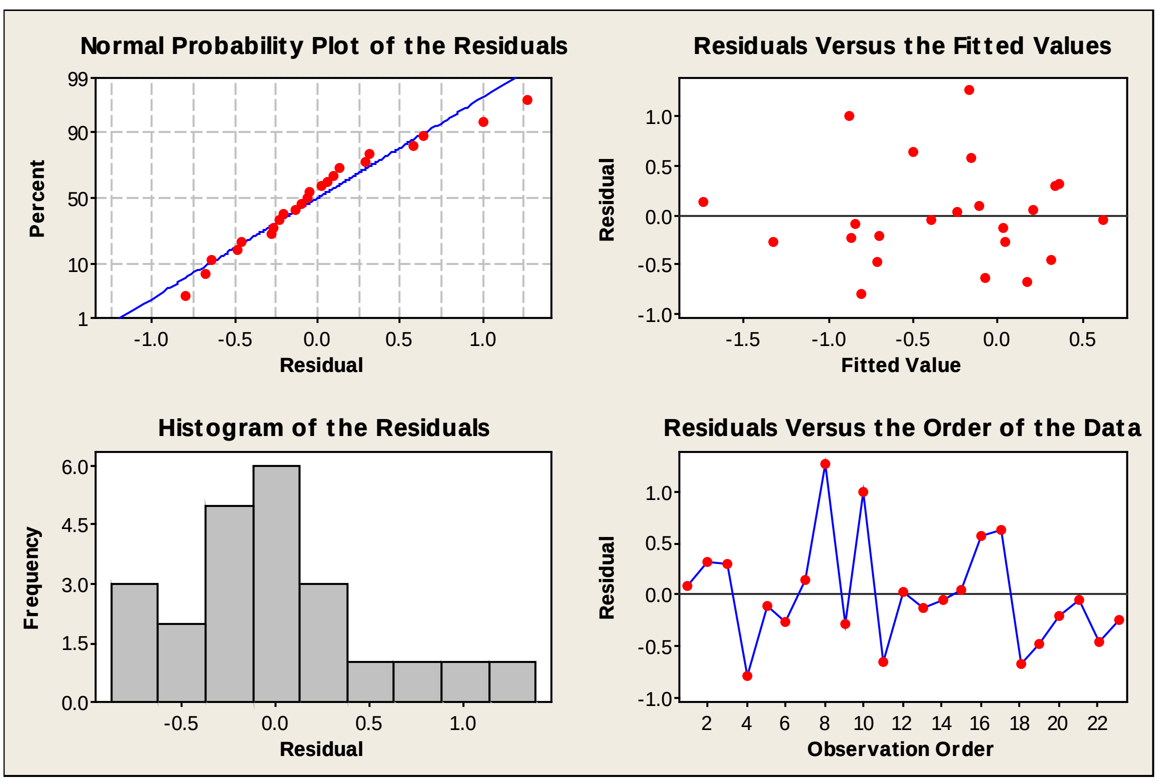
ANOVA

	df	SS	MS	F	Significance F
Regression	1	7.59	7.59	27.6	3.30E-05
Residual	21	5.77	0.274		
Total	22	13.4			

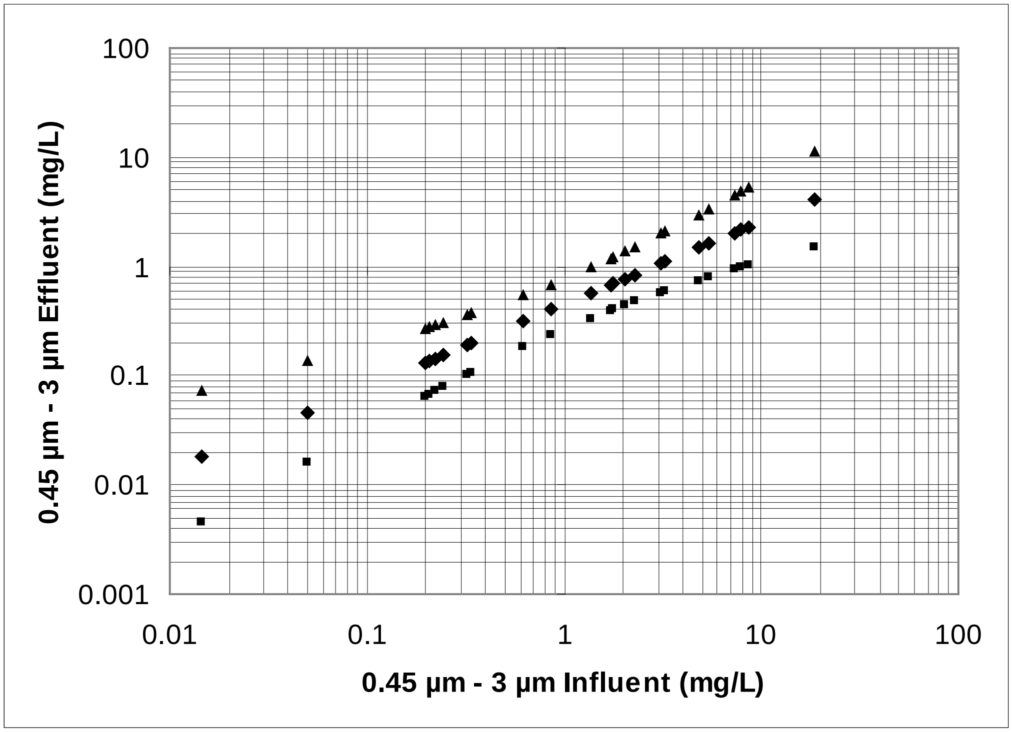
	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Intercept	-0.347	0.109	-3.17	0.00458	-0.575	-0.120
0.45µm - 3µm Log Influent	0.757	0.144	5.25	3.30E-05	0.457	1.06



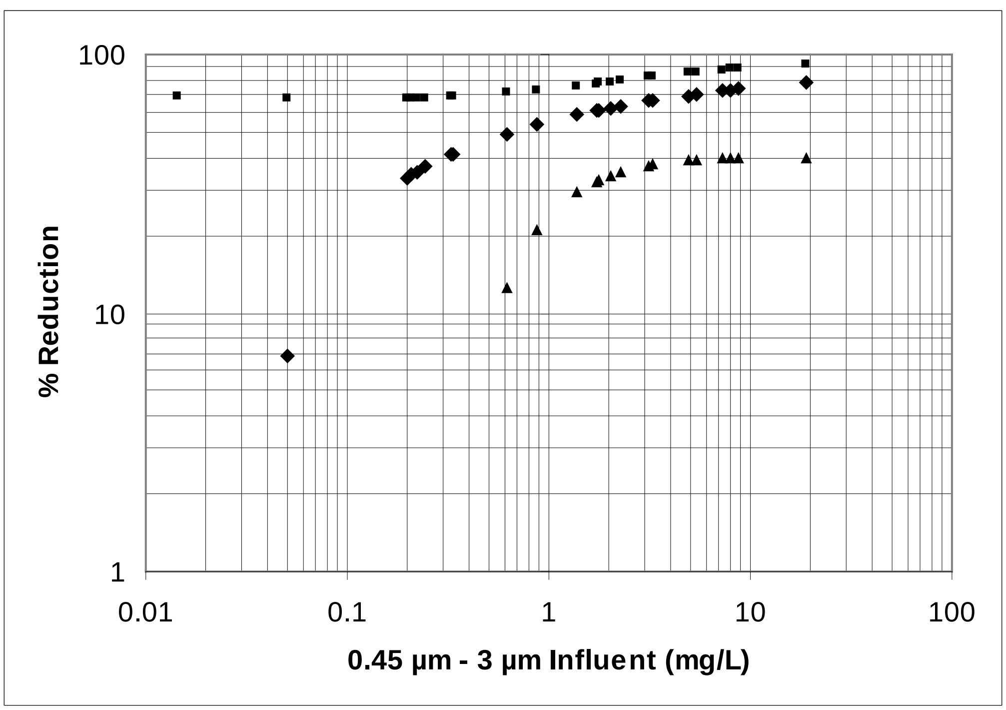
Fitted equation and data points for influent and effluent 0.45 to 3 µm particle size.



Residual analyses of fitted equation for 0.45 to 3 µm particle size influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

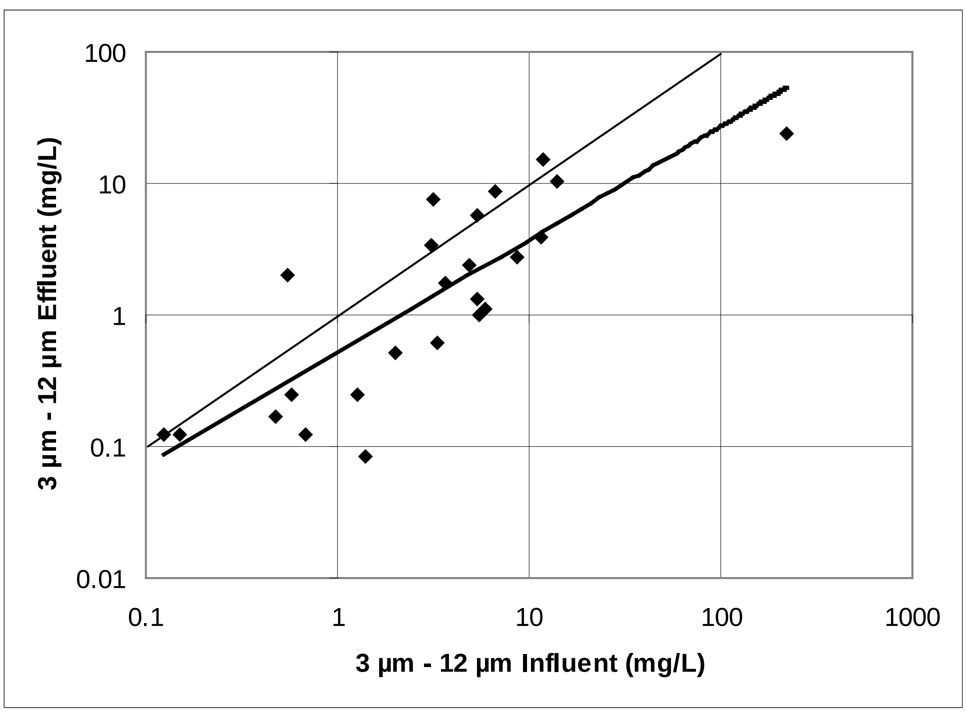


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

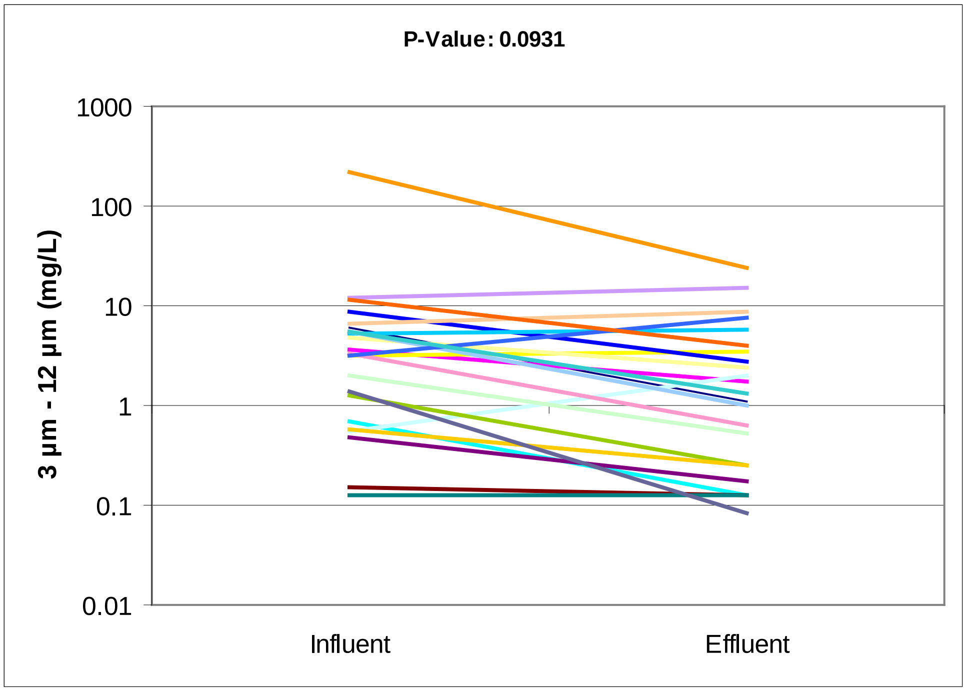
3 to 12 µm Particle Size

Observed 3 to 12 µm Particle Size Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)
1-1	5.96	1.12
2-1	3.62	1.74
2-2	3.105	3.45
3-1	0.68	0
3-2	0.48	0.17
4-1	0.15	0
4-2	0.124	0
5-1	8.58	2.78
5-2	5.32	5.74
6-1	0.551	2.03
6-2	2.02	0.519
6-3	4.866	2.36
6-4	5.505	1
6-5	3.28	0.617
7-1	11.93	15.43
7-2	6.65	8.67
7-3	3.15	7.61
7-4	5.41	1.33
7-5	1.27	0.25
7-6	0.58	0.25
8-1	221.44	23.71
8-2	11.67	3.93
9-1	1.39	0.085
10-1	13.92	10.25



Scatterplot of observed influent and effluent 3 to 12 µm particle size concentrations.



Paired influent and effluent 3 µm to 12 µm particle size concentrations.

Fitted Equation:

Effluent Turbidity, (3 to 12 µm particle size log mg/L) = -0.322 + 0.923 * (3 to 12 µm particle size log mg/L)

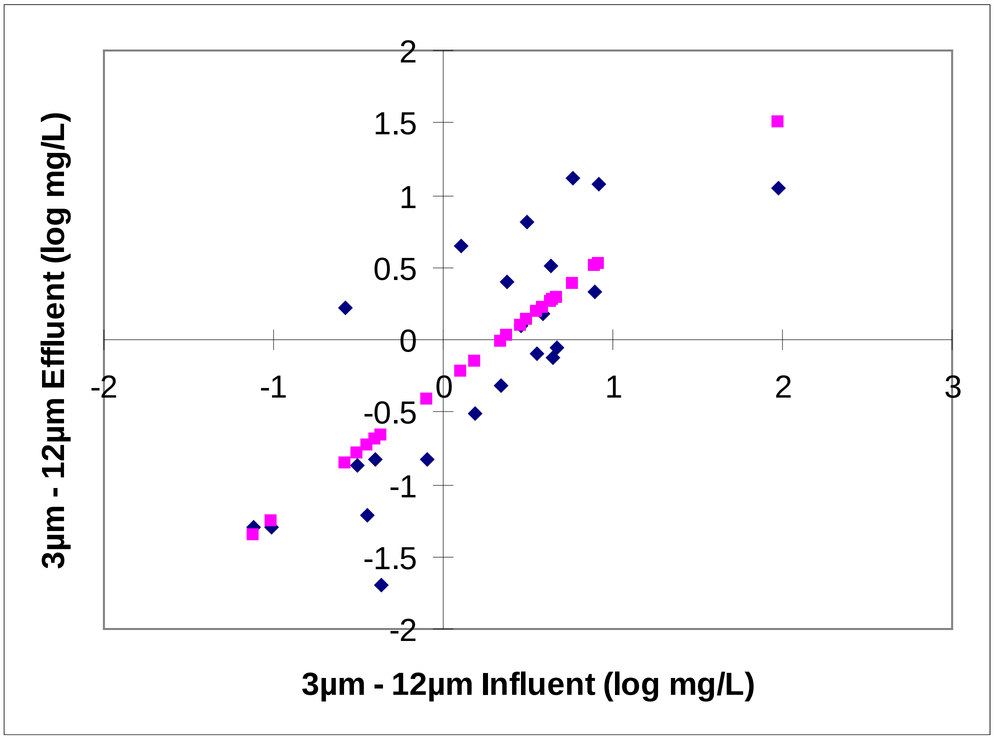
Regression Statistics

Multiple R	0.787
R Square	0.619
Adjusted R Square	0.601
Standard Error	0.525
Observations	23

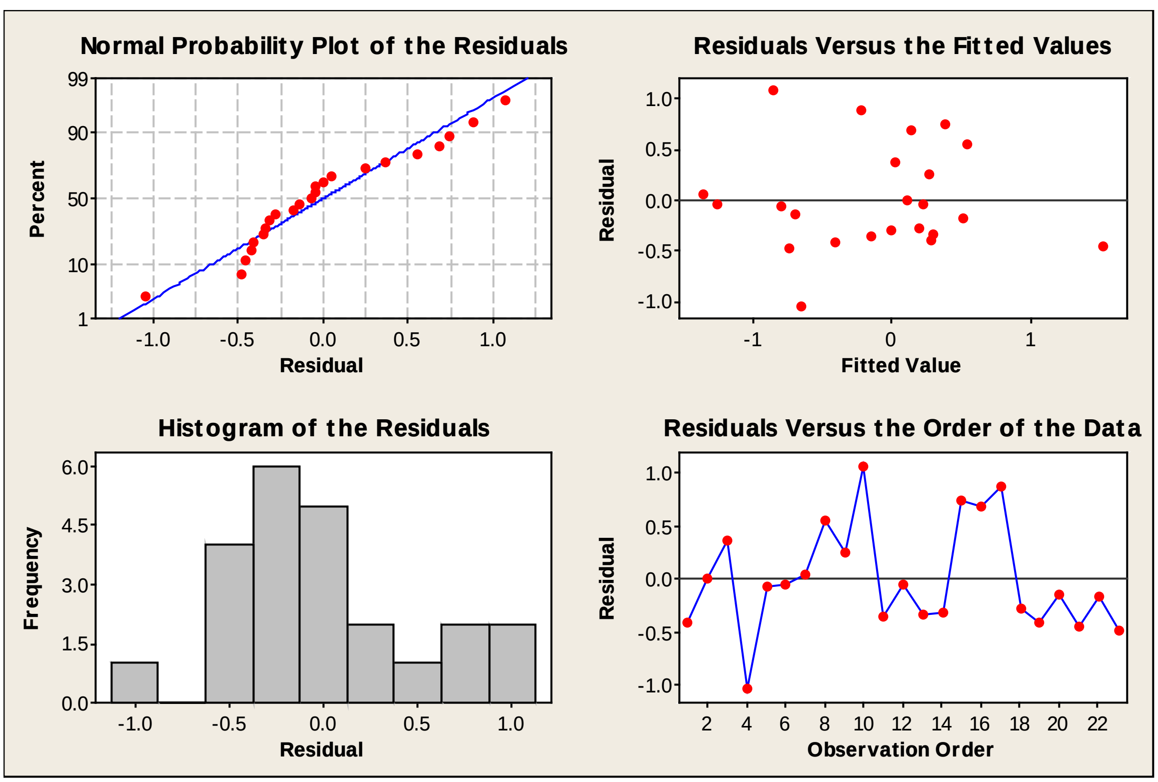
ANOVA

	df	SS	MS	F	Significance F
Regression	1	9.41	9.41	34.2	8.391E-06
Residual	21	5.78	0.275		
Total	22	15.2			

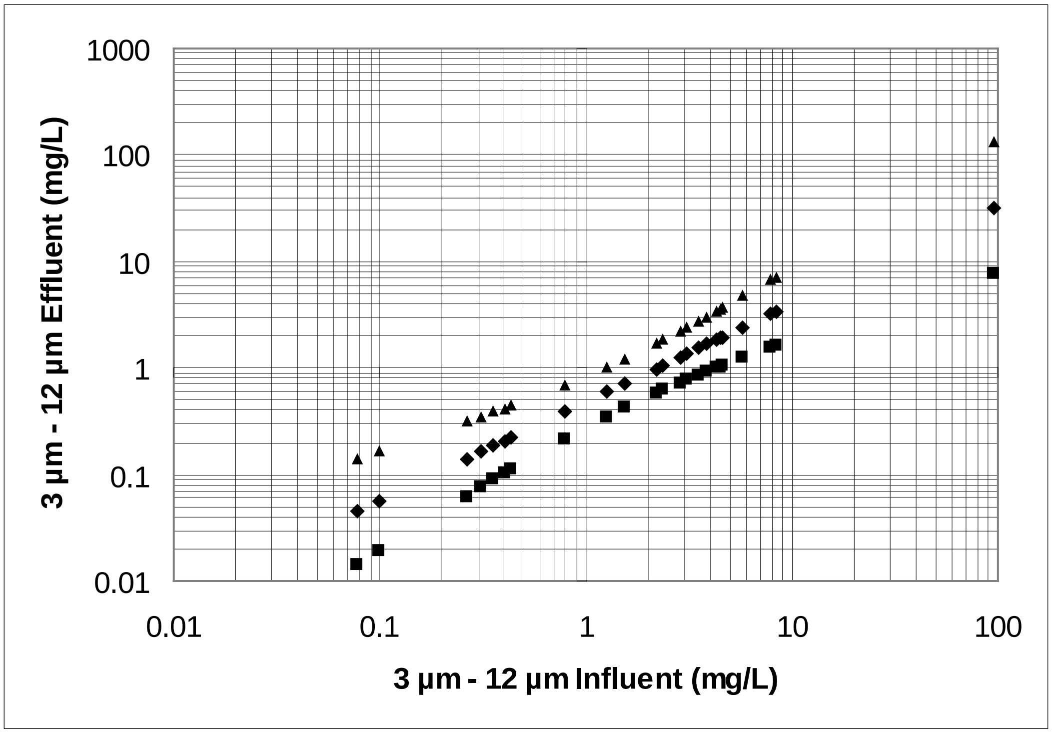
	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Intercept	-0.322	0.115	-2.80	0.0108	-0.560	-0.0823
3µm - 12µm Log Influent	0.923	0.158	5.85	8.39E-06	0.595	1.25



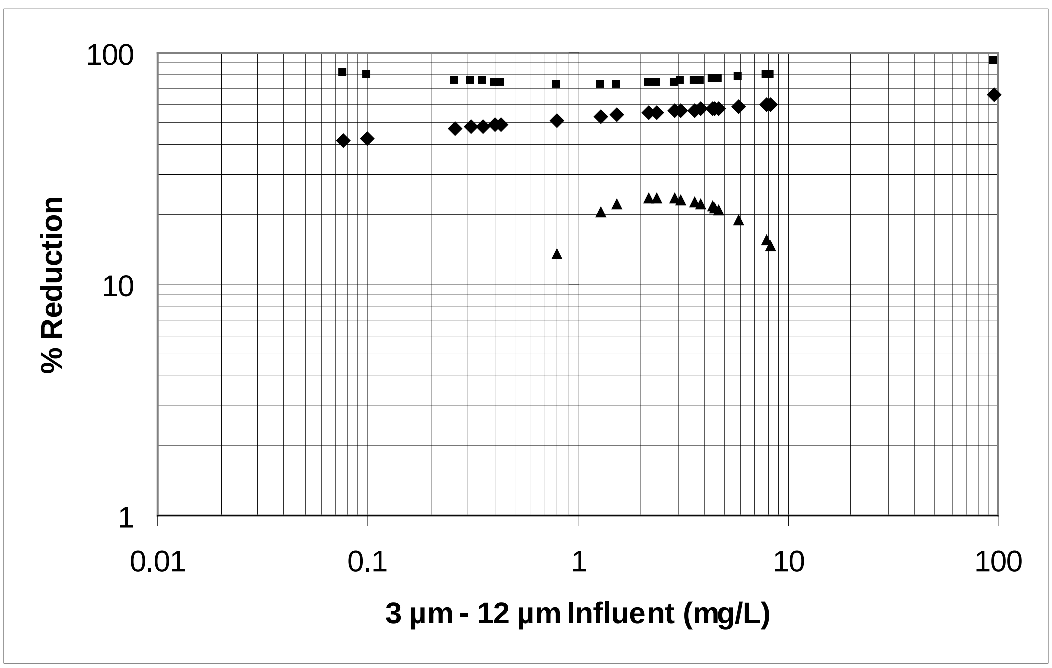
Fitted equation and data points for influent and effluent 3 to 12 µm particle size.



Residual analyses of fitted equation for 3 to 12 µm particle size influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

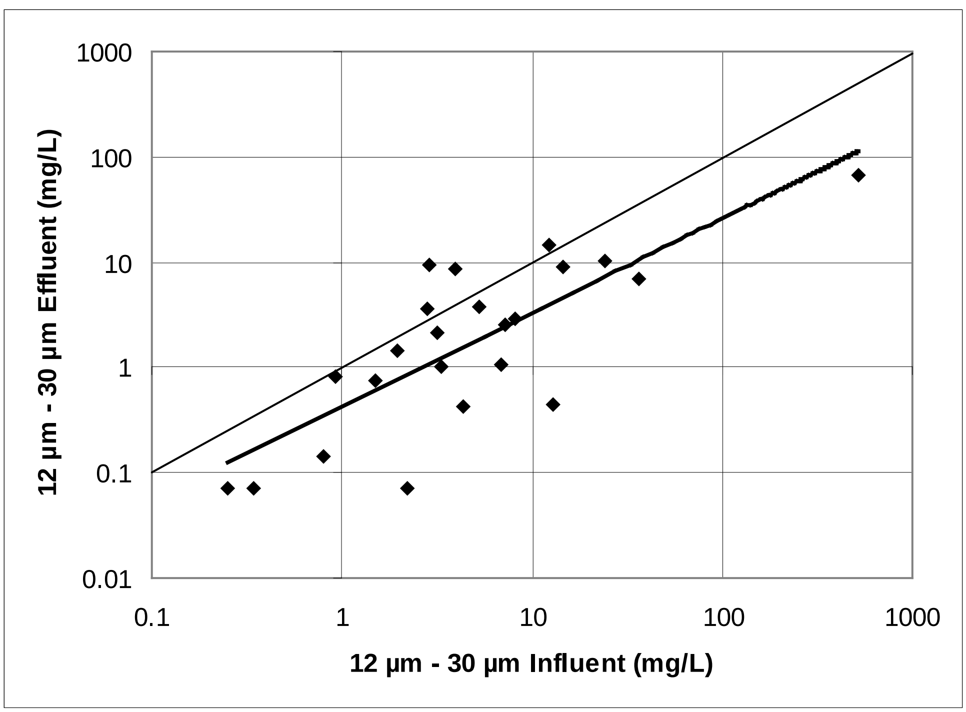


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

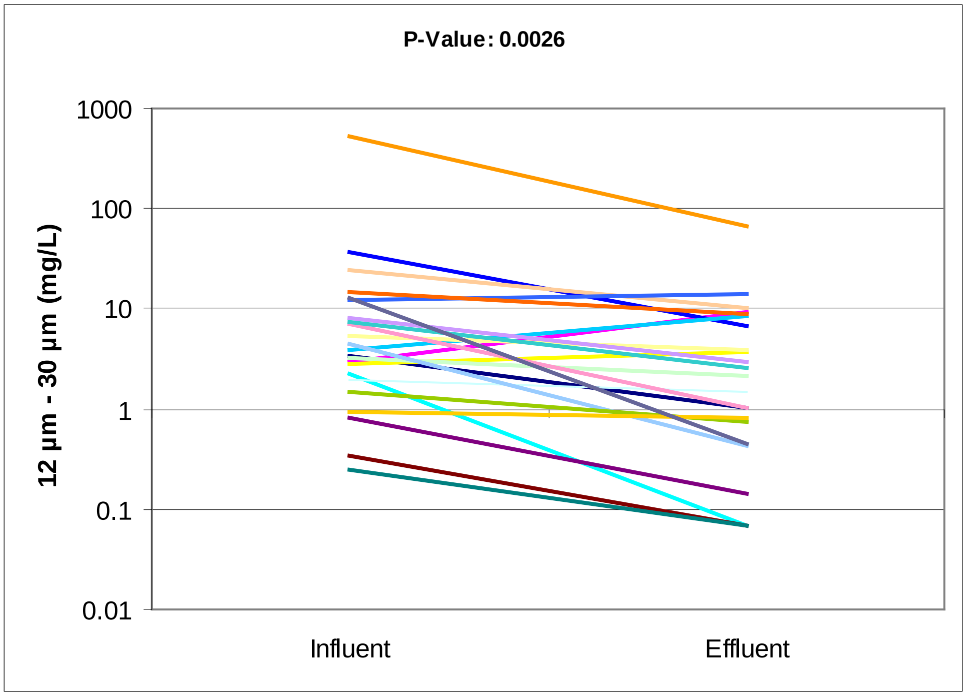
12 to 30 µm Particle Size

Observed 12 to 30 µm Particle Size Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)
1-1	3.36	1.04
2-1	2.91	9.23
2-2	2.8	3.63
3-1	2.23	0
3-2	0.8	0.14
4-1	0.344	0
4-2	0.252	0
5-1	36.86	6.83
5-2	3.97	8.45
6-1	1.961	1.45
6-2	3.193	2.135
6-3	5.3025	3.804
6-4	4.39	0.423
6-5	6.95	1.045
7-1	8.17	2.92
7-2	24.37	10.11
7-3	12.26	14.33
7-4	7.25	2.54
7-5	1.49	0.753
7-6	0.92	0.82
8-1	515.53	66.82
8-2	14.61	9.04
9-1	13.02	0.453
10-1	15.732	1.266



Scatterplot of observed influent and effluent 12 to 30 µm particle size concentrations.



Paired influent and effluent 12 µm to 30 µm particle size concentrations.

Fitted Equation:

Effluent Turbidity, (12 to 30 µm particle size log mg/L) = -0.319 + 0.796 * (12 to 30 µm particle size log mg/L)

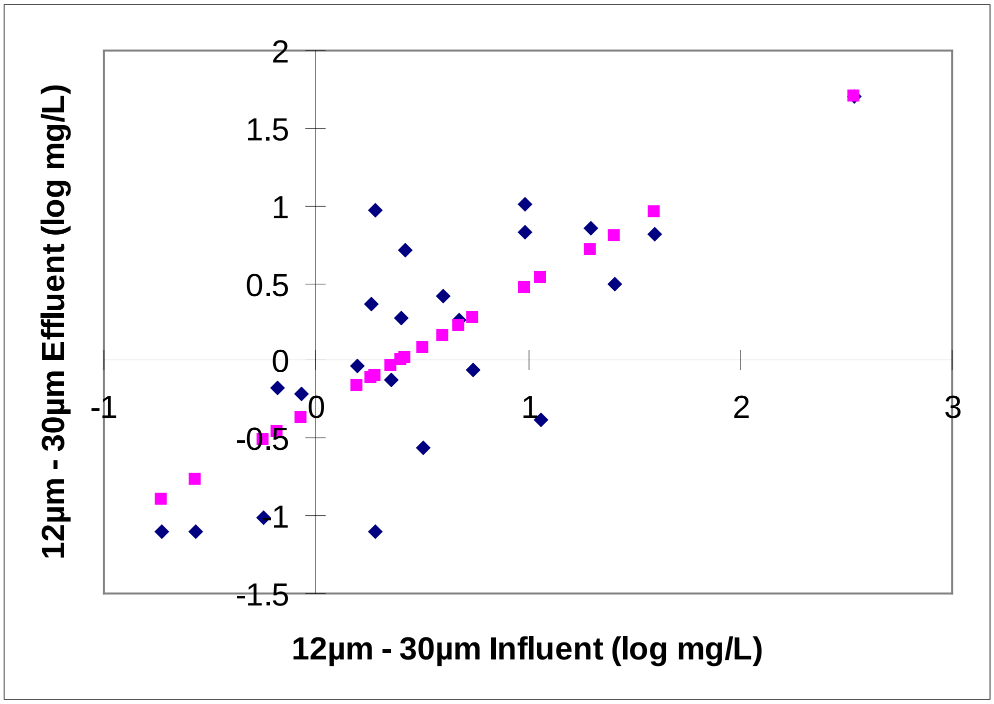
Regression Statistics

Multiple R	0.763
R Square	0.582
Adjusted R Square	0.562
Standard Error	0.508
Observations	23

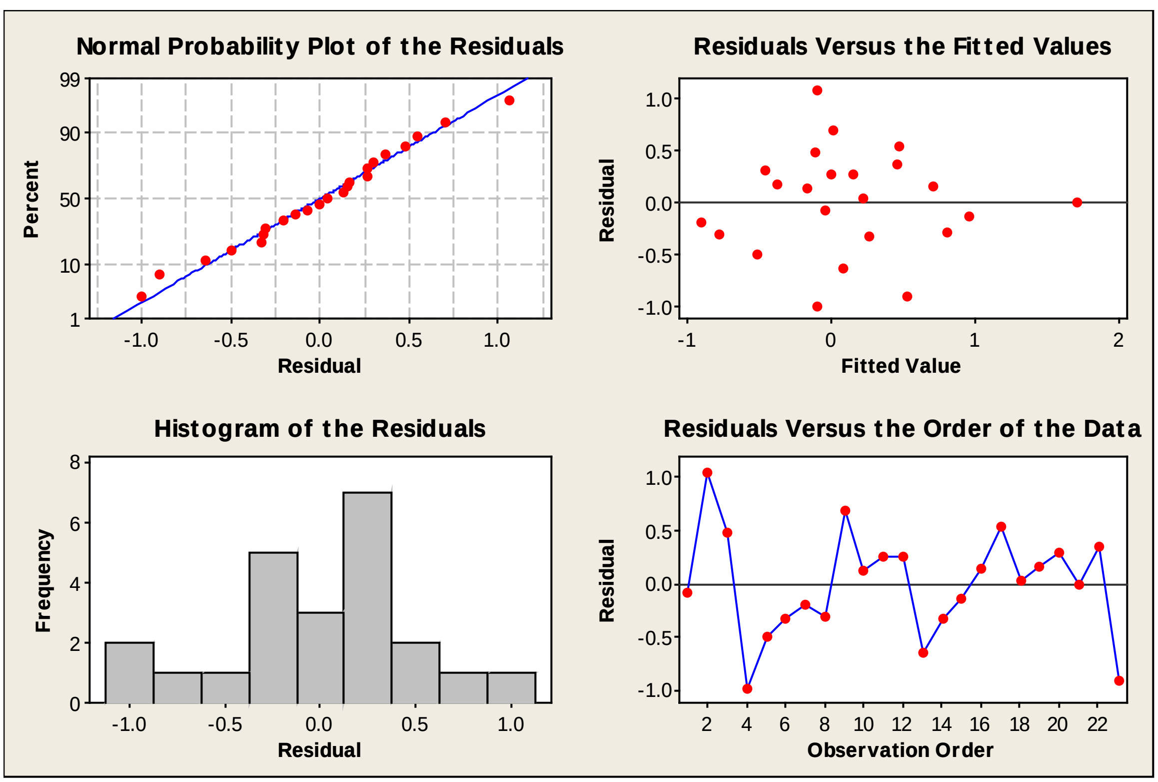
ANOVA

	df	SS	MS	F	Significance F
Regression	1	7.55	7.55	29.2	2.33E-05
Residual	21	5.43	0.258		
Total	22	13.0			

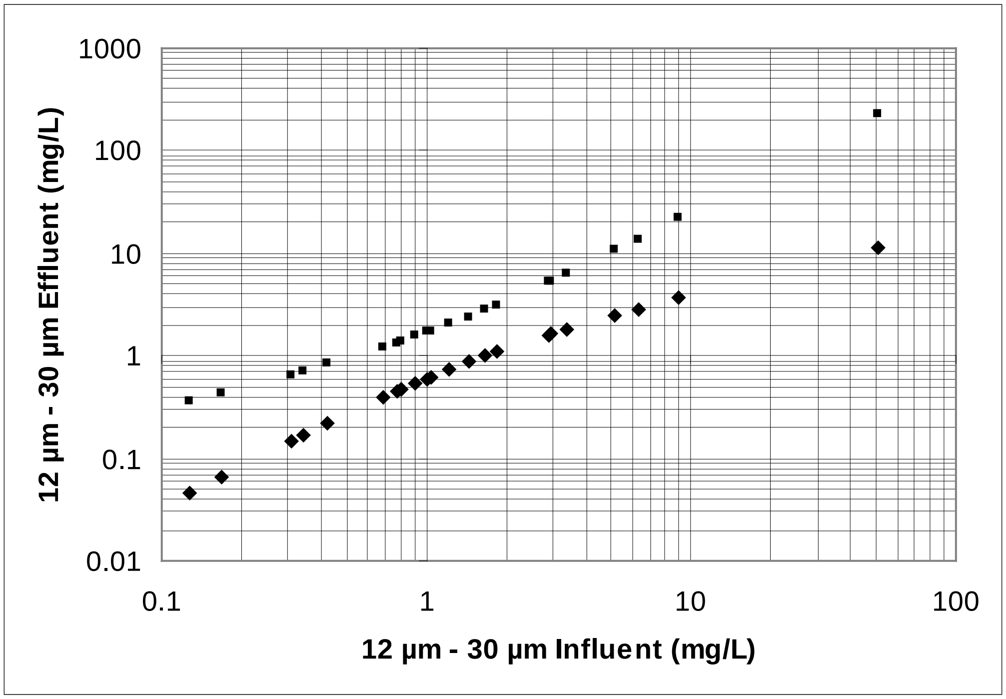
	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Intercept	-0.319	0.134	-2.38	0.0270	-0.598	-0.0399
12µm - 30µm Log Influent	0.796	0.147	5.406	2.33E-05	0.490	1.10



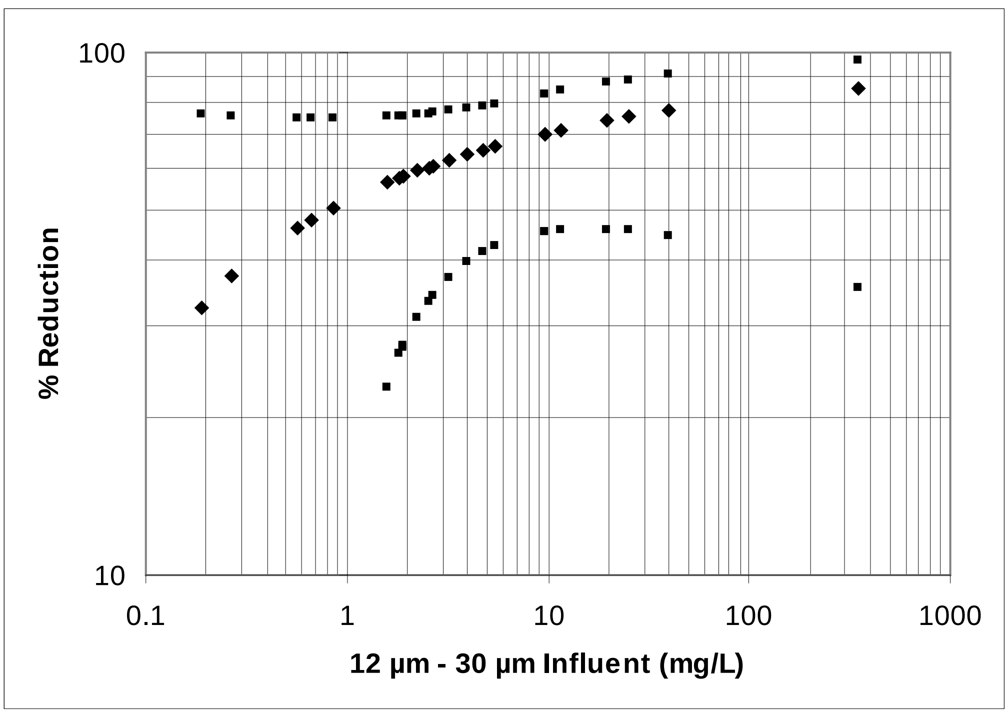
Fitted equation and data points for influent and effluent 12 to 30 µm particle size.



Residual analyses of fitted equation for 12 to 30 µm particle size influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

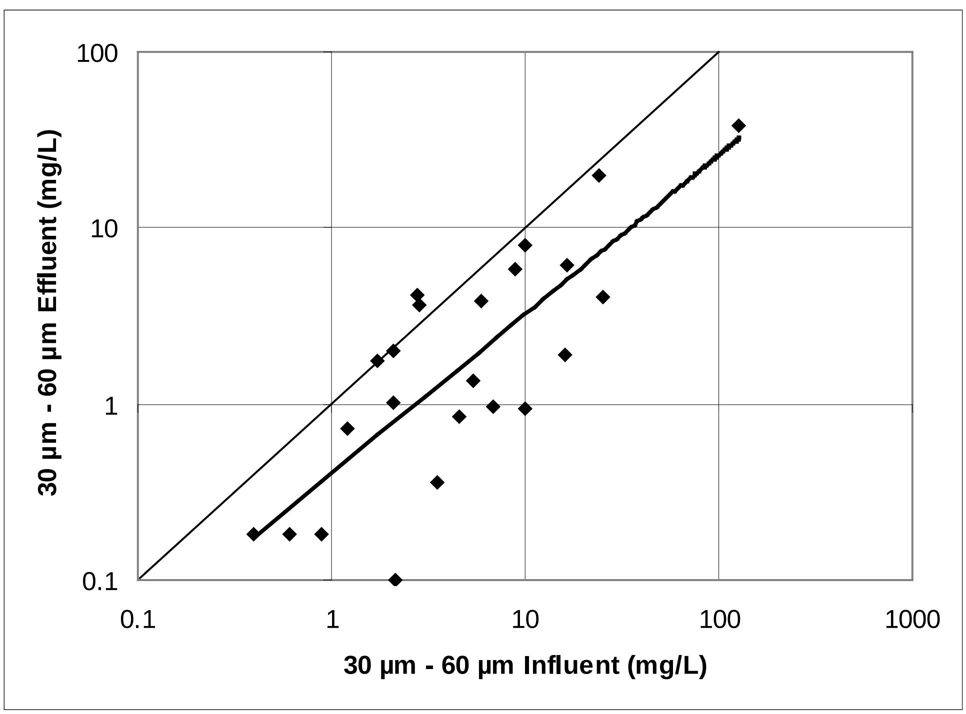


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

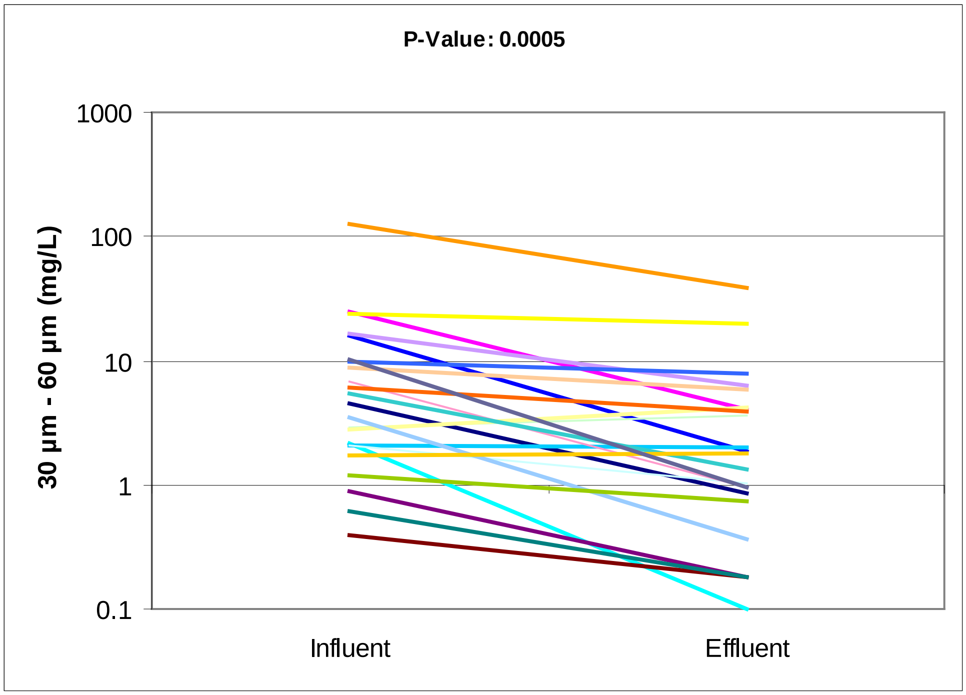
30 to 60 µm Particle Size

Observed 30 to 60 µm Particle Size Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)
1-1	4.5883	0.848
2-1	25.0902	4.086
2-2	24.1	19.758
3-1	2.1594	0
3-2	0.879	0.1841
4-1	0.394	0
4-2	0.6026	0
5-1	16.16	1.8796
5-2	2.1075	2.0196
6-1	2.0645	1.0194
6-2	2.8721	3.6696
6-3	2.7885	4.1717
6-4	3.4952	0.3618
6-5	6.8712	0.9606
7-1	16.3095	6.2028
7-2	8.7935	5.889
7-3	9.9934	7.9233
7-4	5.3866	1.3428
7-5	1.205	0.7338
7-6	1.7272	1.7772
8-1	128.0939	38.13
8-2	6.0024	3.8394
9-1	10.092	0.9374
10-1	5.3928	1.4297



Scatterplot of observed influent and effluent 30 to 60 µm particle size concentrations.



Paired influent and effluent 30 to 60 µm particle size concentrations.

Fitted Equation:

Effluent Turbidity, (30 to 60 µm particle size log mg/L) = -0.396 + 0.905 * (30 to 60 µm particle size log mg/L)

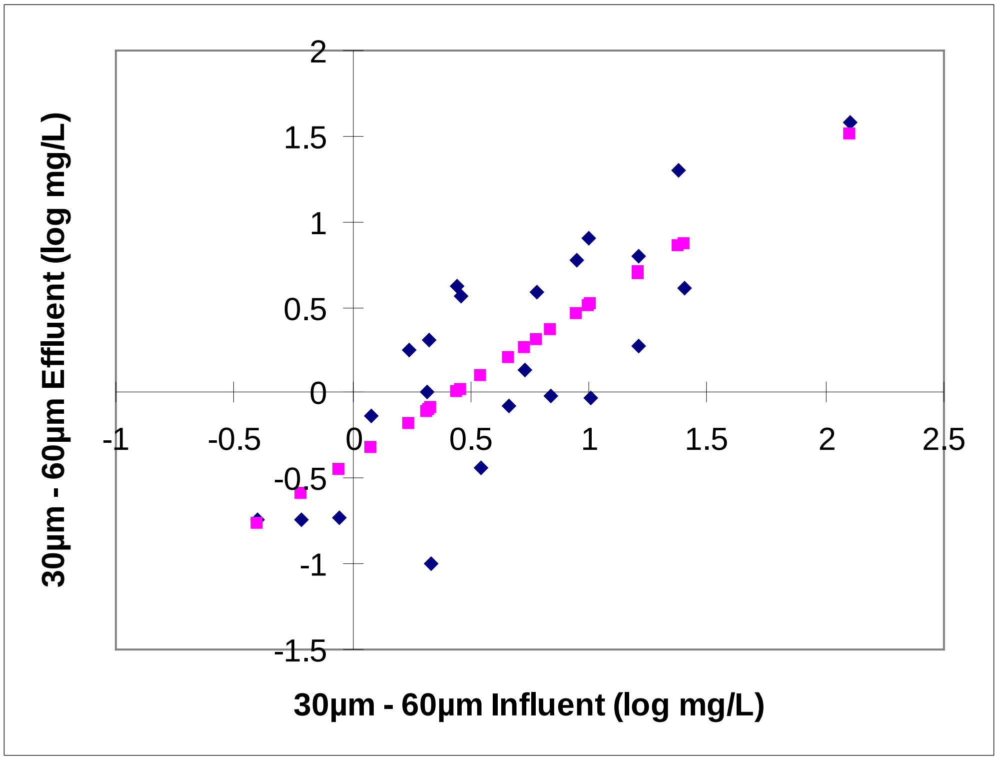
Regression Statistics

Multiple R	0.794
R Square	0.631
Adjusted R Square	0.613
Standard Error	0.414
Observations	23

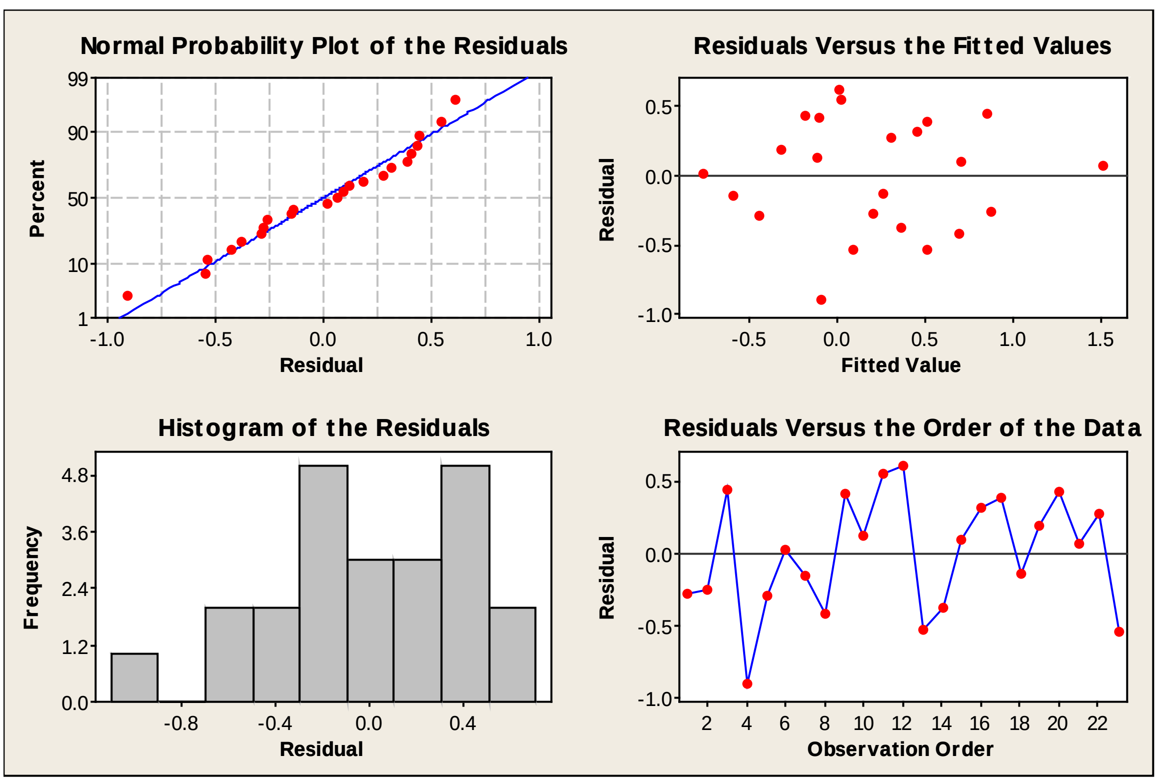
ANOVA

	df	SS	MS	F	Significance F
Regression	1	6.15	6.15	35.9	6.06E-06
Residual	21	3.60	0.172		
Total	22	9.75			

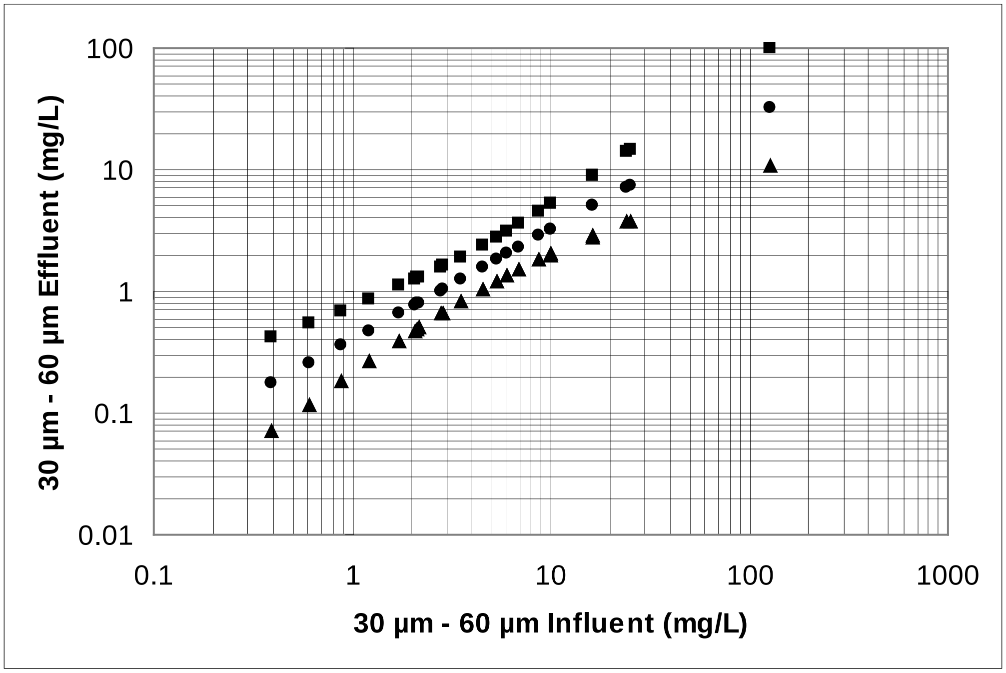
	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Intercept	-0.396	0.134	-2.98	0.00707	-0.672	-0.120
30µm - 60µm Log Influent	0.905	0.151	5.99	6.06	0.591	1.22



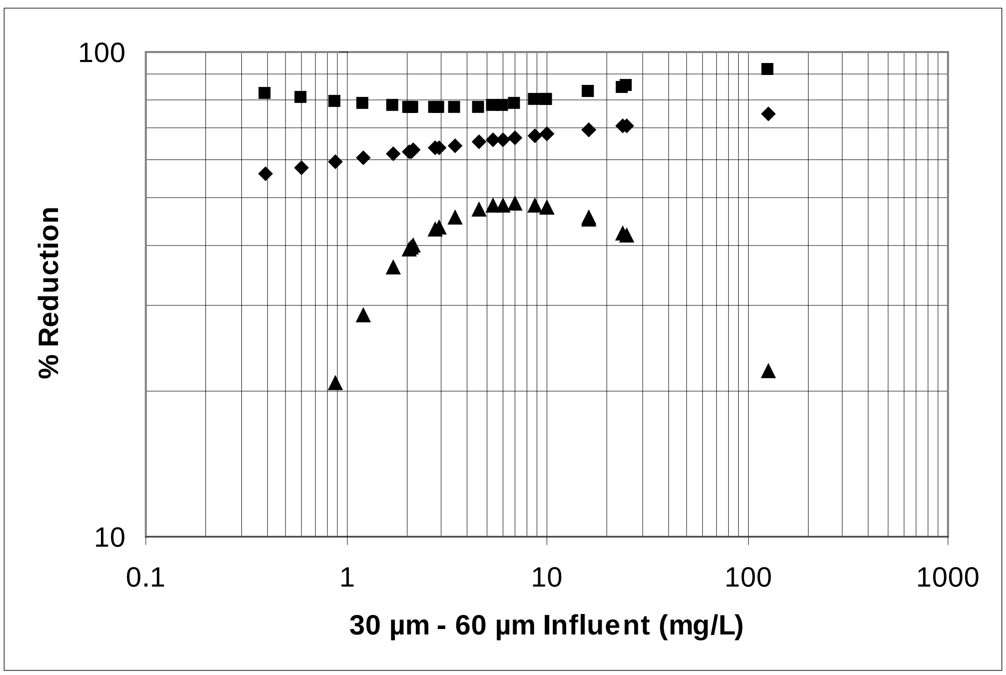
Fitted equation and data points for influent and effluent 30 to 60 µm particle size.



Residual analyses of fitted equation for 30 to 60 µm particle size influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

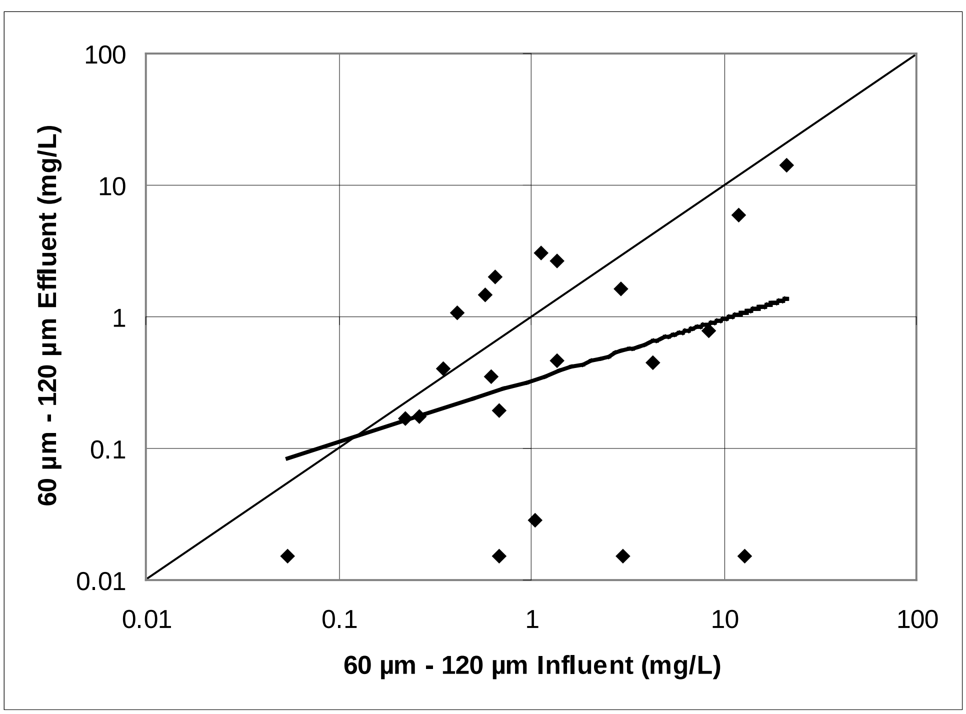


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

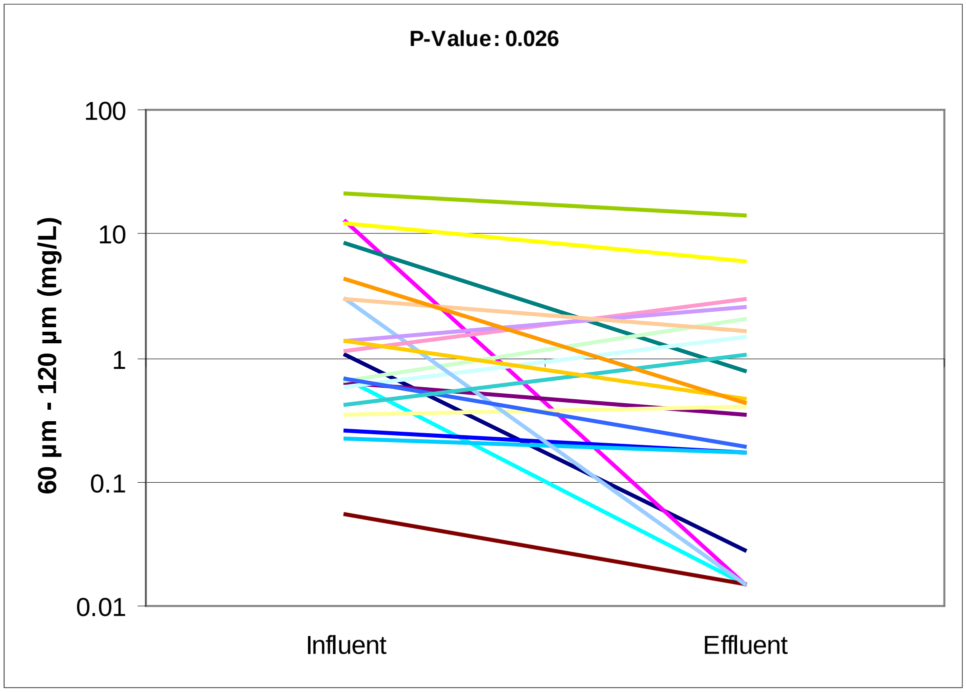
60 to 120 µm Particle Size

Observed 60 to 120 µm Particle Size Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)
1-1	1.0523	0.028
2-1	12.7041	0
2-2	12.055	5.9422
3-1	0.6768	0
3-2	0.6195	0.3467
4-1	0.0543	0
4-2	0	0
5-1	8.288	0.7955
5-2	0.261	0.1734
6-1	0.2235	0.1698
6-2	0.5753	1.4848
6-3	0.6435	2.0319
6-4	0.3519	0.4038
6-5	2.9988	0
7-1	1.1205	3.0312
7-2	1.3631	2.613
7-3	2.9609	1.65
7-4	0	0.4596
7-5	0.69	0.1936
7-6	0.4116	1.0548
8-1	21.2729	14.115
8-2	1.3735	0.4716
9-1	4.2775	0.441
10-1	1.2024	0.0272



Scatterplot of observed influent and effluent 60 to 120 µm particle size concentrations.



Paired influent and effluent 60 to 120 µm particle size concentrations.

Fitted Equation:

Effluent Turbidity, (60 to 120 µm particle size log mg/L) = -0.483 + 0.466 * (60 to 120 µm particle size log mg/L)

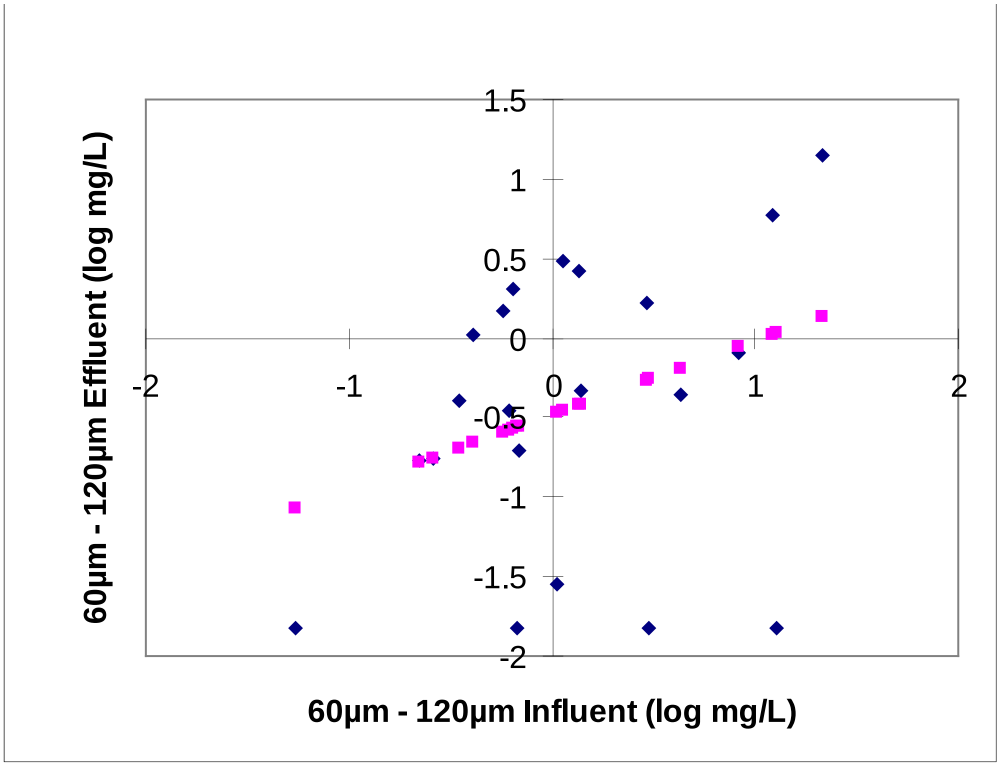
Regression Statistics

Multiple R	0.336
R Square	0.113
Adjusted R Square	0.0665
Standard Error	0.877
Observations	21

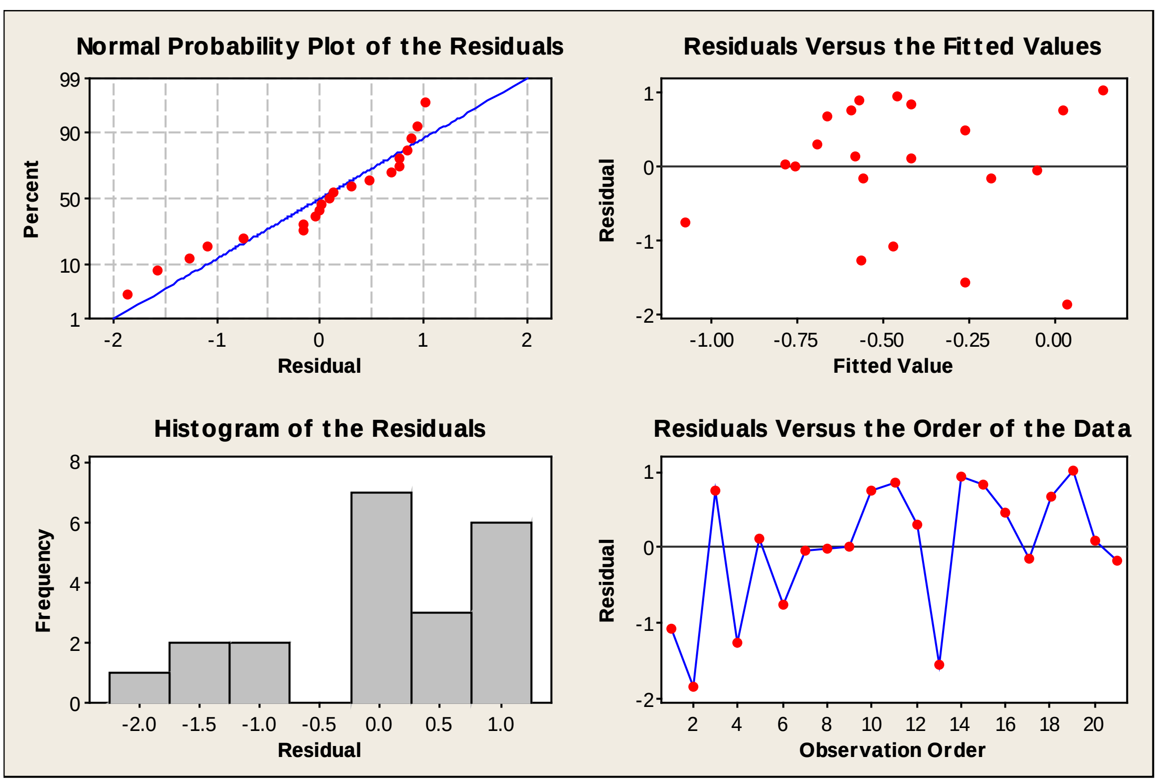
ANOVA

	df	SS	MS	F	Significance F
Regression	1	1.87	1.87	2.42	0.136
Residual	19	14.6	0.770		
Total	20	16.5			

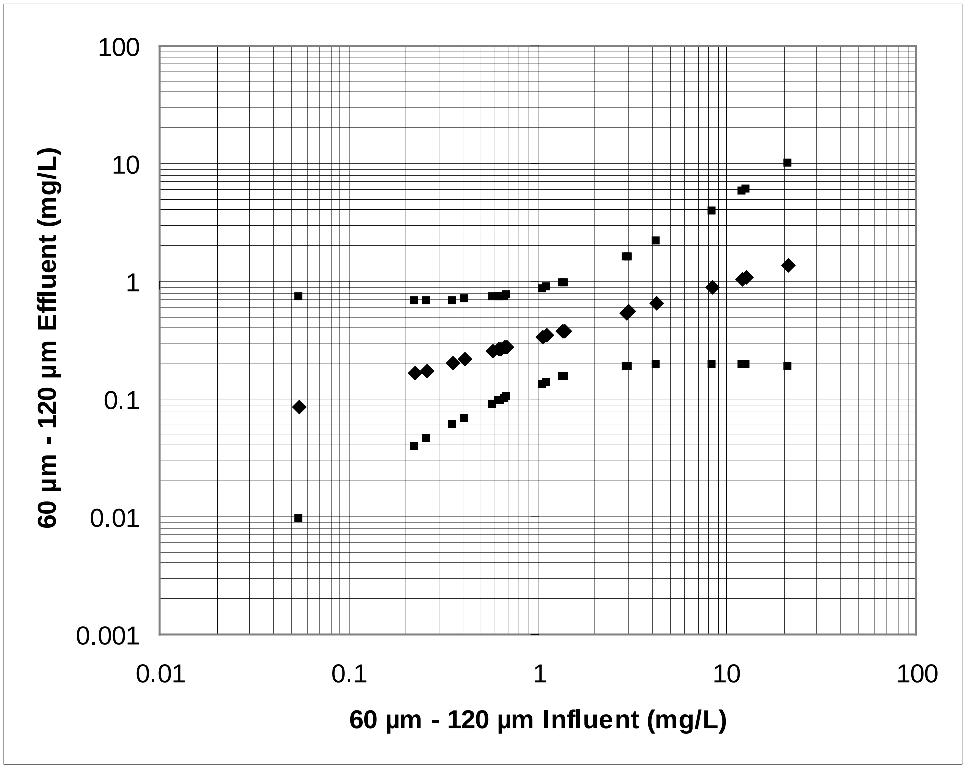
	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Intercept	-0.483	0.194	-2.49	0.022	-0.888	-0.0775
60µm - 120µm Log Influent	0.466	0.299	1.562	0.136	-0.160	1.09



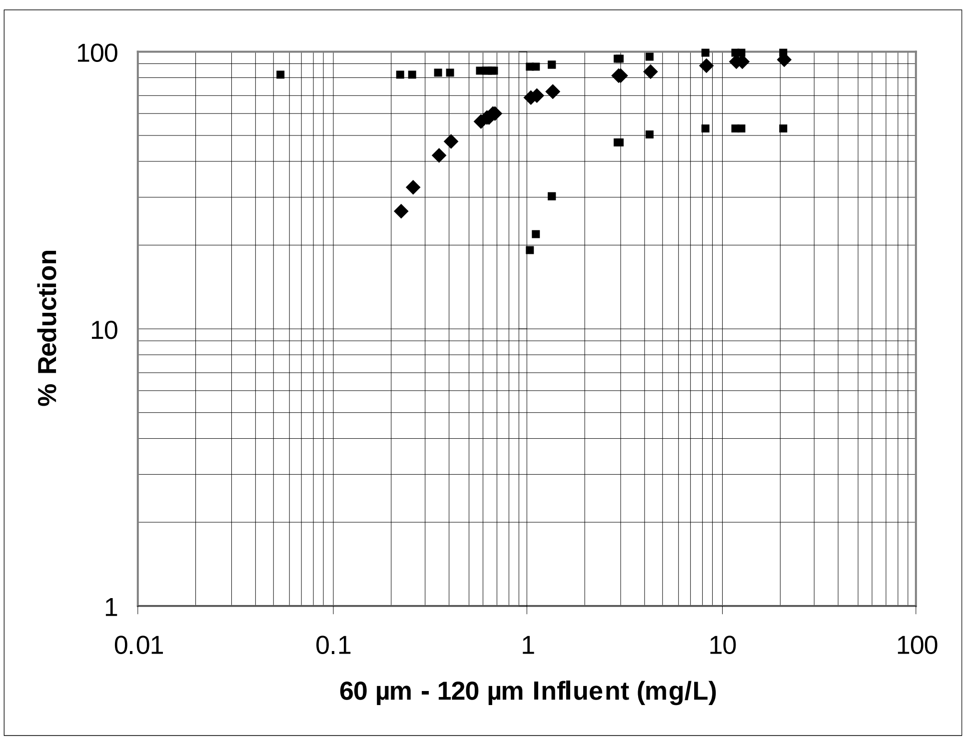
Fitted equation and data points for influent and effluent 60 to 120 µm particle size.



Residual analyses of fitted equation for 60 to 120 µm particle size influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.



Percentage reductions as a function of influent concentrations, with 95% confidence limits.

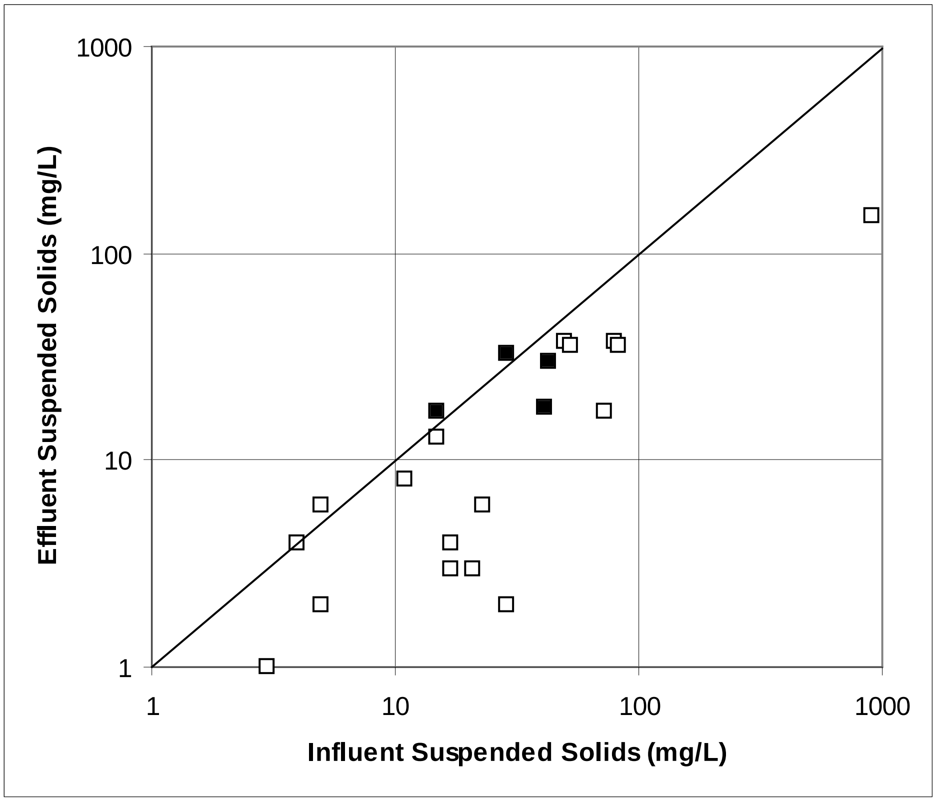
Appendix C

Actual Storm Events Pollutant Removal Performance, Box, Scatter and Probability plots

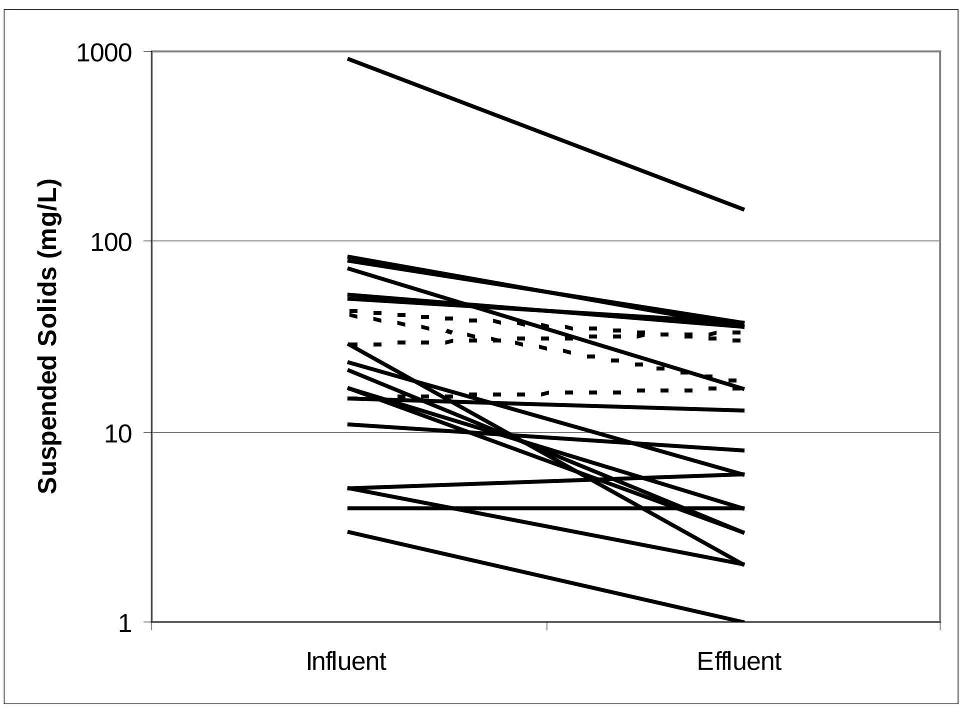
Suspended Solids

Observed Suspended Solids Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)
1-1	17	4
2-1	53	36
2-2	50	37
3-1	6	0
3-2	3	1
4-1	1	0
4-2	1	0
5-1	80	37
5-2	15	17
6-1	5	6
6-2	11	8
6-3	15	13
6-4	17	3
6-5	21	3
7-1	83	36
7-2	43	30
7-3	29	33
7-4	23	6
7-5	5	2
7-6	4	4
8-1	913	150
8-2	41	18
9-1	29	2
10-1	72	17



Scatterplot of observed influent and effluent suspended solids concentrations (filled symbols are events that had minor filter bypasses).



Paired influent and effluent suspended solids concentrations.

Fitted Equation:

Effluent Suspended Solids, log mg/L = 0.730 * (Influent Suspended Solids, log mg/L)

Regression Statistics on Observed Influent vs. Effluent Suspended Solids, log mg/L

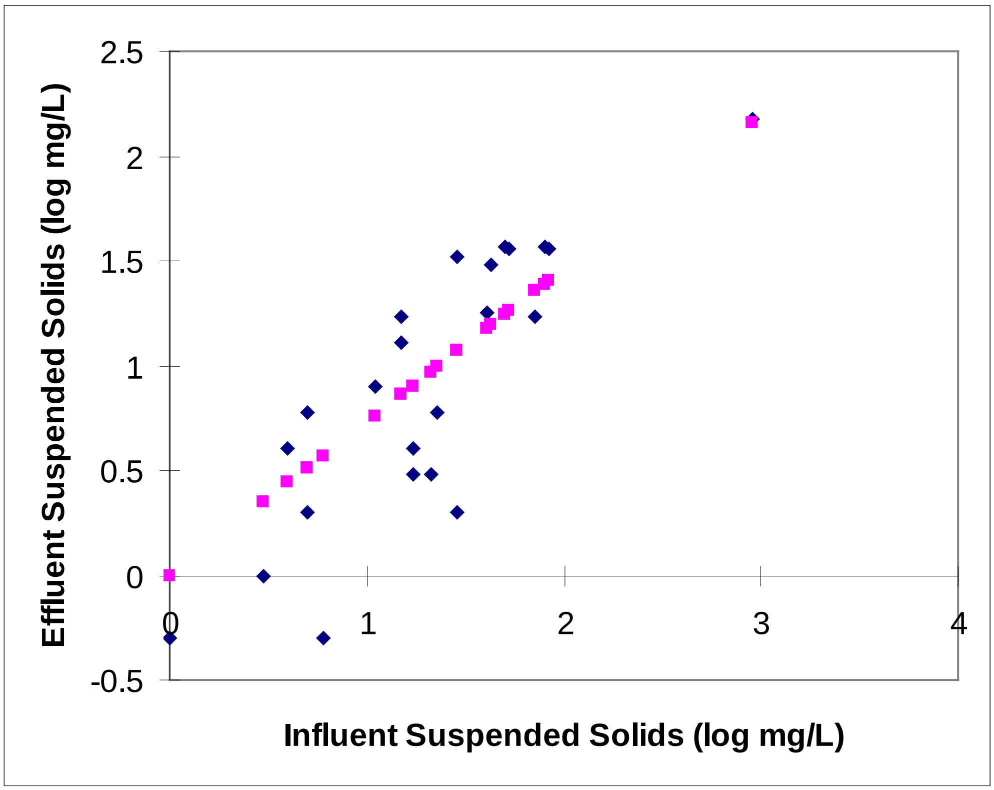
Multiple R	0.94
R Square	0.89
Adjusted R Square	0.85
Standard Error	0.37
Observations	24

ANOVA

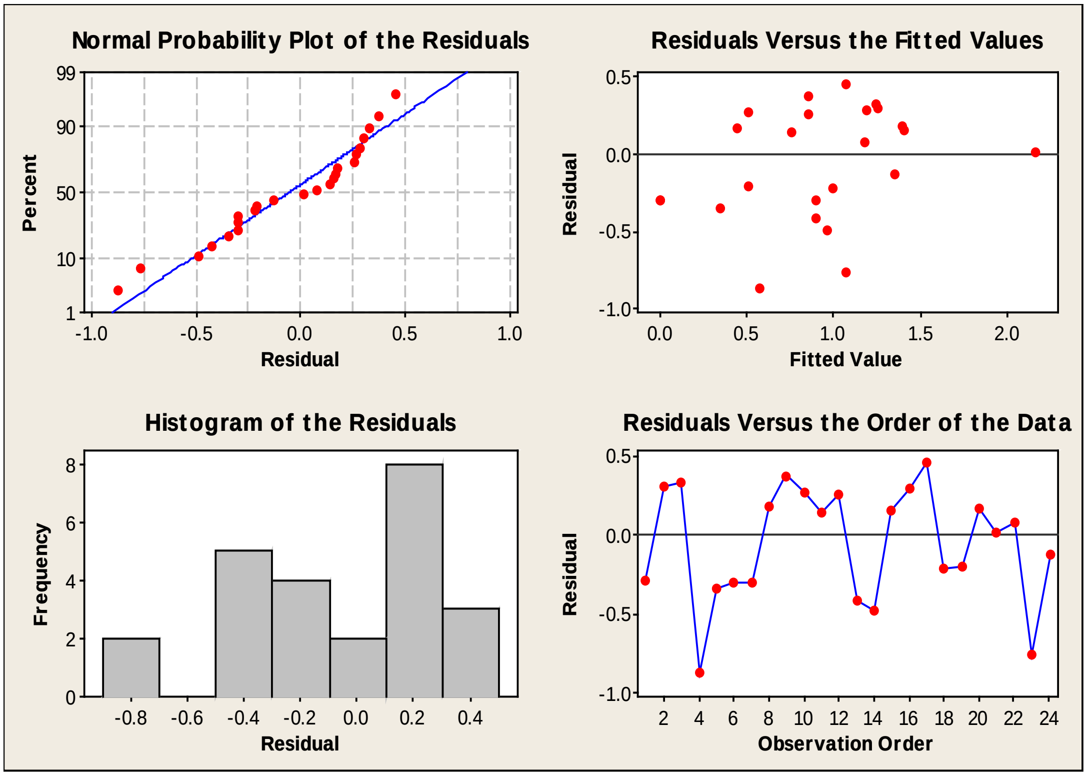
	df	SS	MS	F	Significance F
Regression	1	25.4	25.4	187	3.11E-12
Residual	23	3.12	0.136		
Total	24	28.55			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
X Variable 1*	0.730	0.053	13.7	1.56E-12	0.620	0.841

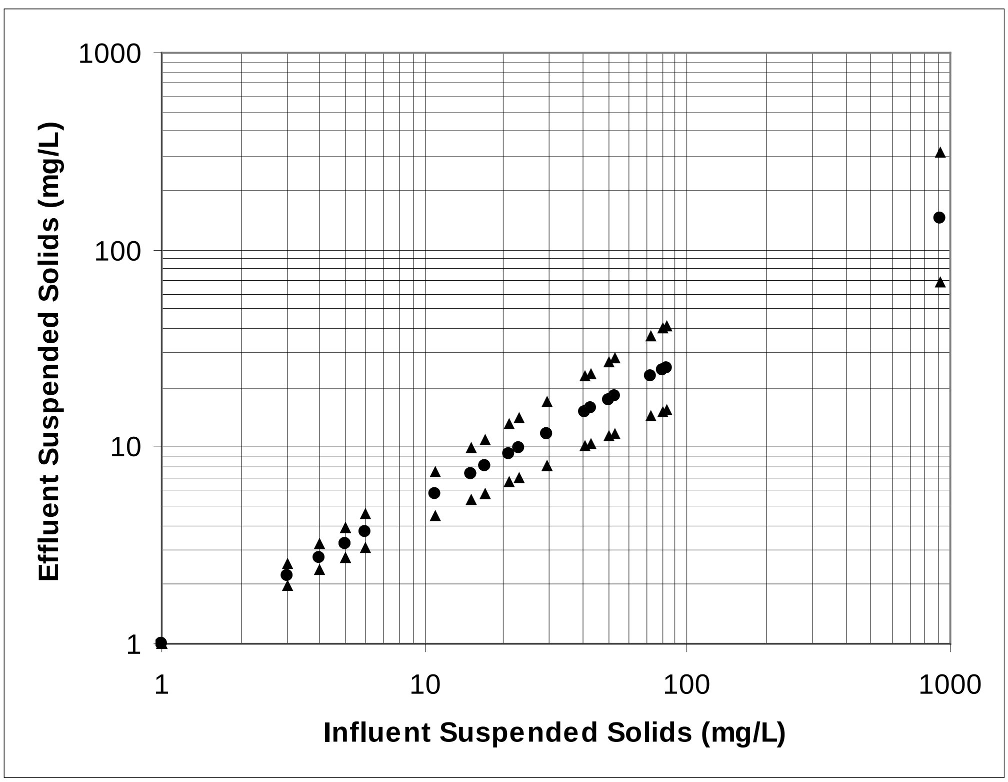
* the intercept term was determined to be not significant



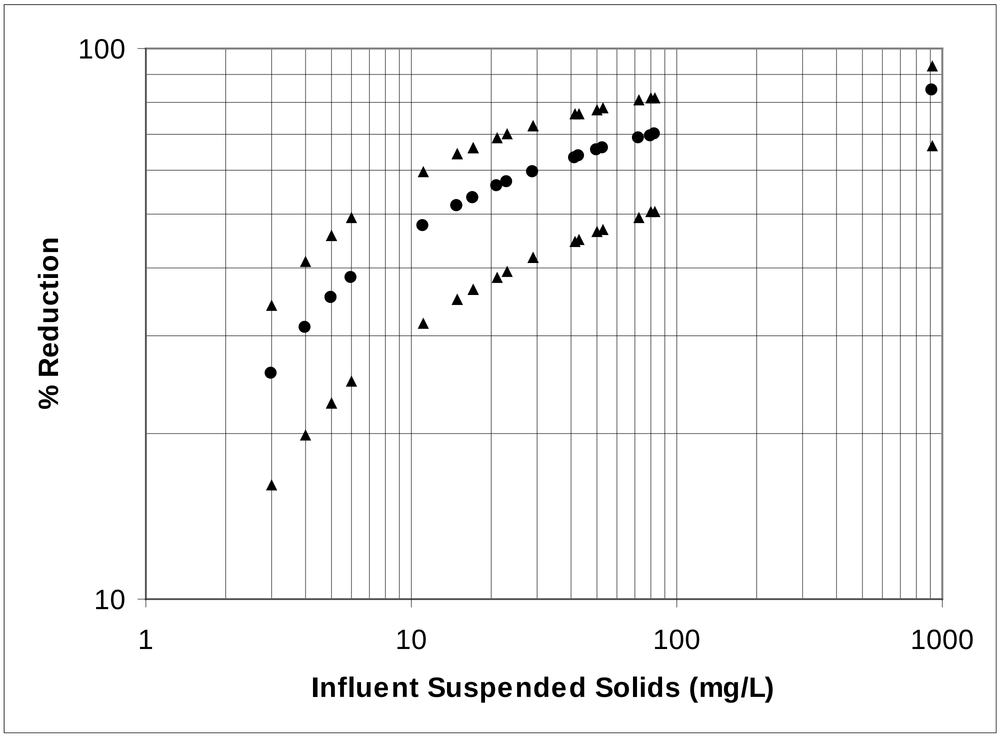
Fitted equation and data points for influent and effluent suspended solids.



Residual analyses of fitted equation for suspended solids influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

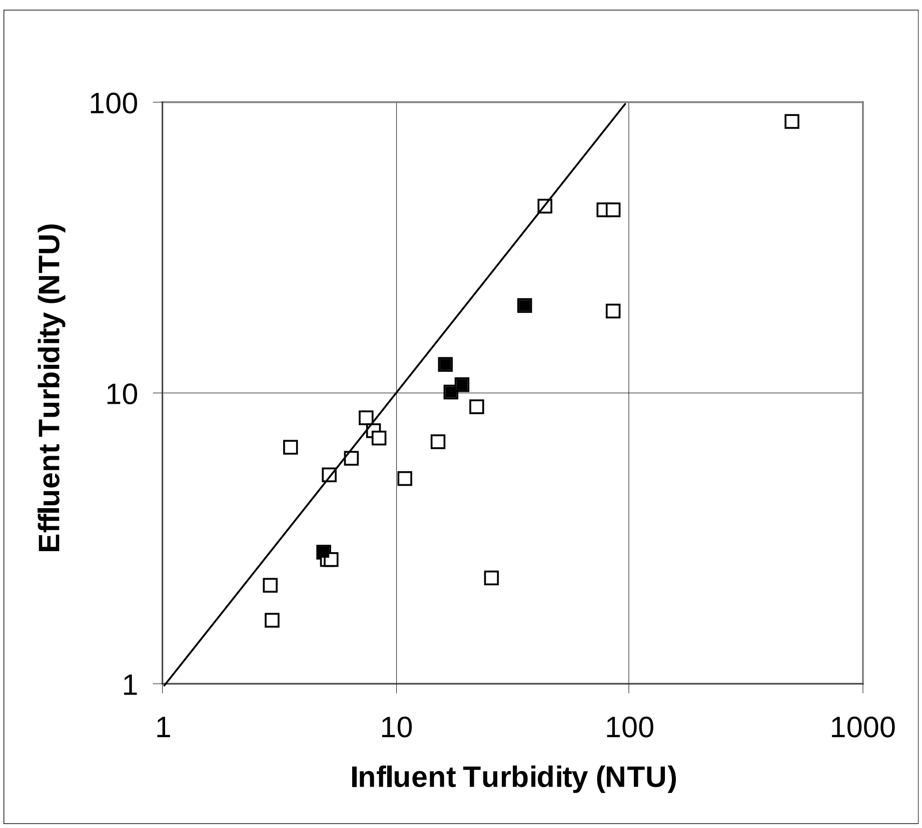


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

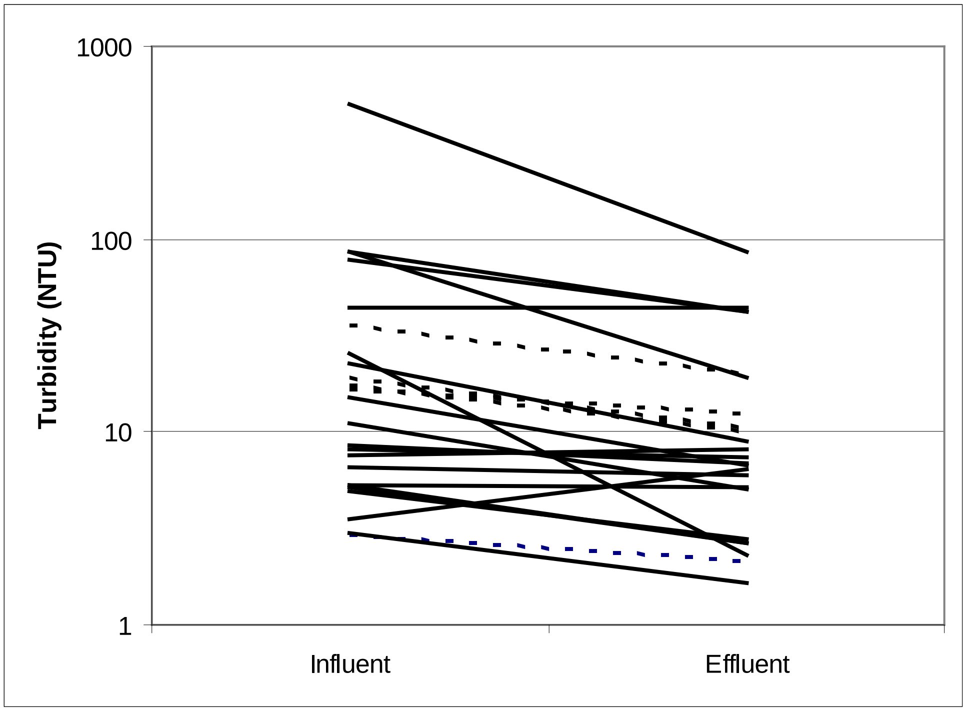
Turbidity

Observed Turbidity Concentrations

Sample Number	Influent (NTU)	Effluent (NTU)
1-1	15.2	6.76
2-1	85.6	42.4
2-2	78.6	42.3
3-1	8.53	6.95
3-2	2.97	1.64
4-1	4.93	2.8
4-2	2.93	2.16
5-1	22.5	8.94
5-2	17.5	9.98
6-1	8.08	7.33
6-2	6.51	5.92
6-3	5.35	2.65
6-4	7.51	8.15
6-5	3.54	6.45
7-1	44.2	43.7
7-2	19.3	10.6
7-3	16.5	12.5
7-4	5.27	5.21
7-5	11.1	5.02
7-6	5.15	2.67
8-1	502	85.8
8-2	35.8	19.9
9-1	25.7	2.28
10-1	85.7	19.0



Scatterplot of observed influent and effluent turbidity concentrations (filled symbols are events that had minor filter bypasses).



Paired influent and effluent Turbidity concentrations.

Fitted Equation:

Effluent Turbidity, log NTU = 0.772 * (Influent Turbidity, log NTU)

Regression Statistics on Observed Influent vs. Effluent Turbidity, log NTU

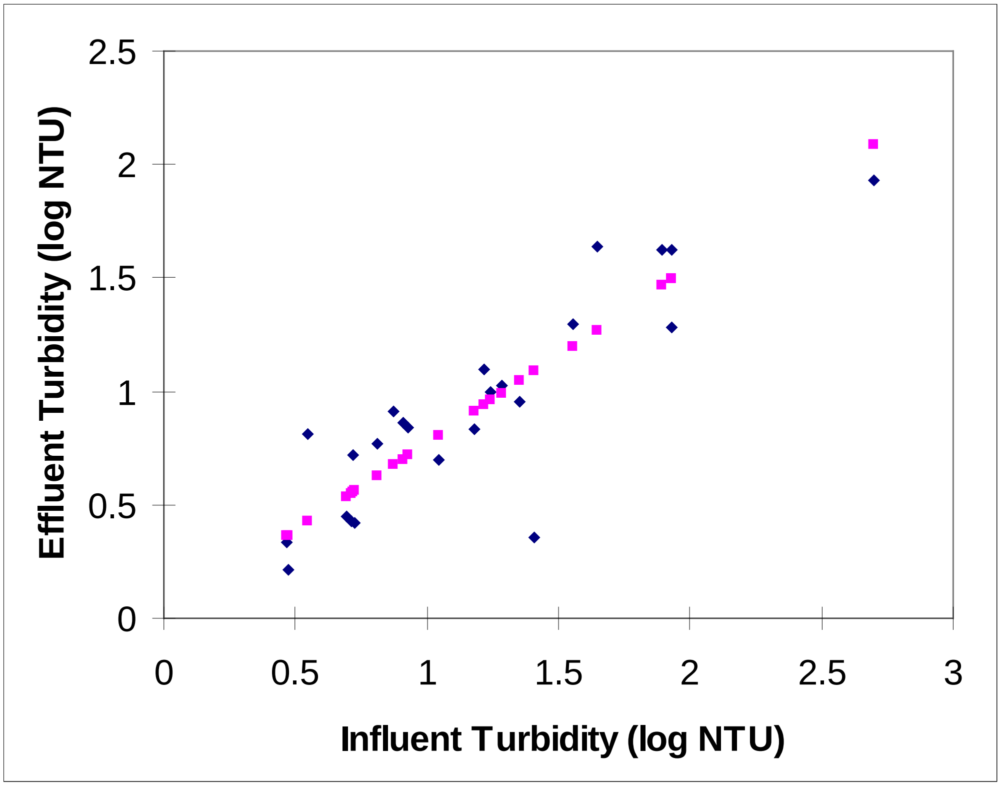
Multiple R	0.98
R Square	0.95
Adjusted R Square	0.91
Standard Error	0.23
Observations	24

ANOVA

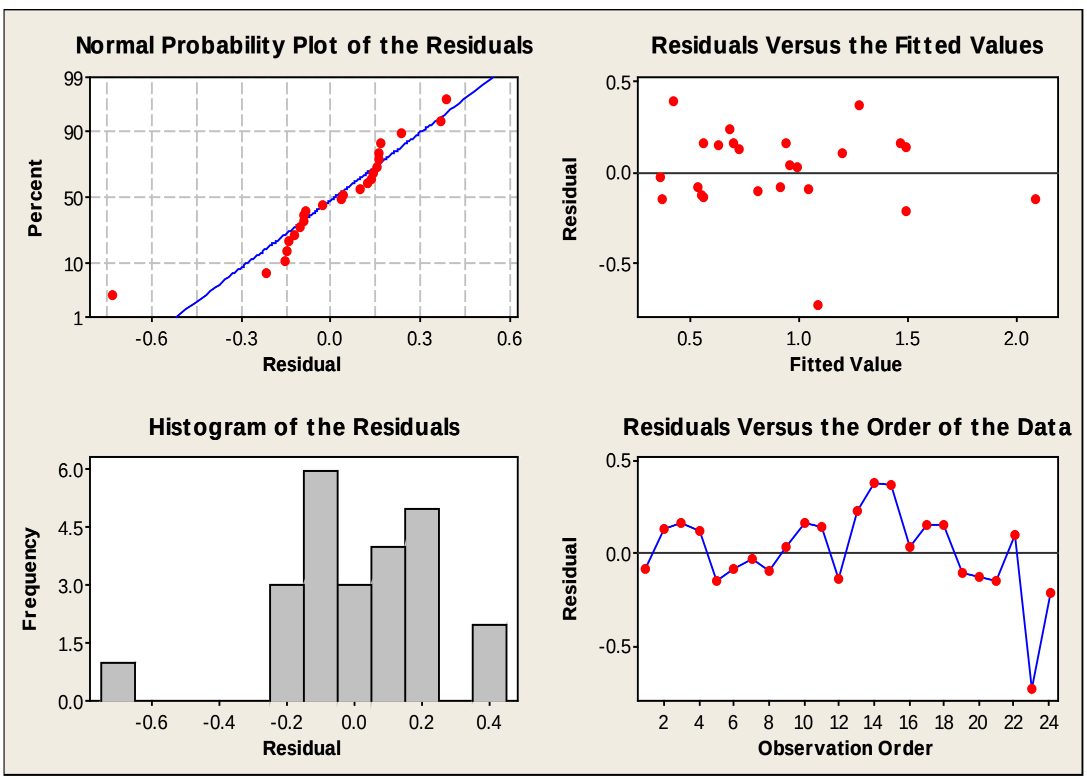
	df	SS	MS	F	Significance F
Regression	1	24.1	24.1	459	3.14E-16
Residual	23	1.21	0.0524		
Total	24	25.3			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Influent Turbidity*	0.772	0.0360	21.4	1.070E-16	0.698	0.847

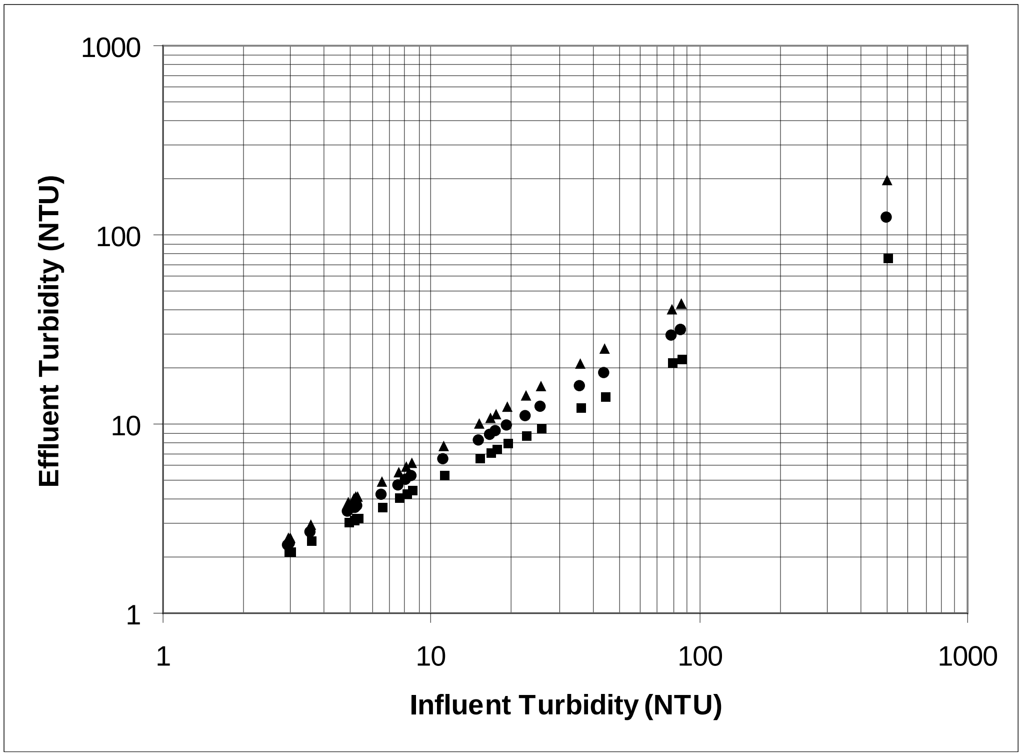
* the intercept term was determined to be not significant



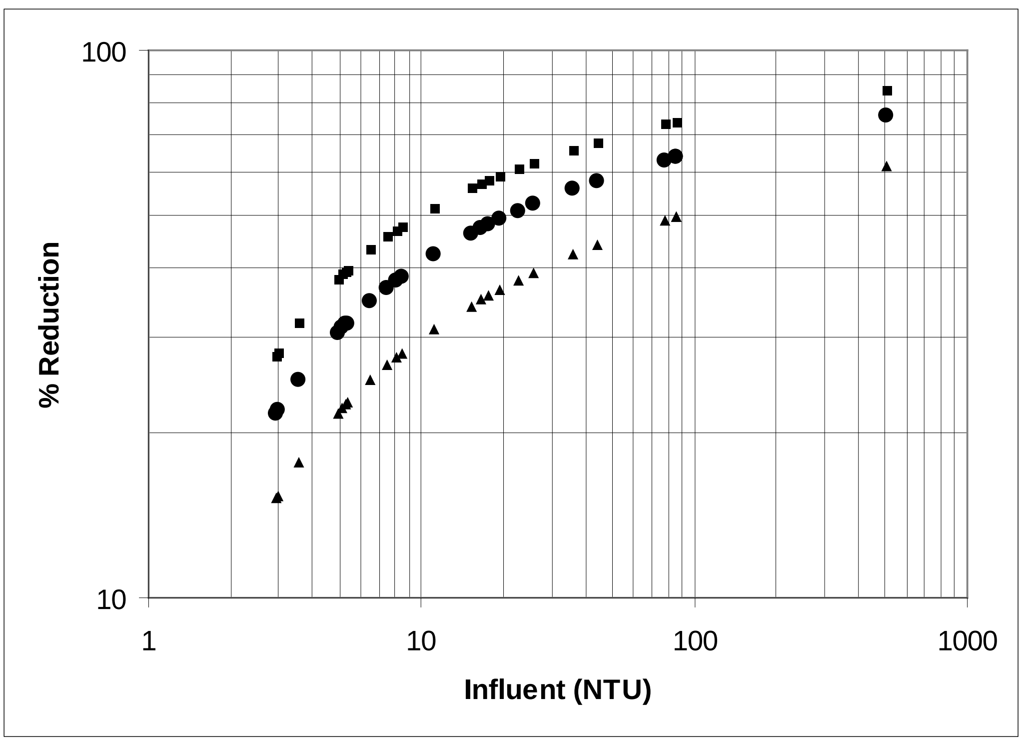
Fitted equation and data points for influent and effluent turbidity.



Residual analyses of fitted equation for turbidity influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

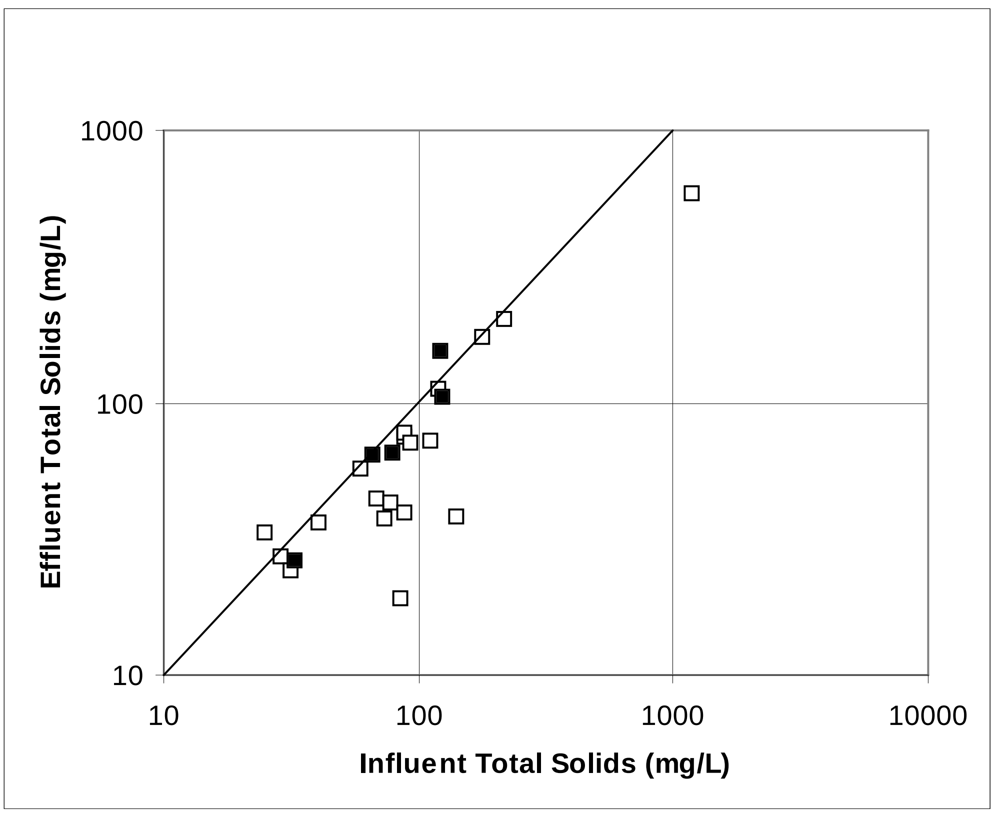


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

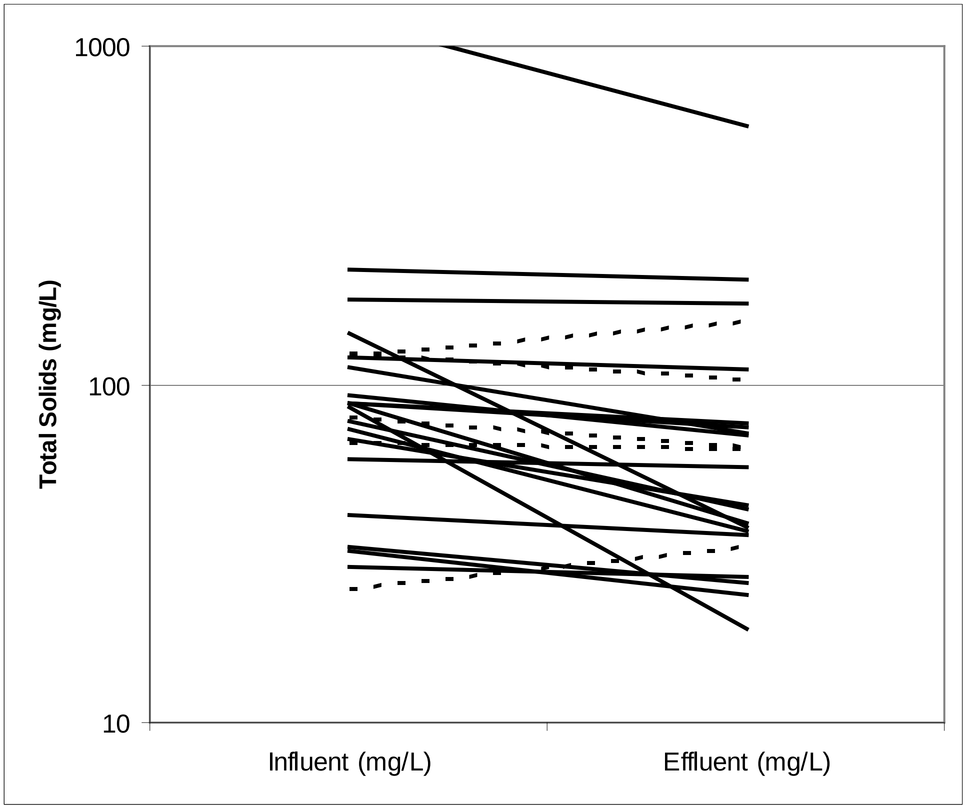
Total Solids

Observed Total Solids Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)
1-1	69	44
2-1	88	75
2-2	93	71
3-1	41	36
3-2	142	38
4-1	33	26
4-2	25	33
5-1	217	203
5-2	124	104
6-1	120	111
6-2	88	77
6-3	60	57
6-4	78	43
6-5	88	39
7-1	178	173
7-2	80	65
7-3	67	64
7-4	29	27
7-5	74	37
7-6	32	24
8-1	1192	580
8-2	122	153
9-1	112	72
10-1	86	19



Scatterplot of observed influent and effluent total solids concentrations (filled symbols are events that had minor filter bypasses).



Paired influent and effluent total solids concentrations.

Fitted Equation:

Effluent Total Solids, log mg/L = 0.928 * (Influent Total Solids, log mg/L)

Regression Statistics on Observed Influent vs. Effluent Total Solids, log mg/L

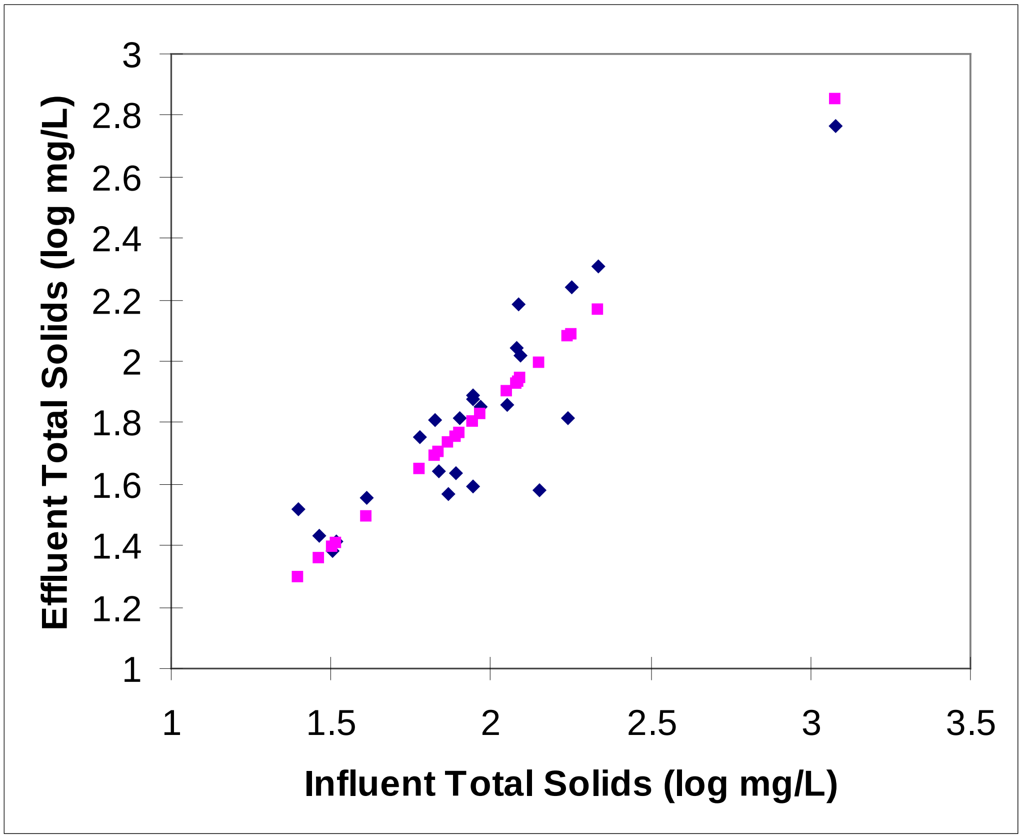
Multiple R	0.99
R Square	0.99
Adjusted R Square	0.95
Standard Error	0.16
Observations	24

ANOVA

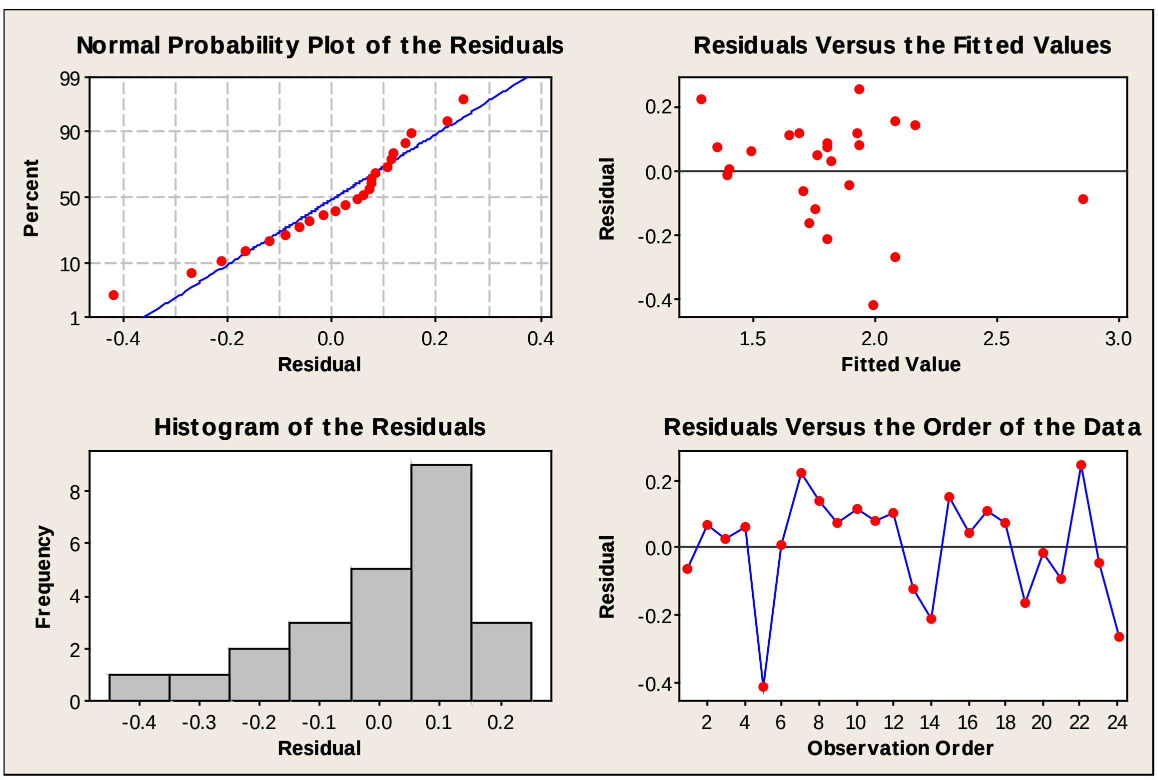
	df	SS	MS	F	Significance F
Regression	1	80.8	80.8	3274	1.98E-25
Residual	23	0.568	0.0247		
Total	24	81.4			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Influent total solids	0.928	0.0162	57.2	2.63E-26	0.894	0.961

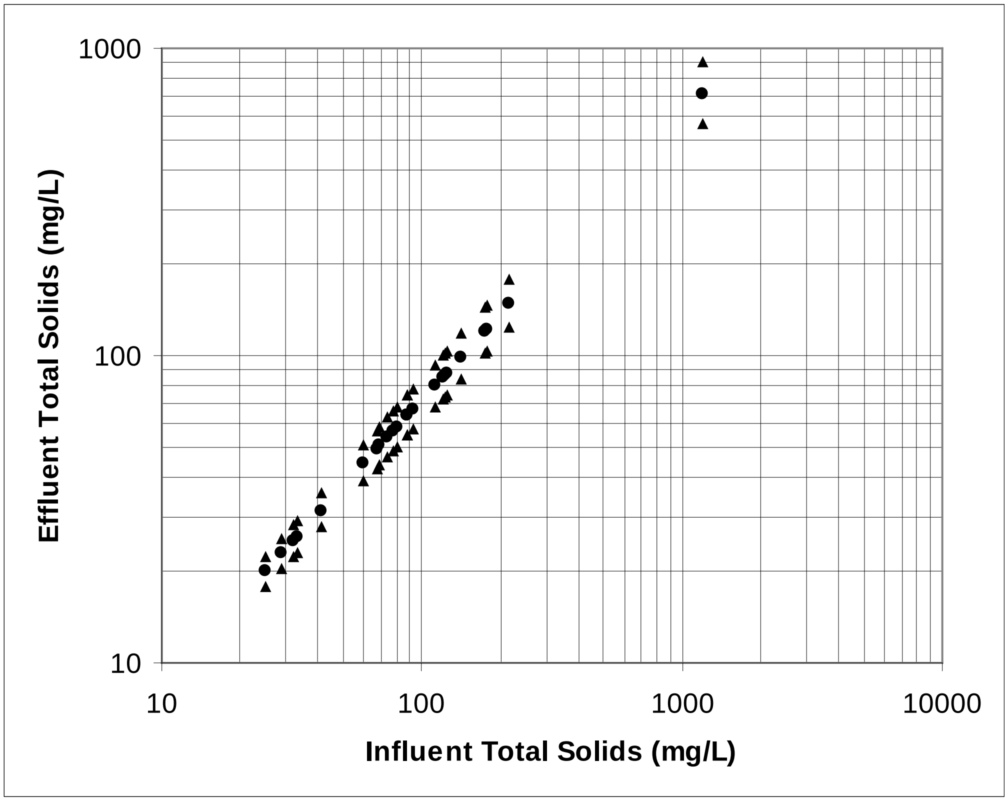
* the intercept term was determined to be not significant



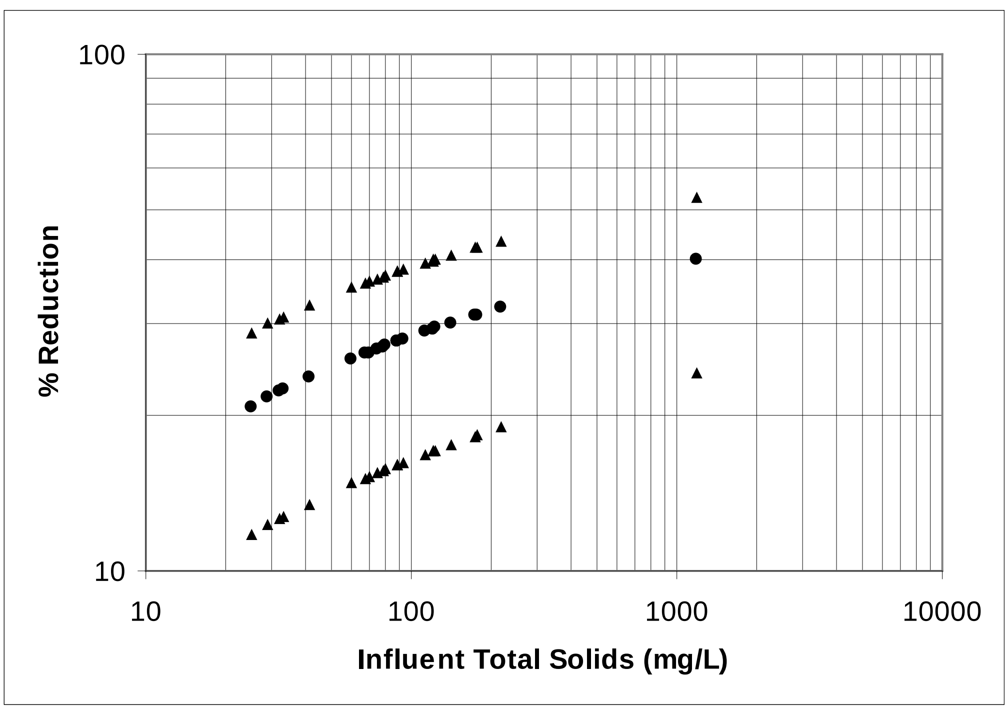
Fitted equation and data points for influent and effluent total solids.



Residual analyses of fitted equation for total solids influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

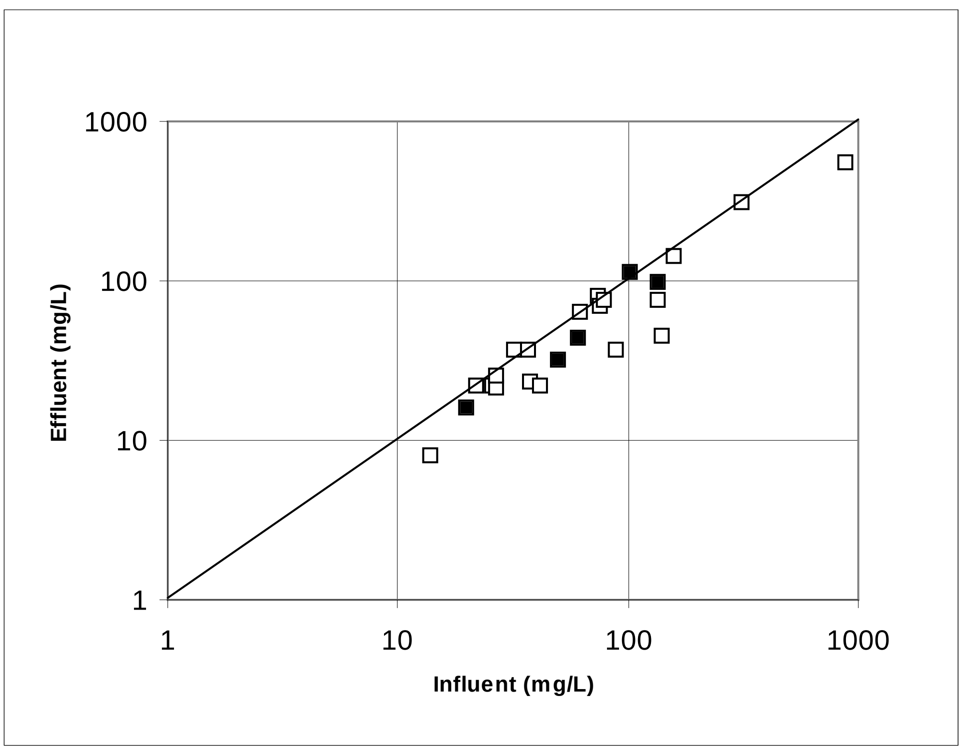


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

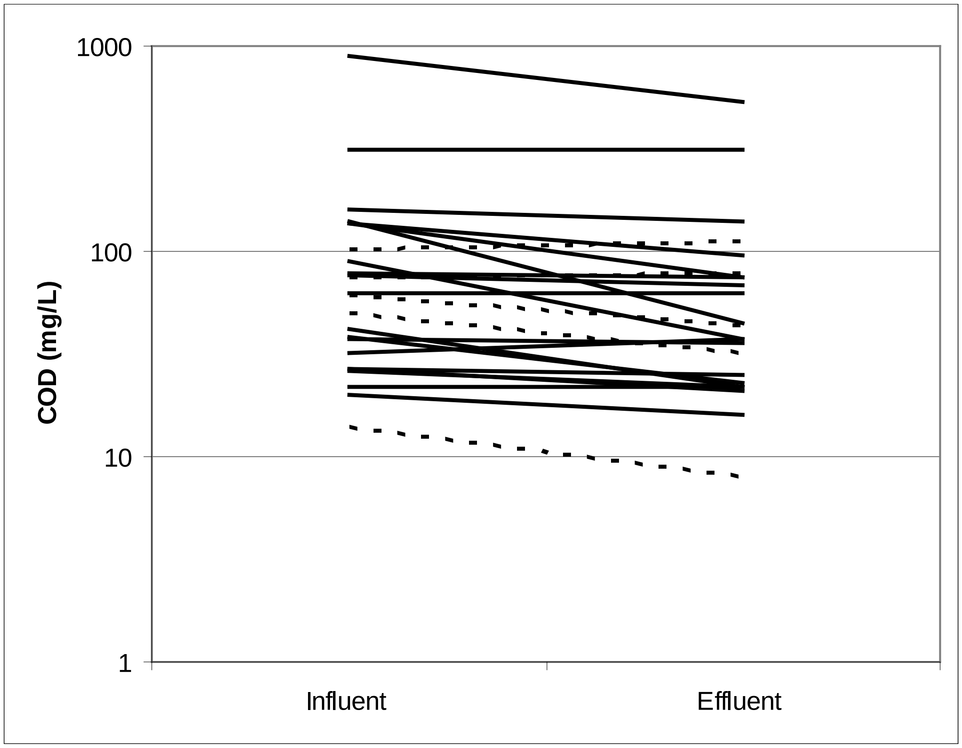
COD

Observed COD Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)
1-1	89	37
2-1	141	45
2-2	137	75
3-1	38	23
3-2	42	22
4-1	20	16
4-2	14	8
5-1	159	141
5-2	75	79
6-1	79	75
6-2	62	63
6-3	37	36
6-4	27	21
6-5	26	22
7-1	137	96
7-2	61	44
7-3	50	32
7-4	22	22
7-5	32	37
7-6	27	25
8-1	891	540
8-2	103	112
9-1	76	68
10-1	312	310



Scatterplot of observed influent and effluent COD concentrations (filled symbols are events that had minor filter bypasses).



Paired influent and effluent COD concentrations.

Fitted Equation:

Effluent COD, log mg/L = 0.933 * (Influent COD, log mg/L)

Regression Statistics

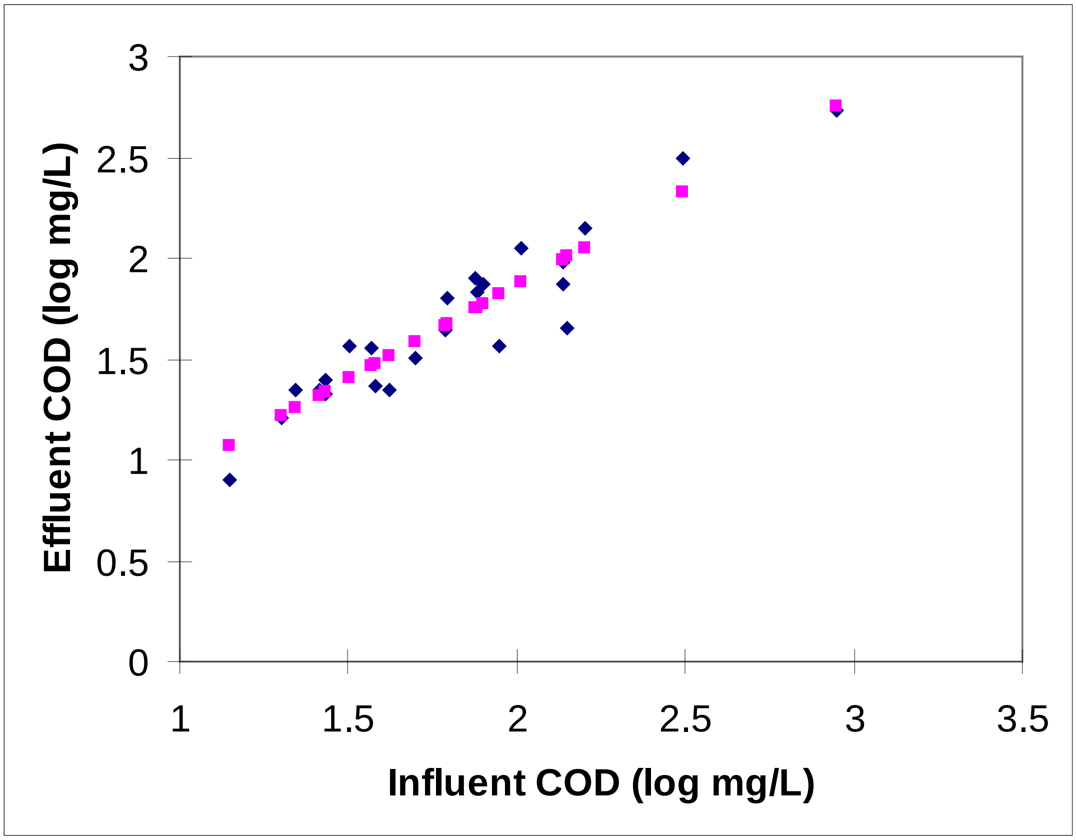
Multiple R	0.997
R Square	0.994
Adjusted R Square	0.950
Standard Error	0.139
Observations	24

ANOVA

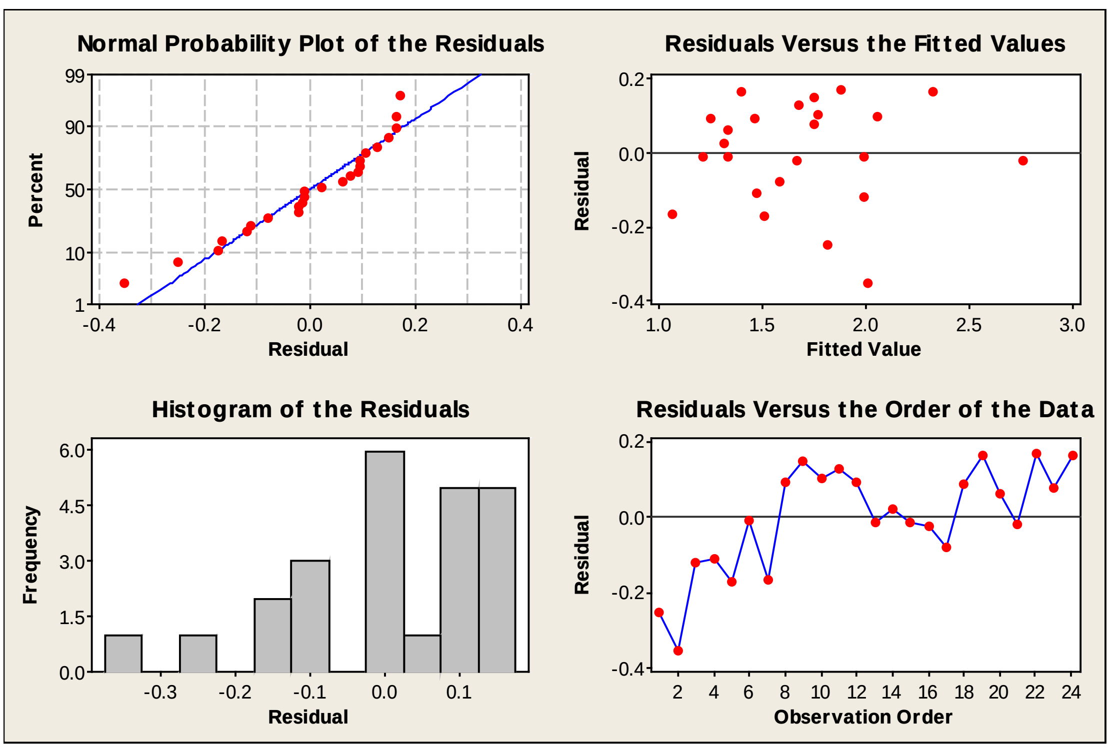
	df	SS	MS	F	Significance F
Regression	1	71.5	71.5	3679	5.52E-26
Residual	23	0.447	0.0194		
Total	24	71.9			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Influent COD*	0.933	0.0154	60.7	6.91E-27	0.901	0.965

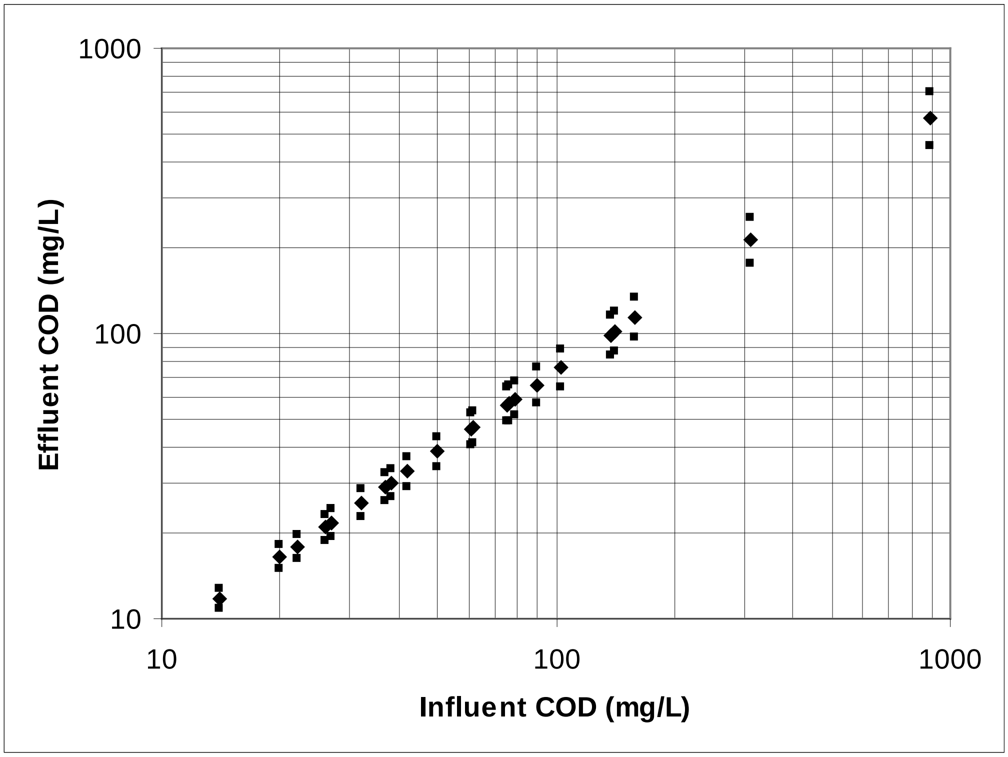
* the intercept term was determined to be not significant



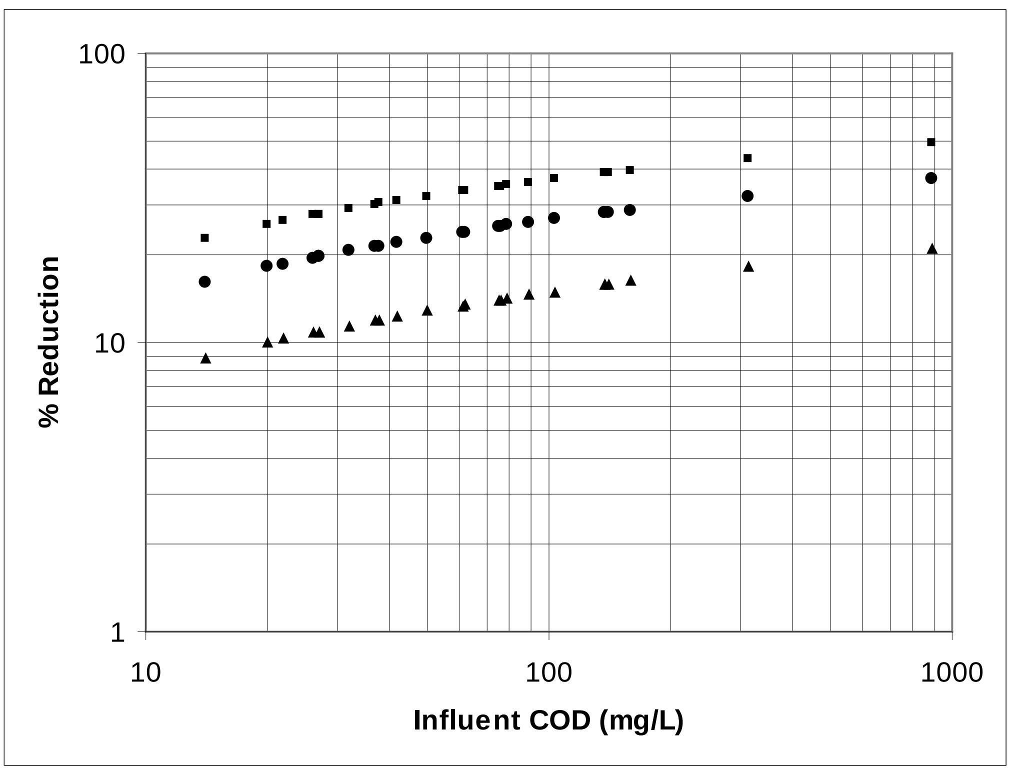
Fitted equation and data points for influent and effluent COD.



Residual analyses of fitted equation for COD influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

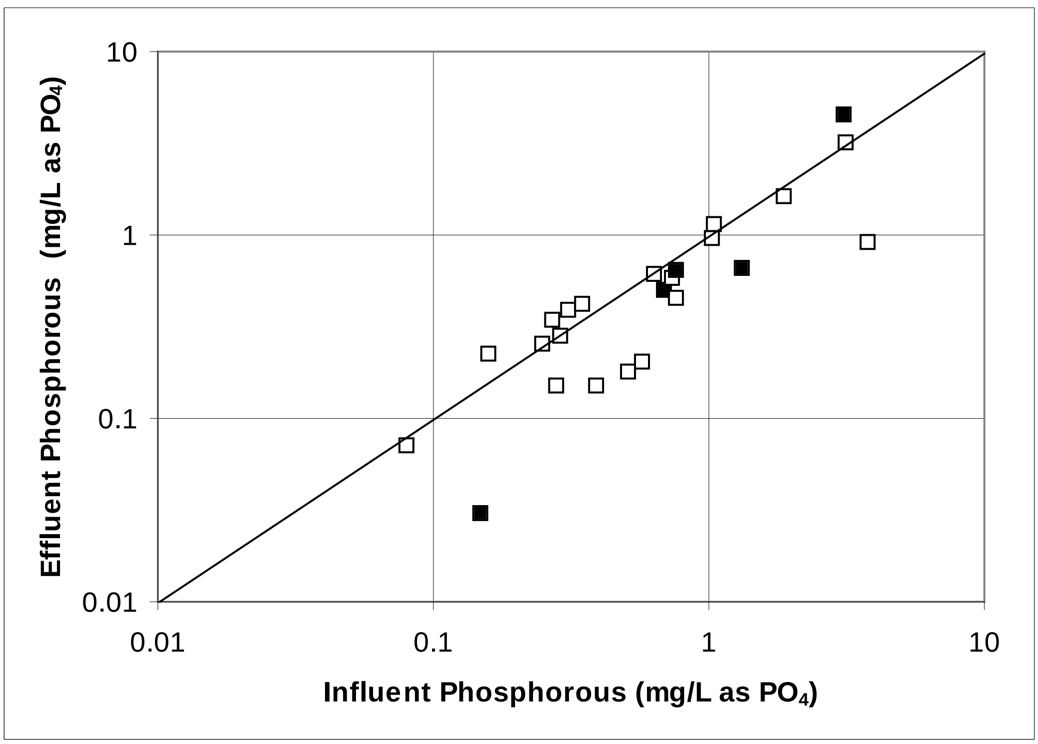


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

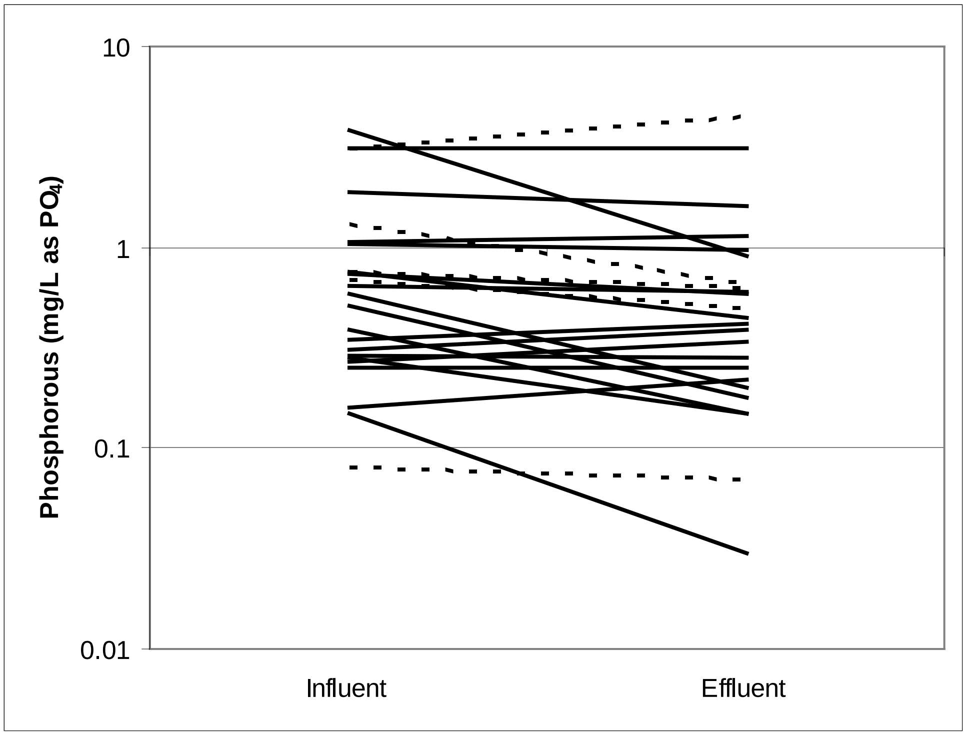
Phosphorus

Observed Phosphorus Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)
1-1	0.35	0.42
2-1	0.64	0.6
3-1	0.25	0.25
3-2	0.58	0.2
4-1	0.15	0.03
4-2	0.08	0.07
5-1	1.06	1.14
5-2	0.76	0.63
6-1	0.31	0.39
6-2	0.27	0.34
6-2	0.28	0.15
6-3	0.39	0.15
6-5	0.16	0.22
7-1	1.88	1.62
7-2	0.69	0.5
7-3	1.32	0.66
7-4	0.29	0.28
7-5	0.76	0.45
7-6	0.51	0.18
8-1	3.14	3.14
8-2	0.74	0.58
8-2	3.12	4.54
9-1	1.04	0.96
10-1	3.81	0.9



Scatterplot of observed influent and effluent phosphorus concentrations (filled symbols are events that had minor filter bypasses).



Paired influent and effluent phosphorus concentrations.

Fitted Equation:

Effluent phosphorus, log mg/L = -0.145 + 0.979 * (Influent phosphorus, log mg/L)

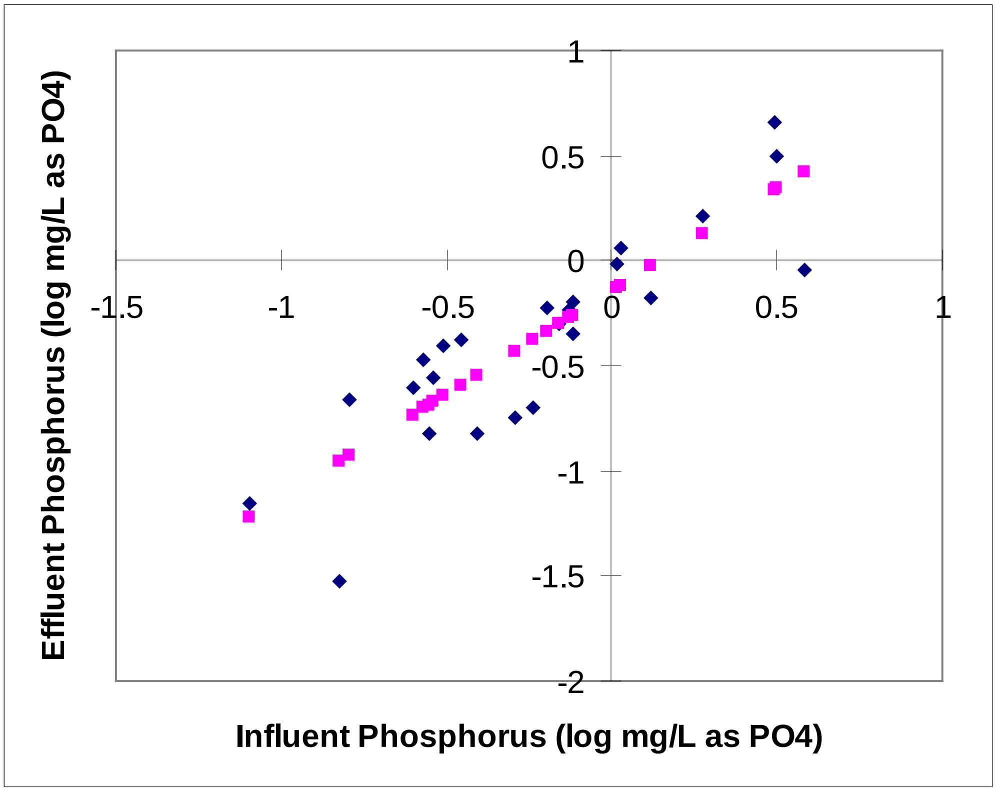
Regression Statistics

Multiple R	0.869
R Square	0.756
Adjusted R Square	0.744
Standard Error	0.246
Observations	24

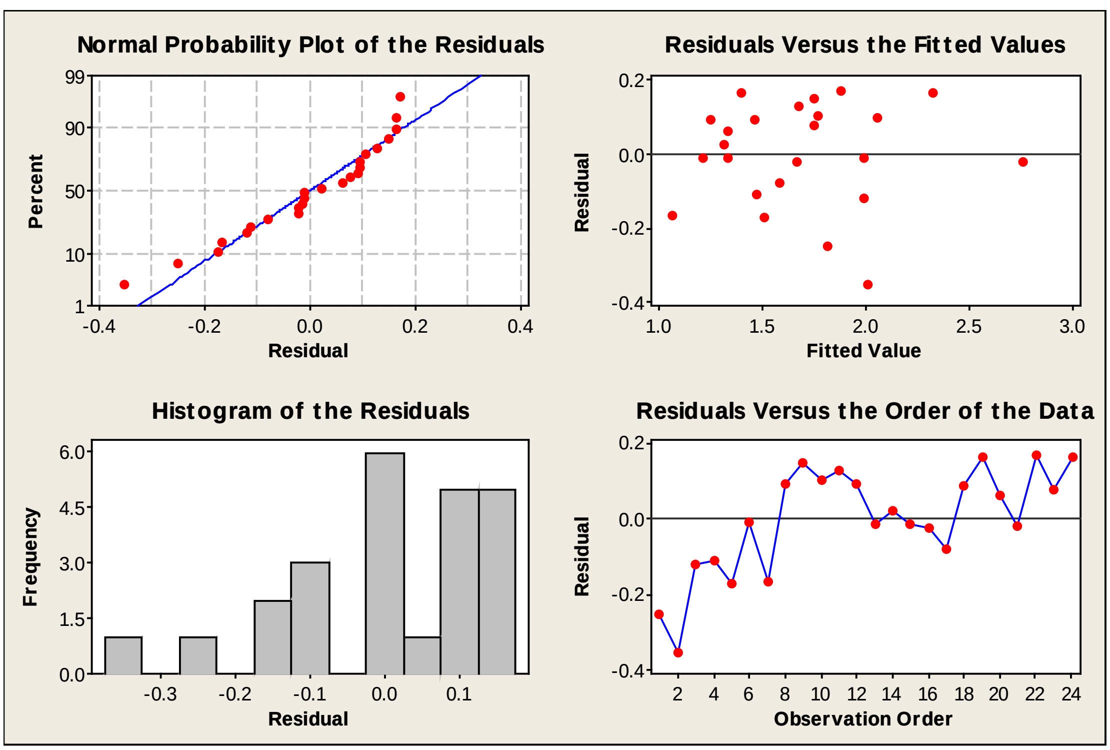
ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	4.12	4.12	68.0	3.56E-08
Residual	22	1.33	0.0606		
Total	23	5.45			

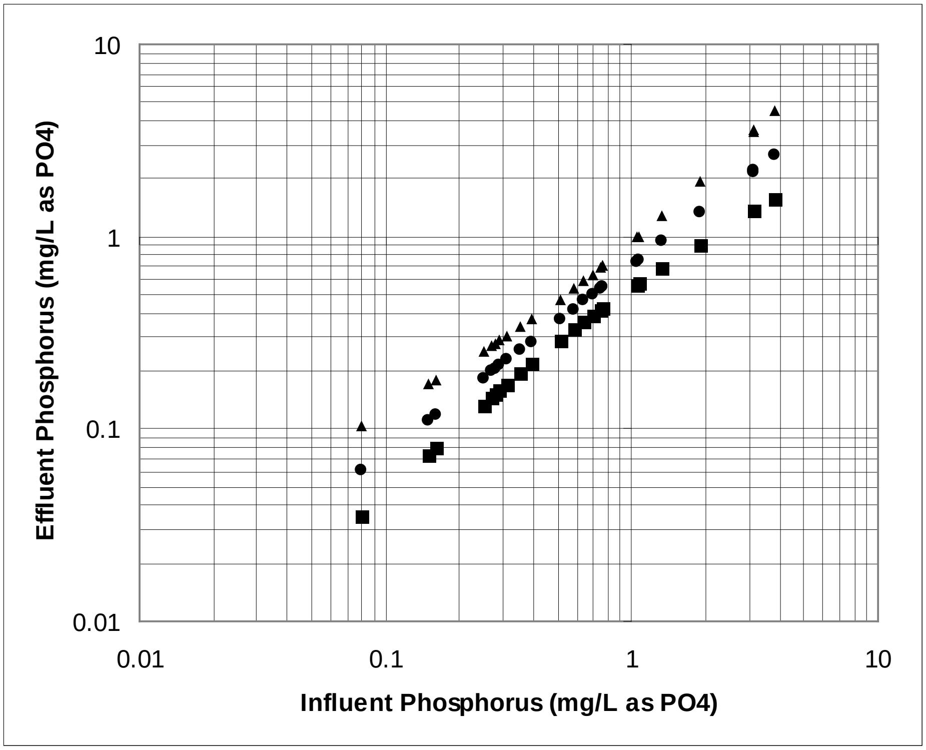
	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	-0.145	0.05746	-2.53	0.0189	-0.264	-0.02644
Influent Phosphates	0.9792	0.119	8.25	3.56E-08	0.733	1.23



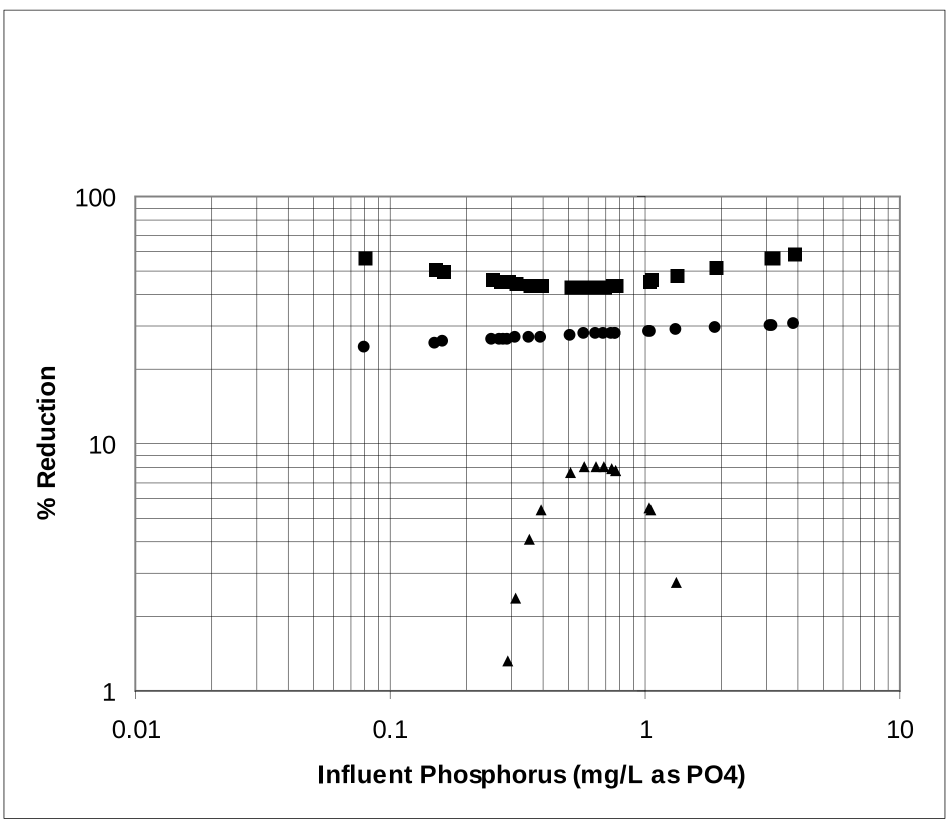
Fitted equation and data points for influent and effluent phosphorus.



Residual analyses of fitted equation for phosphorus influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

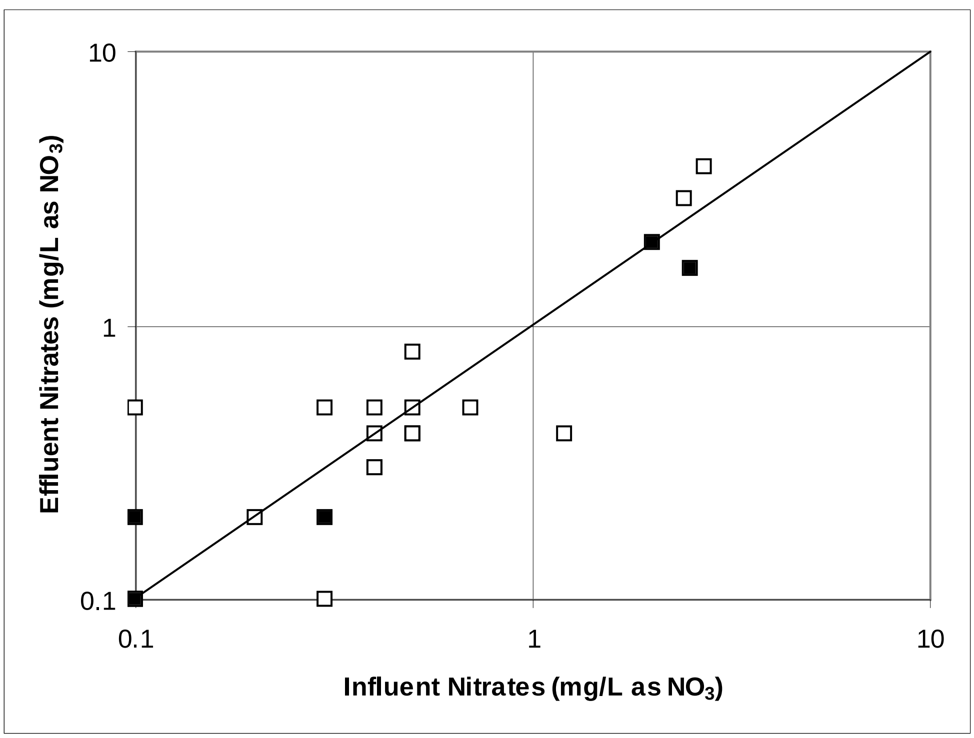


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

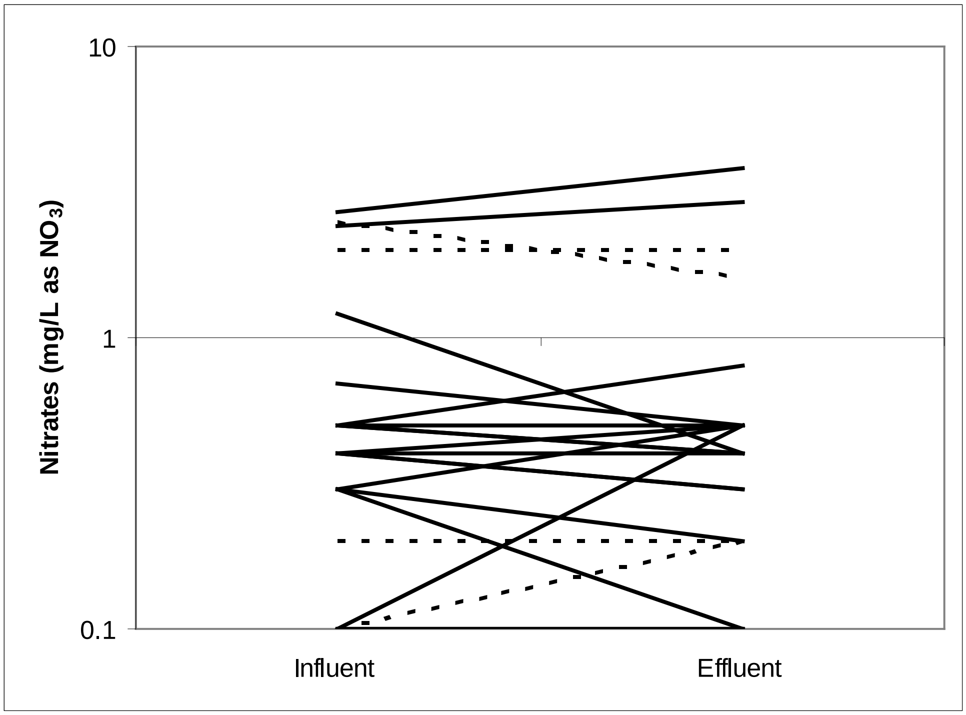
Nitrates

Observed Nitrates Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)
1-1	0.5	0.5
2-1	0.4	0.4
2-2	0.5	0.4
3-1	0.3	0.5
3-2	0.4	0.5
4-1	0.3	0.2
4-2	0.2	0.2
5-1	2.7	3.8
5-2	2.5	1.6
6-1	0.5	0.8
6-2	0.4	0.3
6-3	0.3	0.1
6-4	0.1	0.1
6-5	0.1	0.1
7-1	0.5	0.4
7-2	0.1	0.1
7-3	0.1	0.2
7-4	0	0
7-5	0.1	0.5
7-6	0.4	0.3
8-1	2.4	2.9
8-2	2	2
9-1	0.7	0.5
10-1	1.2	0.4



Scatterplot of observed influent and effluent nitrates concentrations (filled symbols are events that had minor filter bypasses).



Paired influent and effluent nitrates concentrations.

Fitted Equation:

Effluent nitrates, log mg/L = 0.910 * (Influent nitrates, log mg/L)

Regression Statistics

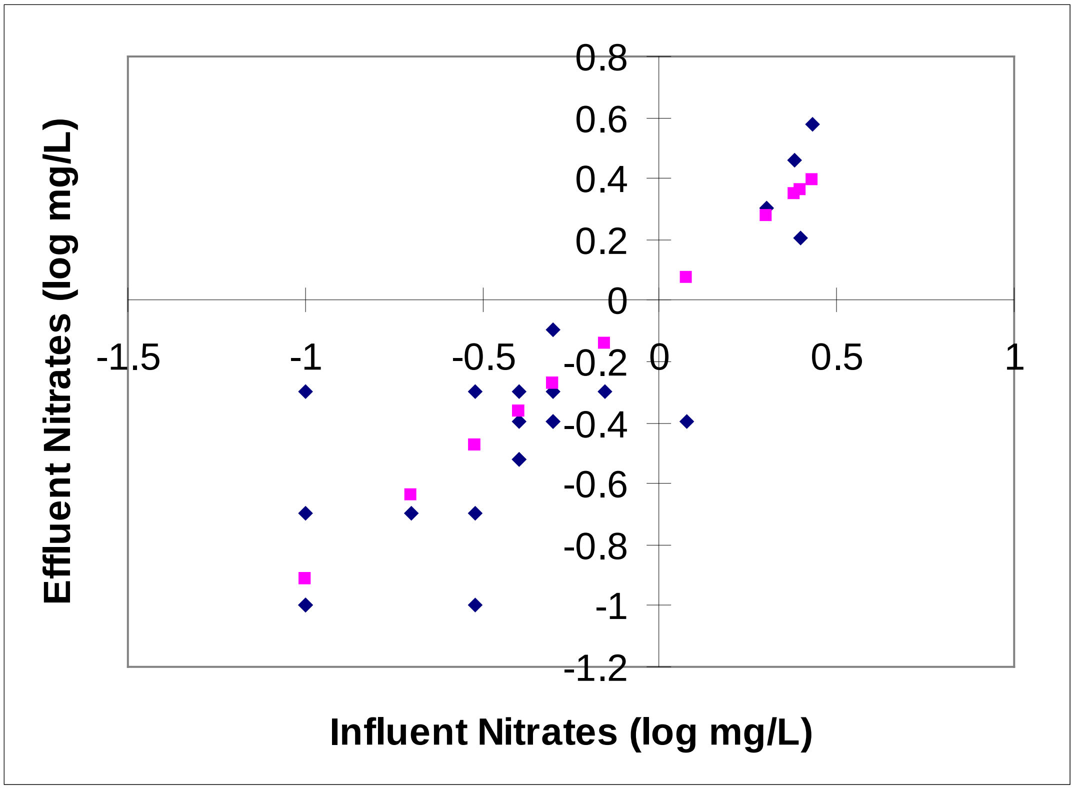
Multiple R	0.917
R Square	0.841
Adjusted R Square	0.795
Standard Error	0.237
Observations	23

ANOVA

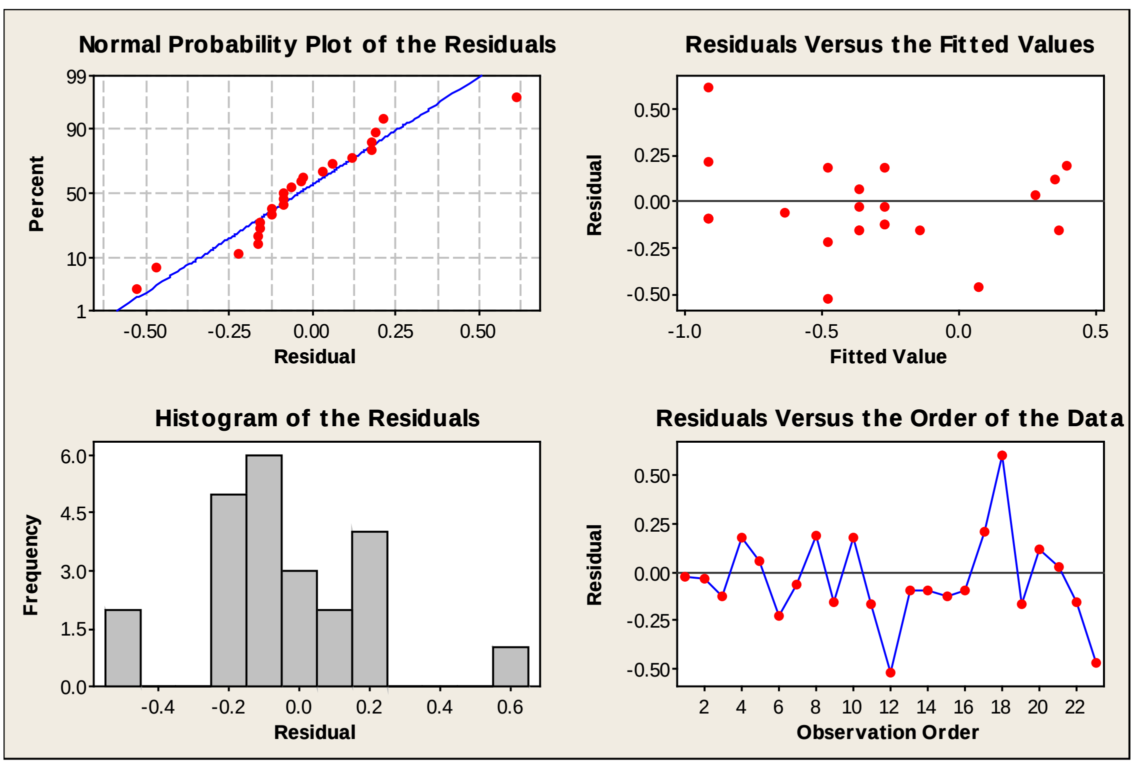
	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	6.55	6.55	116	5.07E-10
Residual	22	1.24	0.0563		
Total	23	7.79			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Influent Nitrates*	0.910	0.0844	10.8	3.005E-10	0.735	1.085

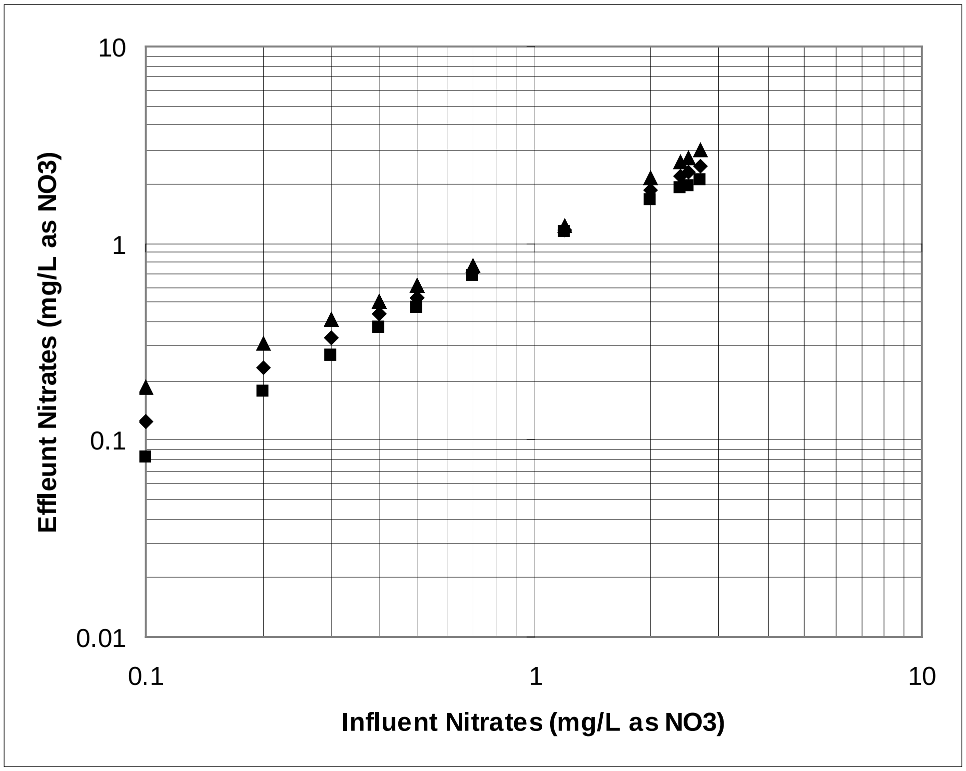
* the intercept term was determined to be not significant



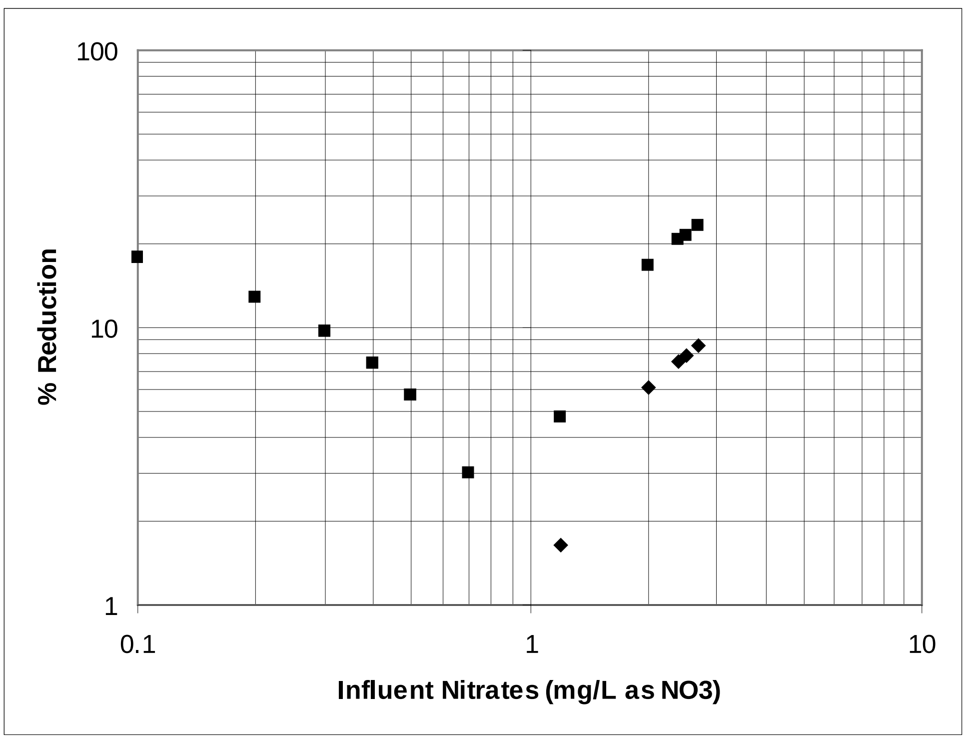
Fitted equation and data points for influent and effluent nitrates.



Residual analyses of fitted equation for nitrates influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

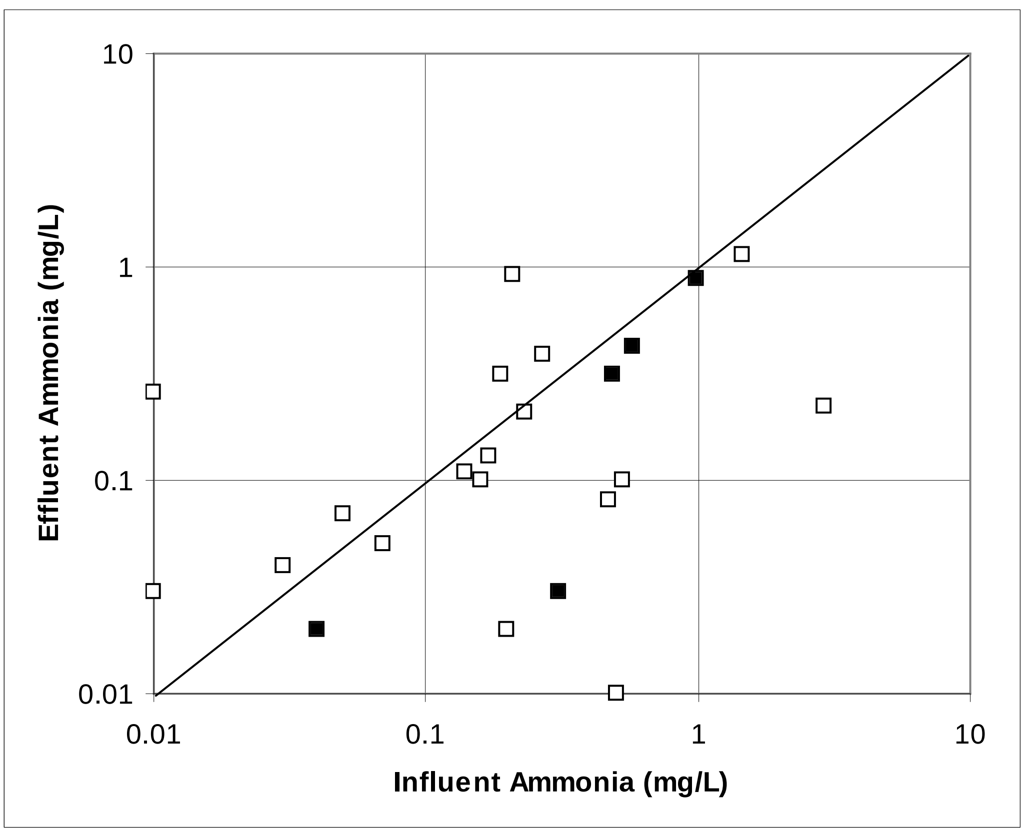


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

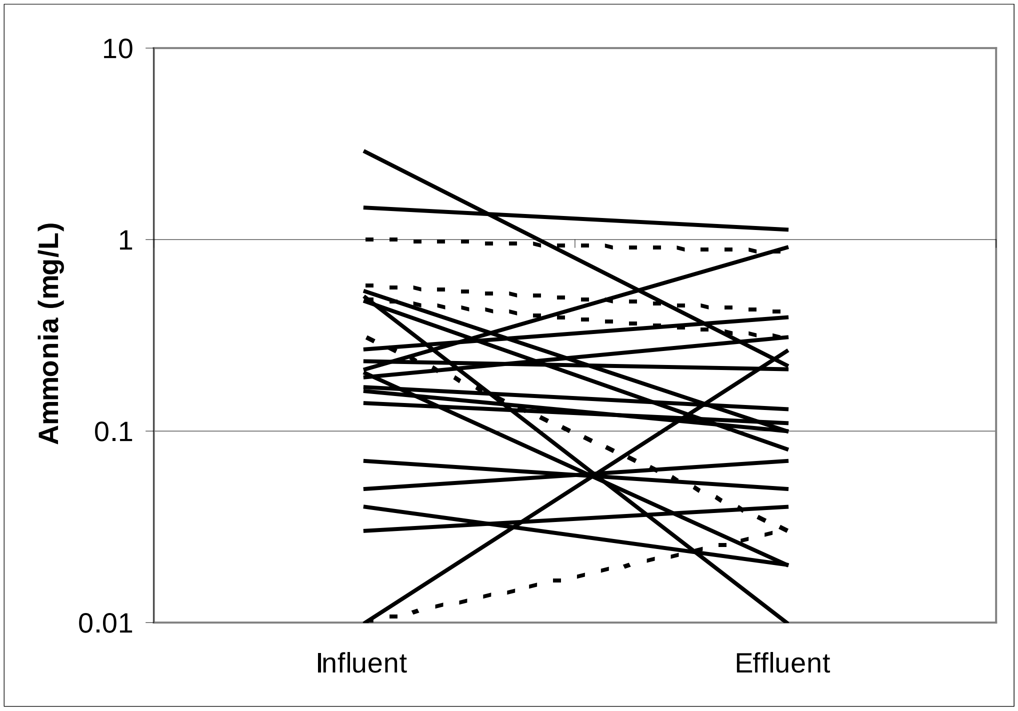
Ammonia

Observed Ammonia Concentrations

Sample Number	Influent (mg/L)	Effluent (mg/L)
1-1	0.21	0.91
2-1	0.23	0.21
2-2	0.27	0.39
3-1	0.16	0.1
3-2	0.5	0.01
4-1	0.04	0.02
4-2	0.01	0.03
5-1	0.19	0.31
5-2	0.31	0.03
6-1	0.17	0.13
6-2	0.14	0.11
6-3	0.07	0.05
6-4	0.05	0.07
6-5	0.03	0.04
7-1	1.47	1.14
7-2	0.58	0.42
7-3	0.49	0.31
7-4	0.47	0.08
7-5	0.53	0.1
7-6	0	0.02
8-1	2.9	0.22
8-2	0.99	0.87
9-1	0.2	0.02
10-1	0.01	0.26



Scatterplot of observed influent and effluent ammonia concentrations (filled symbols are events that had minor filter bypasses).



Paired influent and effluent ammonia concentrations.

Fitted Equation:

Effluent ammonia, log mg/L = -0.647 + 0.373 * (Influent ammonia, log mg/L)

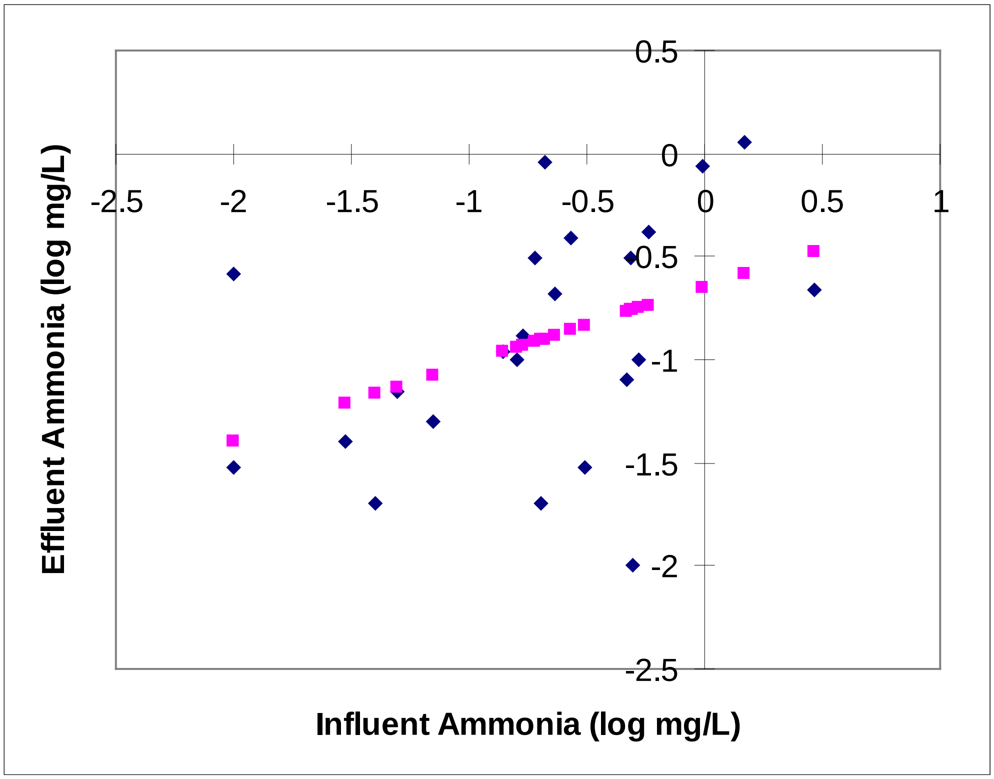
Regression Statistics

Multiple R	0.409
R Square	0.167
Adjusted R Square	0.128
Standard Error	0.534
Observations	23

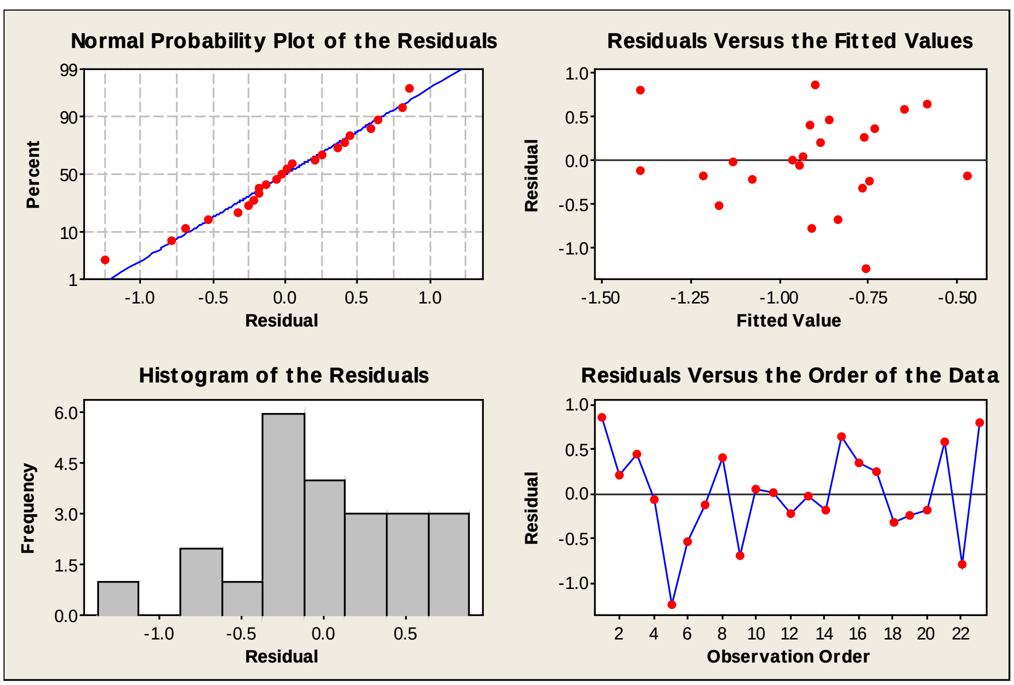
ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	1.202	1.20	4.22	0.0526
Residual	21	5.98	0.285		
Total	22	7.18			

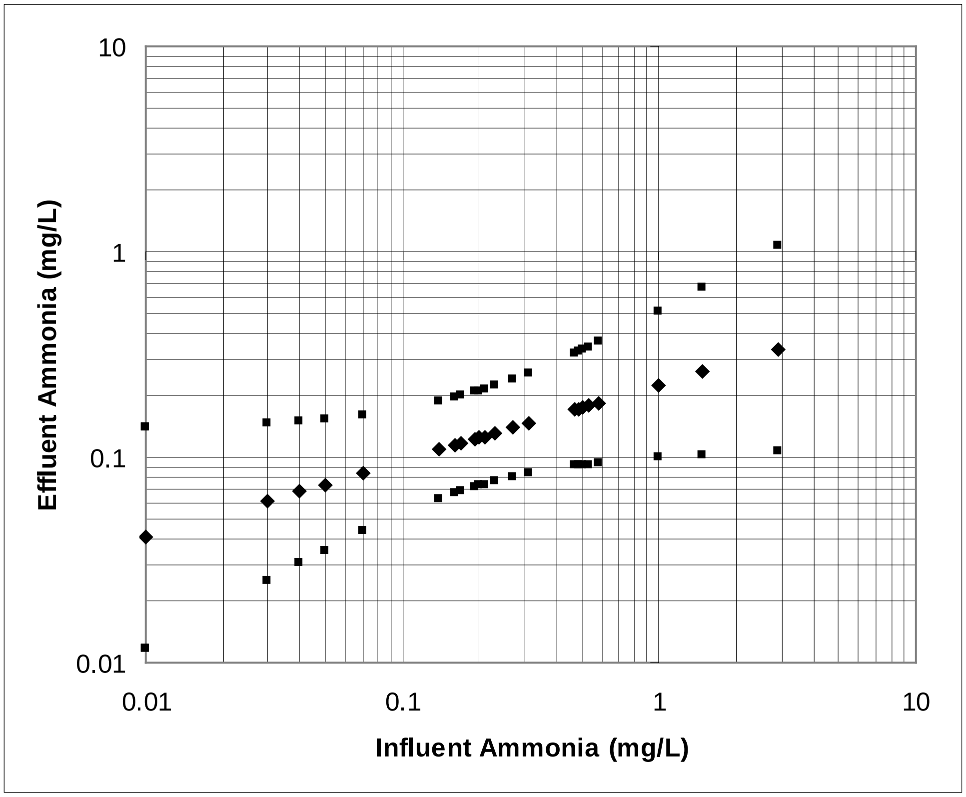
	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	-0.647	0.171	-3.78	0.00109	-1.00	-0.291
Influent Ammonia	0.373	0.182	2.05	0.0526	-0.00456	0.751



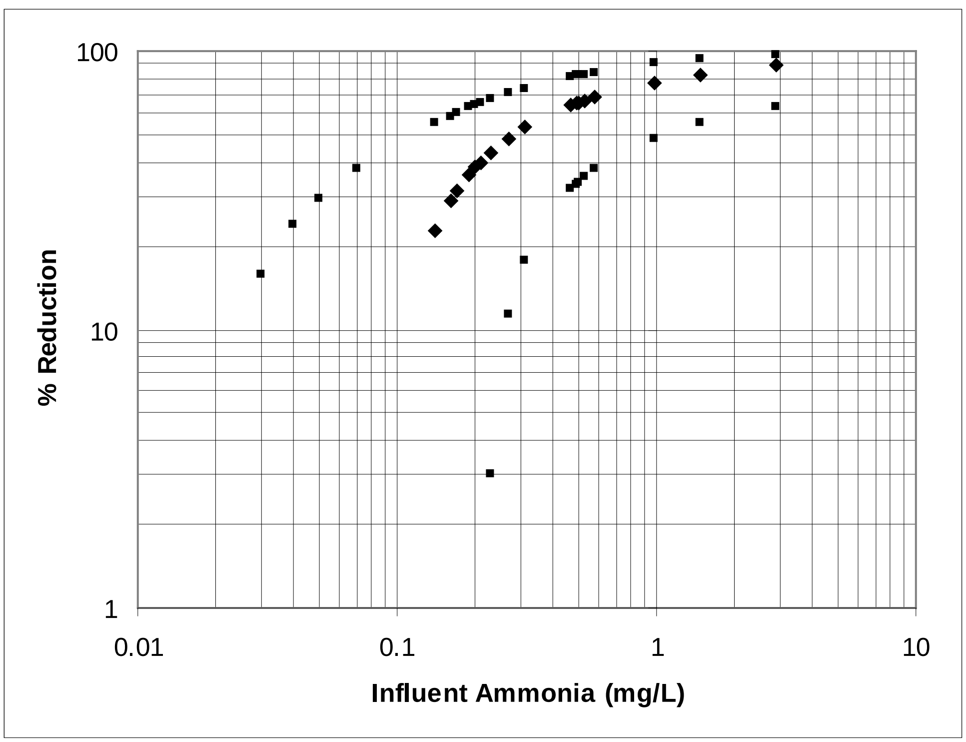
Fitted equation and data points for influent and effluent ammonia.



Residual analyses of fitted equation for ammonia influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

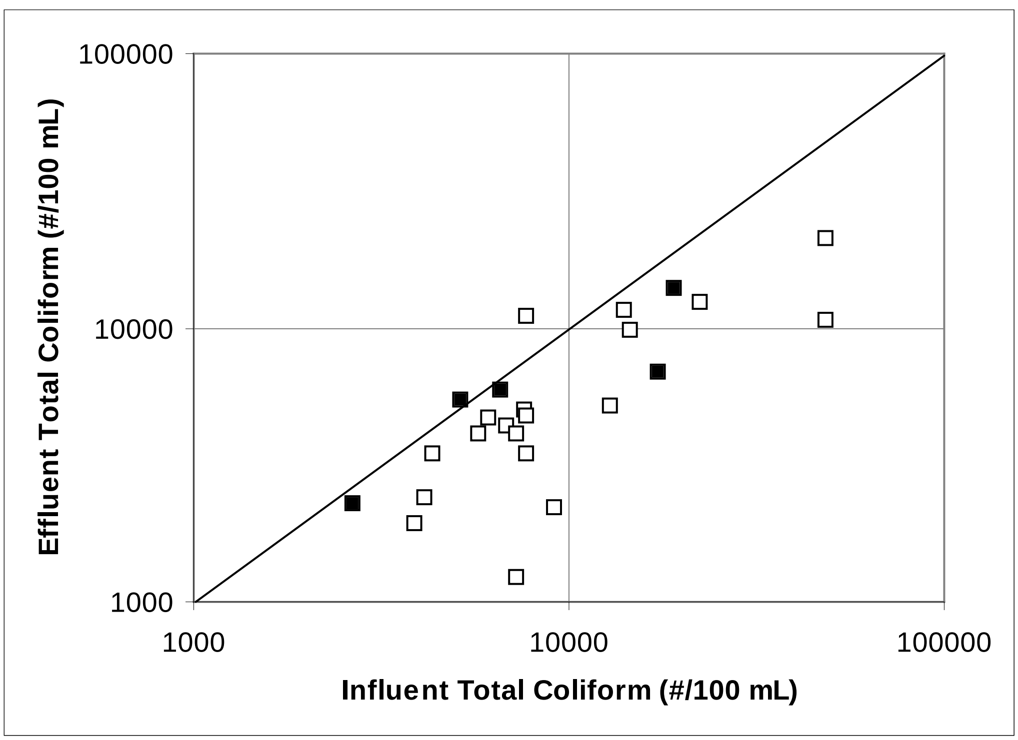


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

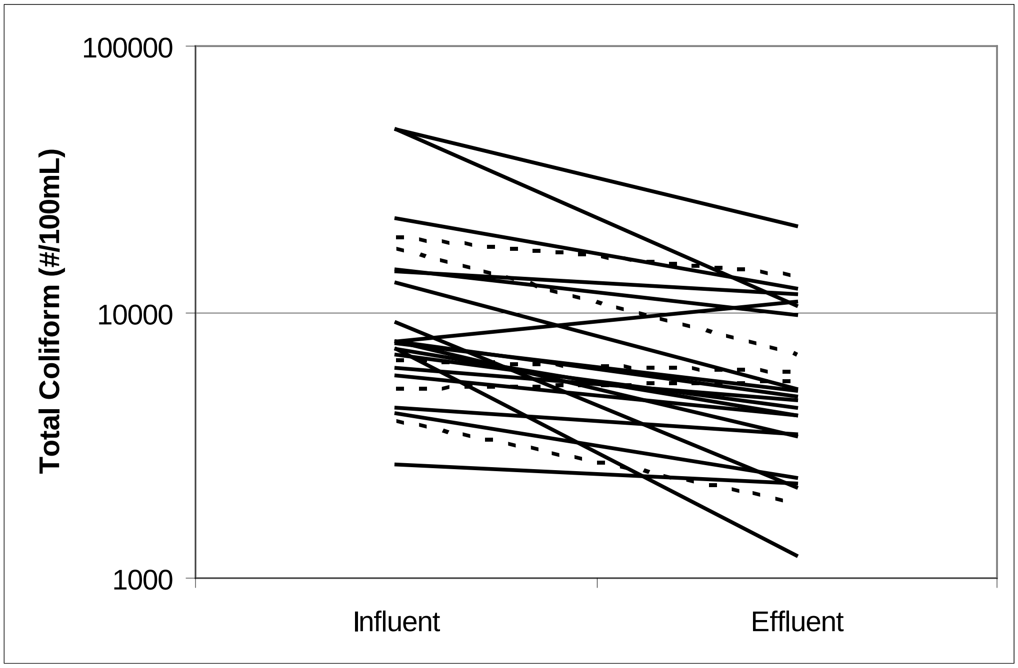
Total Coliforms

Observed Total Coliforms Concentrations

Sample Number	Influent (#/100 mL)	Effluent (#/100 mL)
1-1	48384	20924.8
2-1	7754	10950
2-2	22397	12262
3-1	7701	4786
3-2	9208	2209
4-1	2656	2277
4-2	3877	1937
5-1	14540	9768
5-2	6628	5944
6-1	4352	3448
6-2	6131	4701
6-3	5794	4106
6-4	4160	2382
6-5	6867	4386
6-5	7270	1220
7-1	14136	11620
7-2	17328.7	6910
7-3	5172	5468
7-4	12996.5	5172
7-6	7270	4106
8-1	48394	10670
8-2	19212	13820
9-1	7622	5026
10-1	7770	3440



Scatterplot of observed influent and effluent total coliforms concentrations (filled symbols are events that had minor filter bypasses).



Paired influent and effluent total coliforms concentrations.

Fitted Equation:

Effluent total coliforms, log #/100 mL = 0.937 * (Influent total coliforms, log #/100 mL)

Regression Statistics

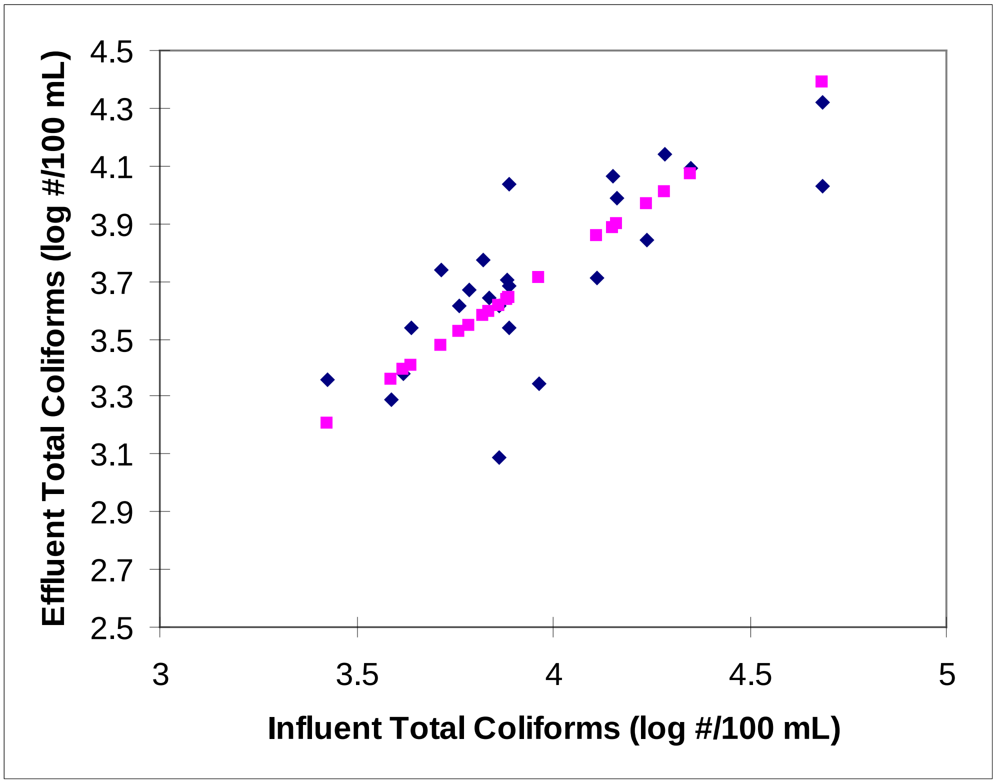
Multiple R	0.999
R Square	0.997
Adjusted R Square	0.953
Standard Error	0.208
Observations	24

ANOVA

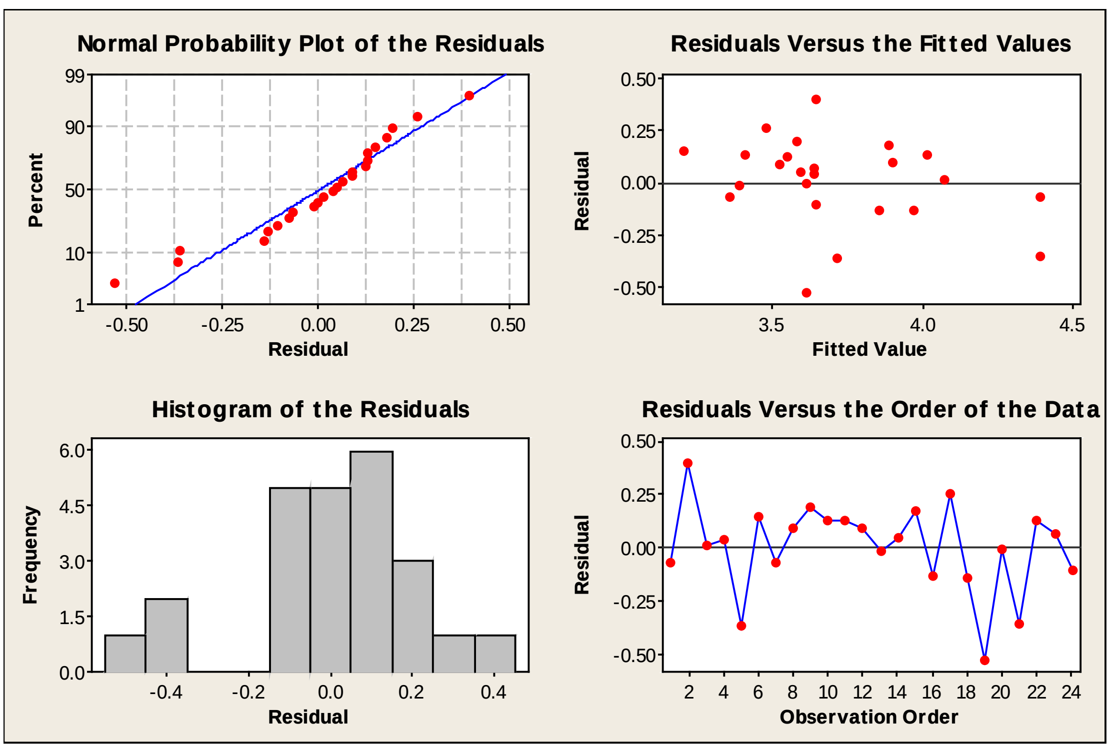
	df	SS	MS	F	Significance F
Regression	1	333	333	7688	1.72E-29
Residual	23	0.995	0.0433		
Total	24	334			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Influent Total Coliform*	0.937	0.0107	87.7	1.450E-30	0.914	0.965

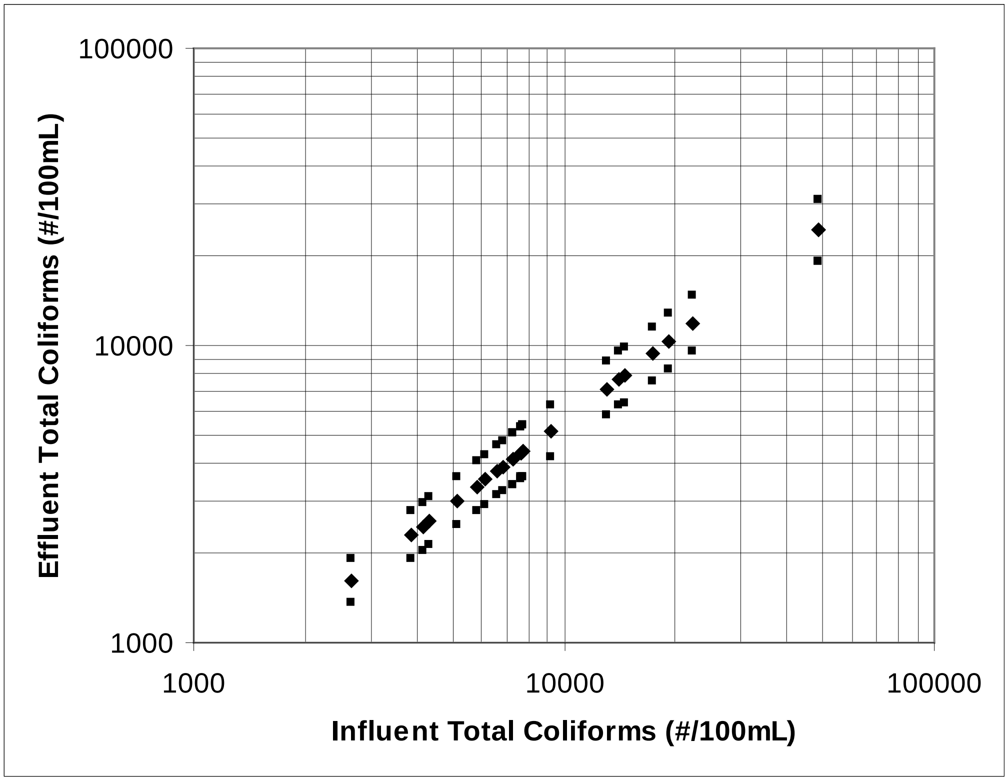
* the intercept term was determined to be not significant



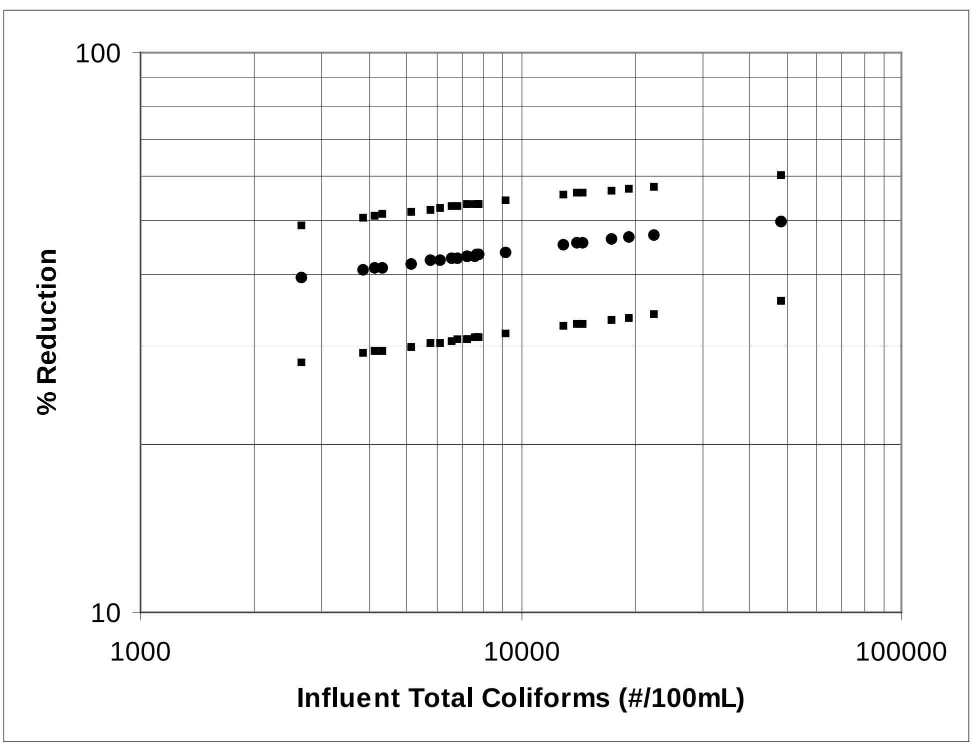
Fitted equation and data points for influent and effluent total coliforms.



Residual analyses of fitted equation for total coliforms influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

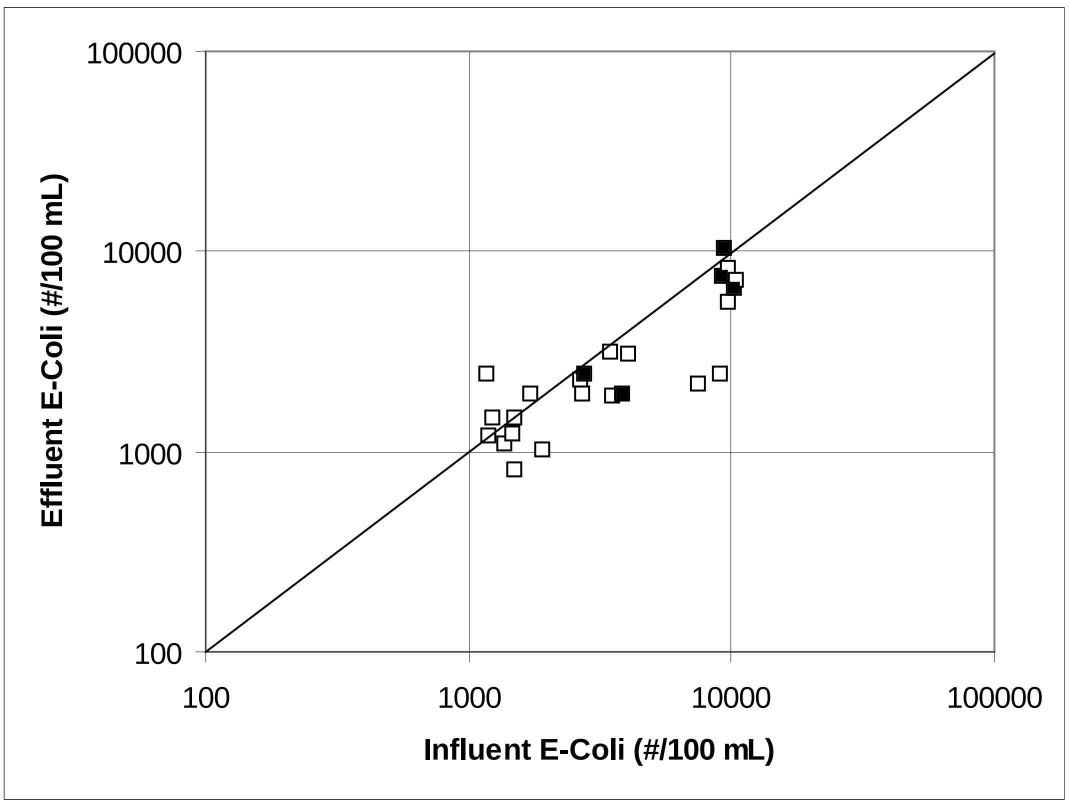


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

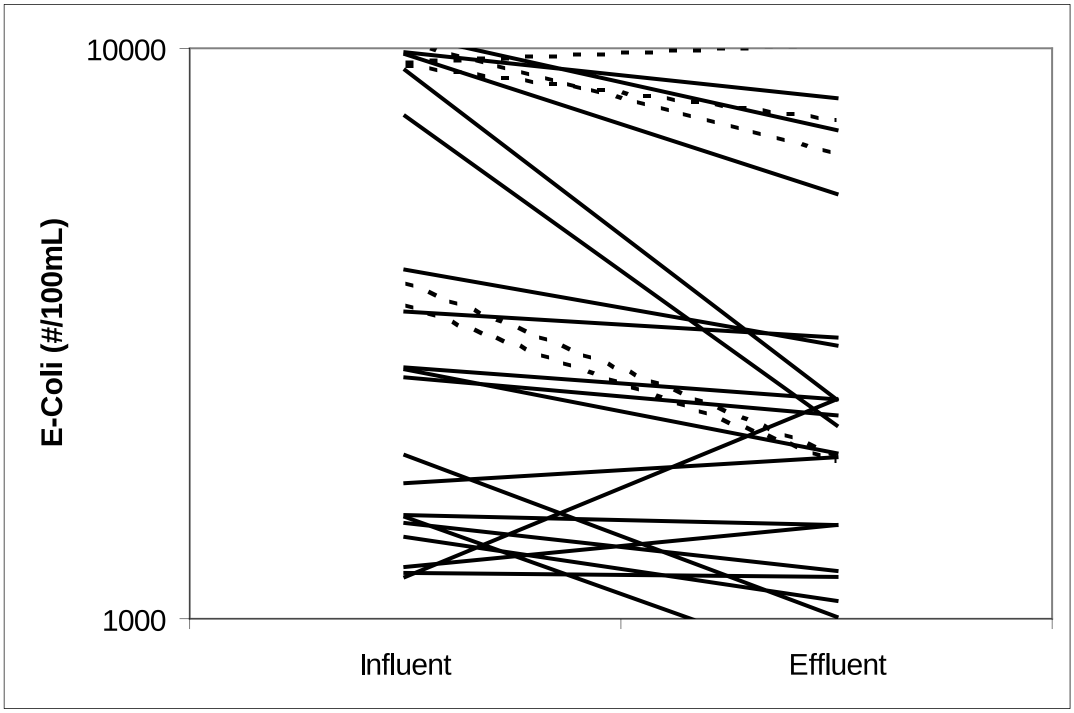
E. Coli.

Observed E. Coli. Concentrations

Sample Number	Influent (#/100 mL)	Effluent (#/100 mL)
1-1	1466	1218
2-1	1202	1182
2-2	1390	1076
3-1	2656	2277
3-2	1508	808
3-2	3877	1937
4-1	2750	2430
4-2	3540	1890
5-1	3458	3122
6-1	1236	1457
6-2	1187	2419
6-3	1935	1011
6-4	1725	1918
6-5	1515	1455
7-1	10560	7180
7-2	9300	7460
7-3	10240	6540
7-4	9740	5560
7-5	7580	2180
7-6	9140	2420
8-1	9814	8166
8-2	9442	10198
9-1	4084	3024
10-1	2745	1952



Scatterplot of observed influent and effluent E. Coli. concentrations (filled symbols are events that had minor filter bypasses).



Paired influent and effluent E. Coli. concentrations.

Fitted Equation:

Effluent E. coli., log #/100 mL = 0.745 + 0.751 * (Influent E. coli., log #/100 mL)

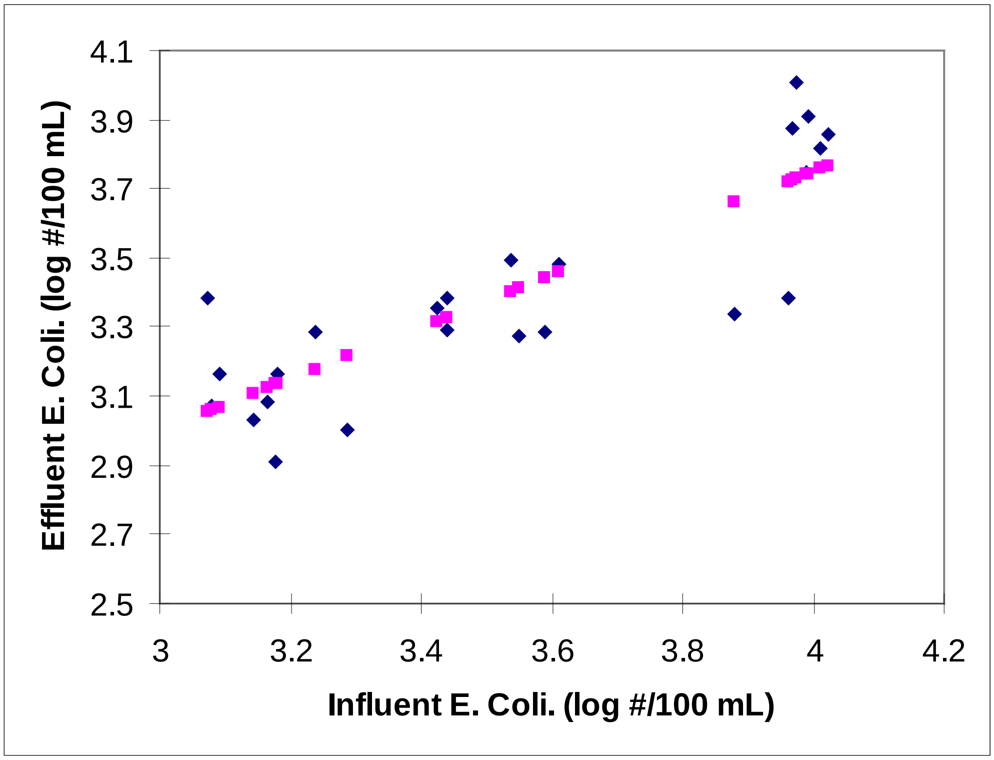
Regression Statistics

Multiple R	0.848
R Square	0.719
Adjusted R Square	0.706
Standard Error	0.171
Observations	24

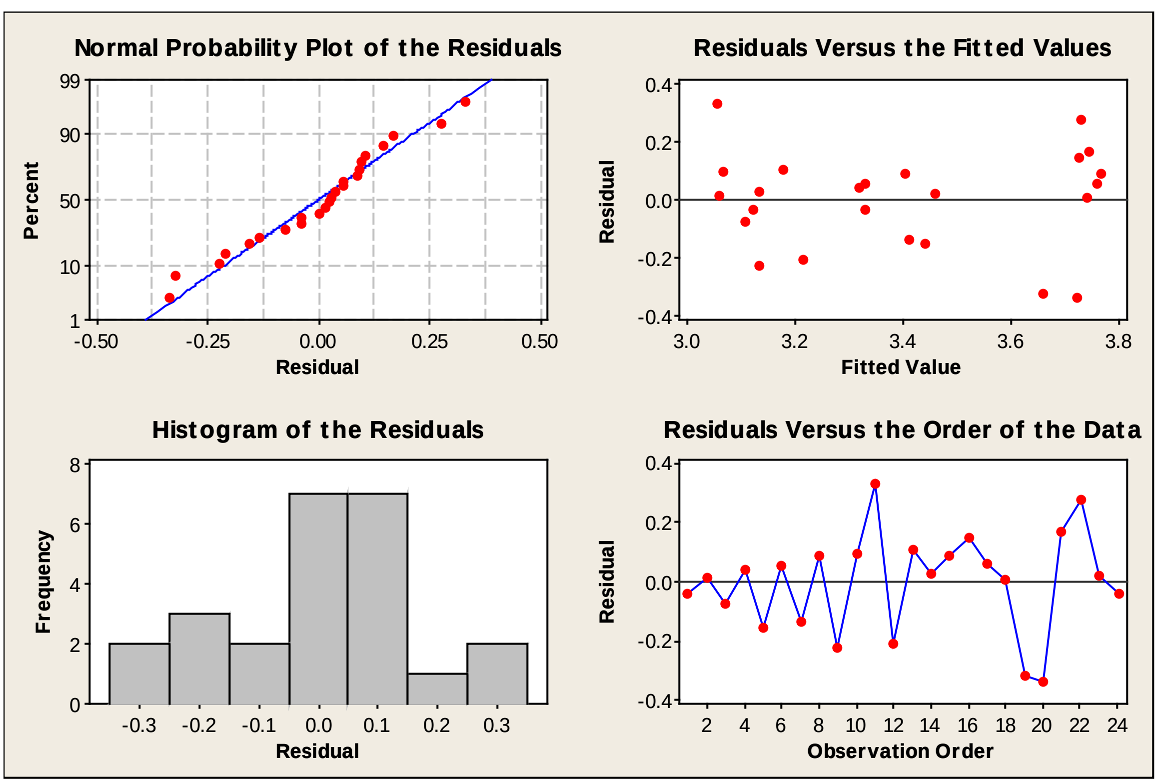
ANOVA

	df	SS	MS	F	Significance F
Regression	1	1.64	1.64	56.2	1.70E-07
Residual	22	0.642	0.0292		
Total	23	2.28			

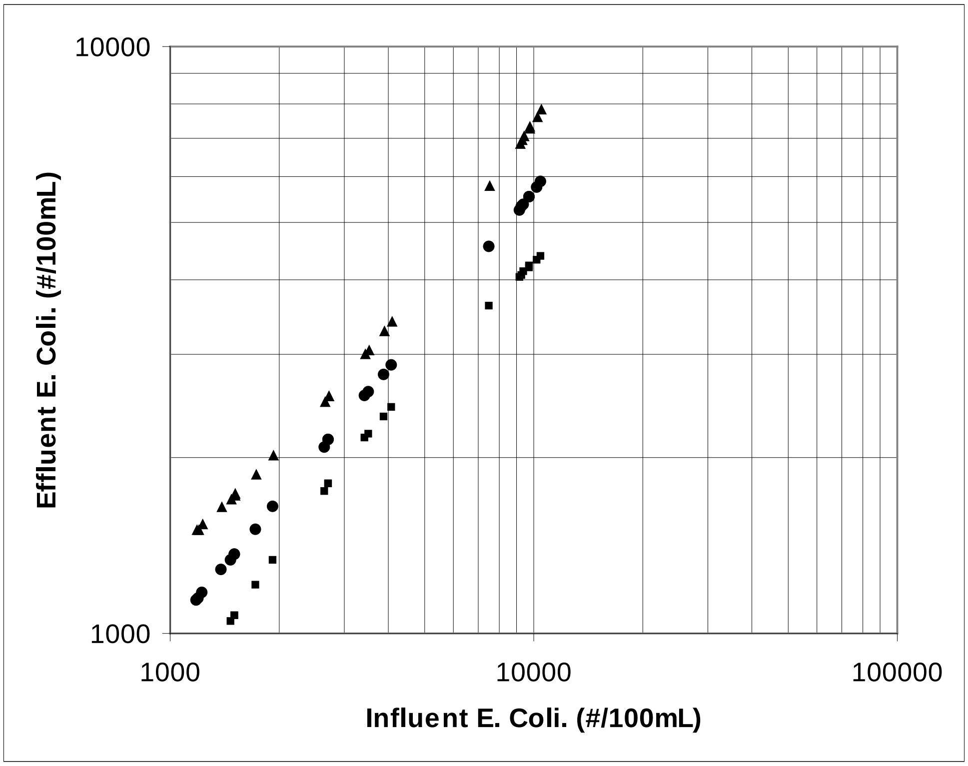
	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Intercept	0.745	0.356	2.09	0.0480	0.00718	1.48
Influent E-Coli	0.751	0.100	7.50	1.70E-07	0.543	0.959



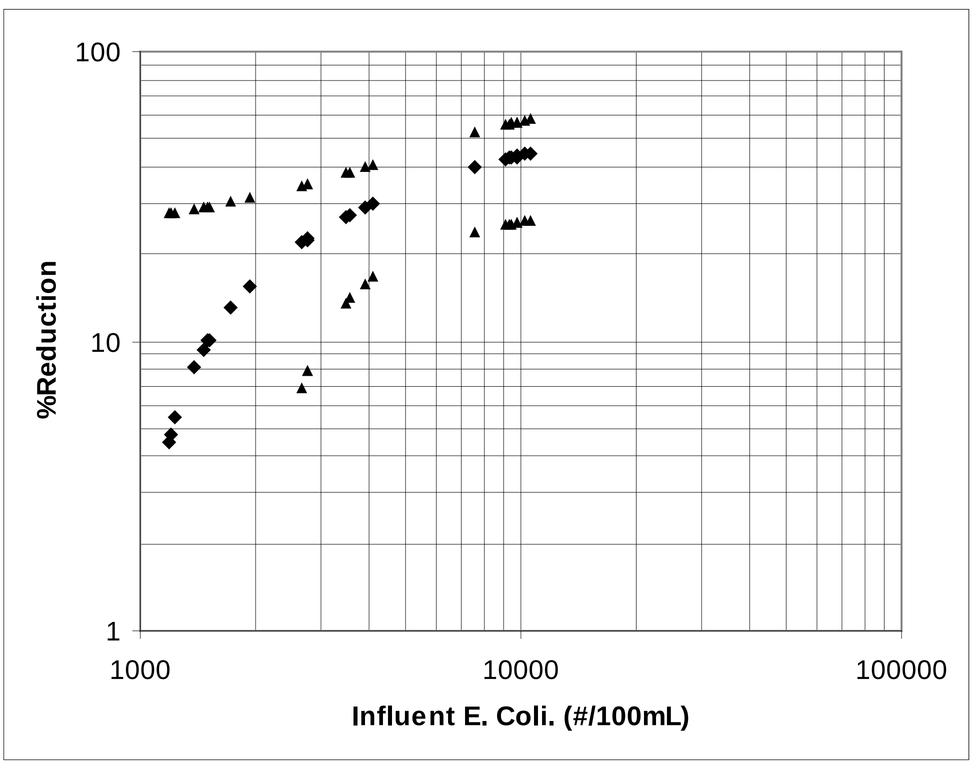
Fitted equation and data points for influent and effluent E. Coli.



Residual analyses of fitted equation for E. coli. influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

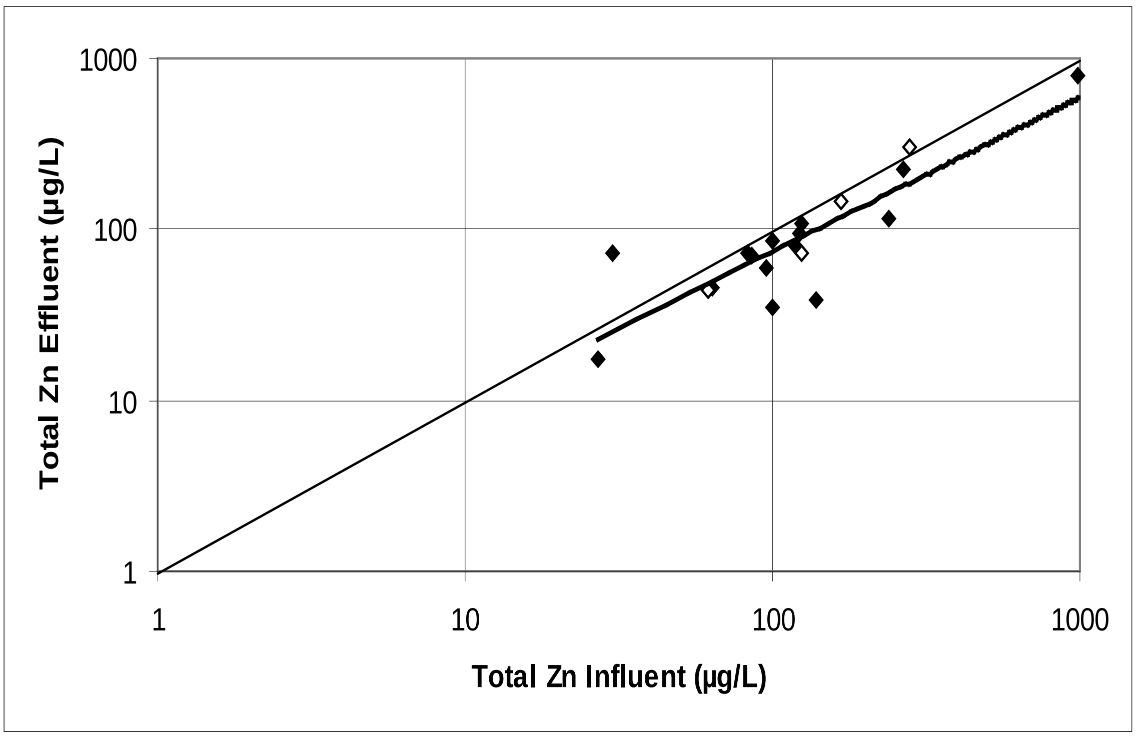


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

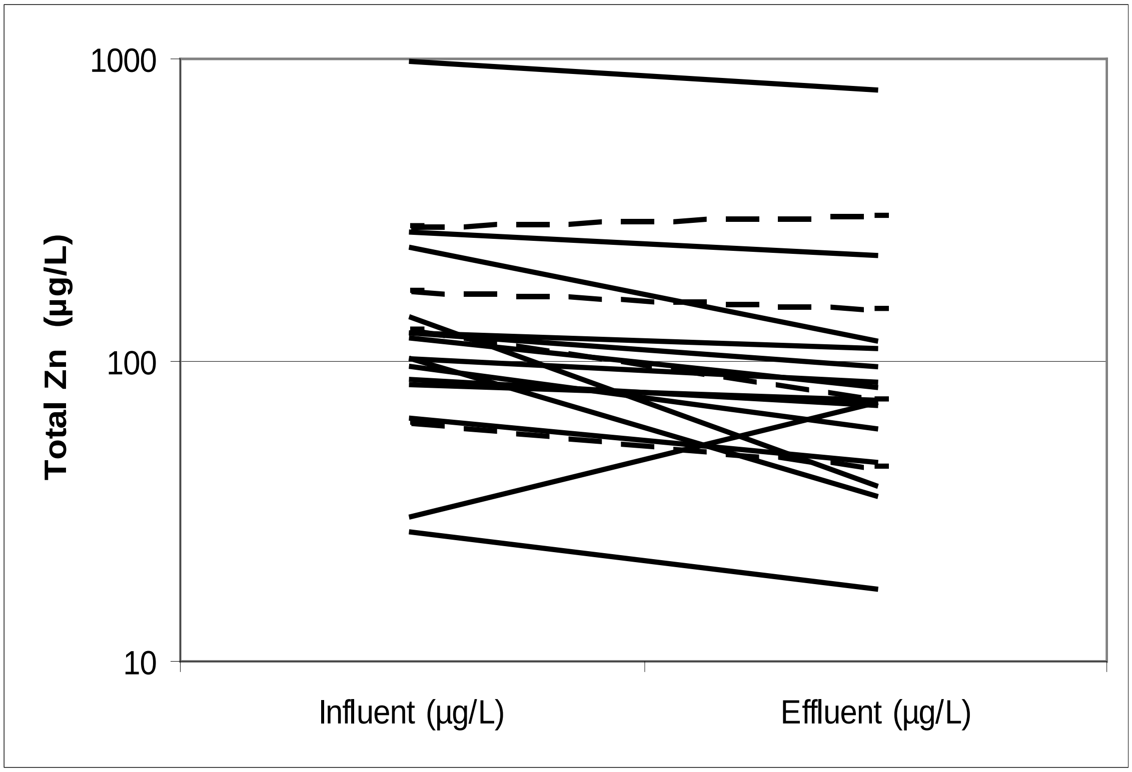
Total Zinc

Observed Total Zinc Concentrations

Sample Number	Influent (µg/L)	Effluent (µg/L)
1-1	118	81
2-1	123	95
5-1	267	221
5-2	168	147
6-1	101	85
6-2	86	70
6-3	83	73
6-4	95	60
6-5	100	35
7-1	124	109
7-2	125	73
7-3	62	44
7-4	27	17
7-5	30	72
7-6	64	46
8-1	990	785
8-2	277	298
9-1	139	38
10-1	237	115



Scatterplot of observed influent and effluent total Zn concentrations (unfilled symbols are events that had minor filter bypasses.



Paired influent and effluent total Zn concentrations (Sign test Significant P-value = 0.0007).

Fitted Equation:

Effluent Total Zn, log µg/L = 0.933 * (Influent Total Zn, log µg/L)

Regression Statistics on Observed Influent vs. Effluent Total Zn, log µg/L

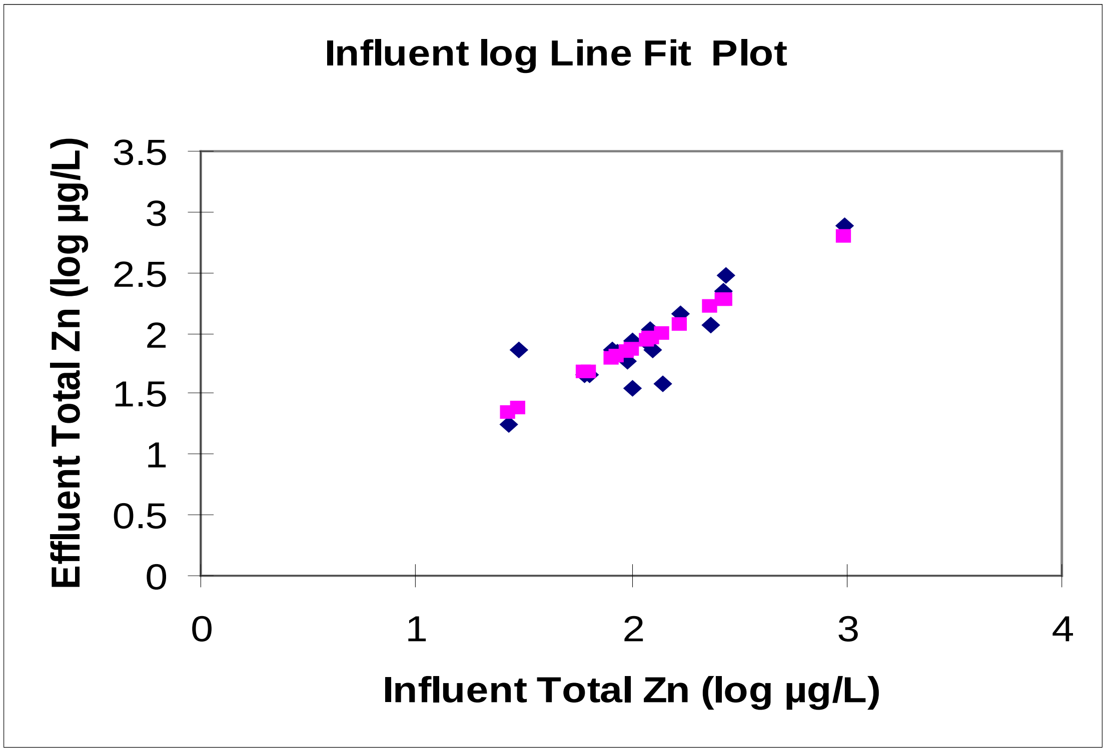
Multiple R	0.860
R Square	0.740
Adjusted R Square	0.684
Standard Error	0.187
Observations	19

ANOVA

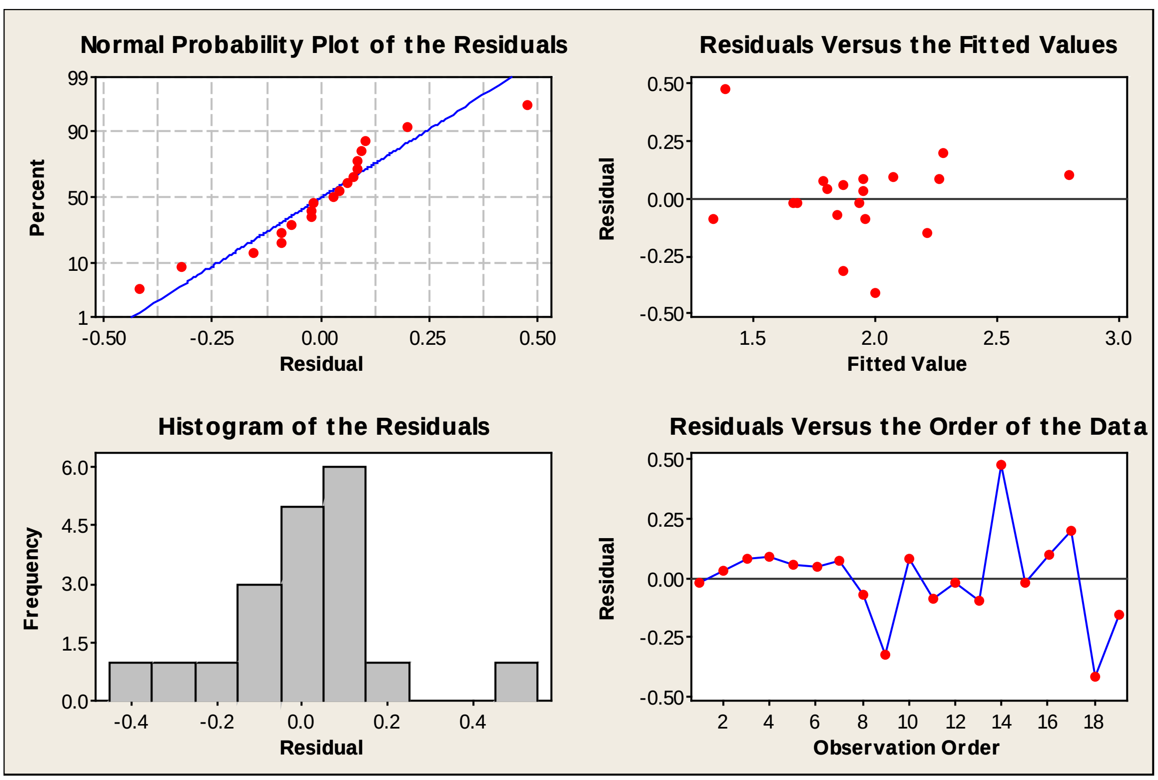
	df	SS	MS	F	Significance F
Regression	1	1.80	1.80	51.2	1.61E-06
Residual	18	0.632	0.0351		
Total	19	2.43			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Influent Total Zn	0.933	0.0205	45.5	4.93E-20	0.889	0.976

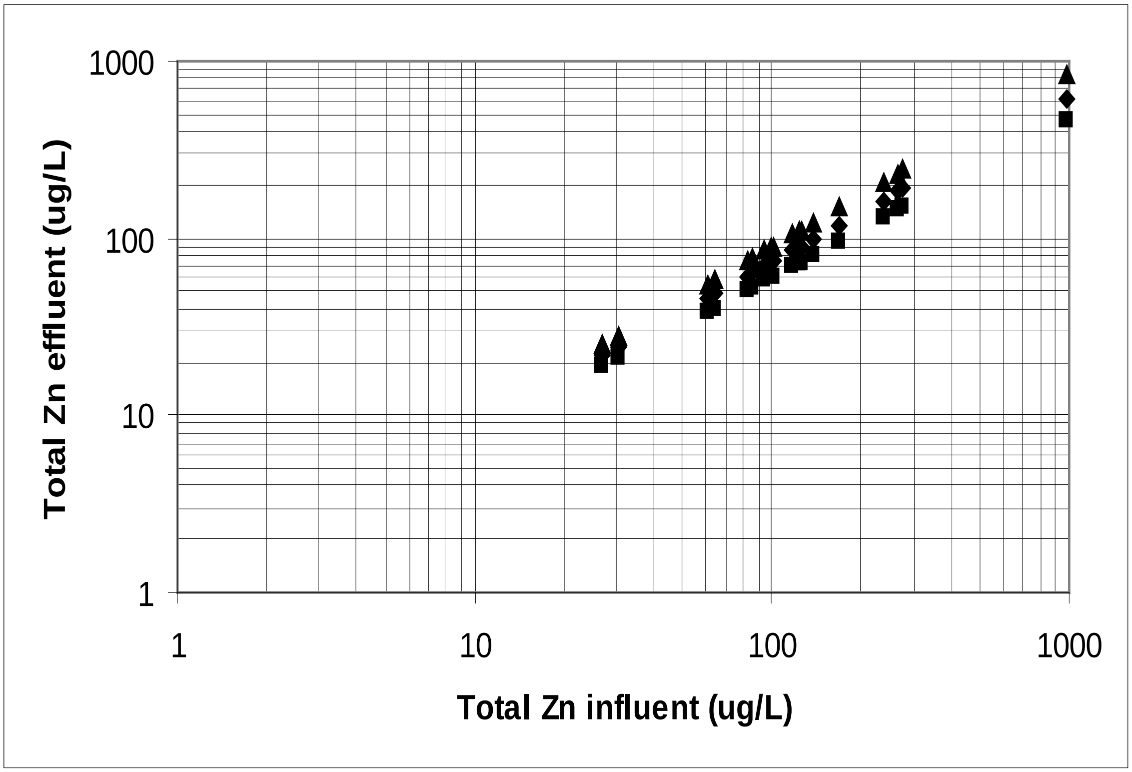
* the intercept term was determined to be not significant



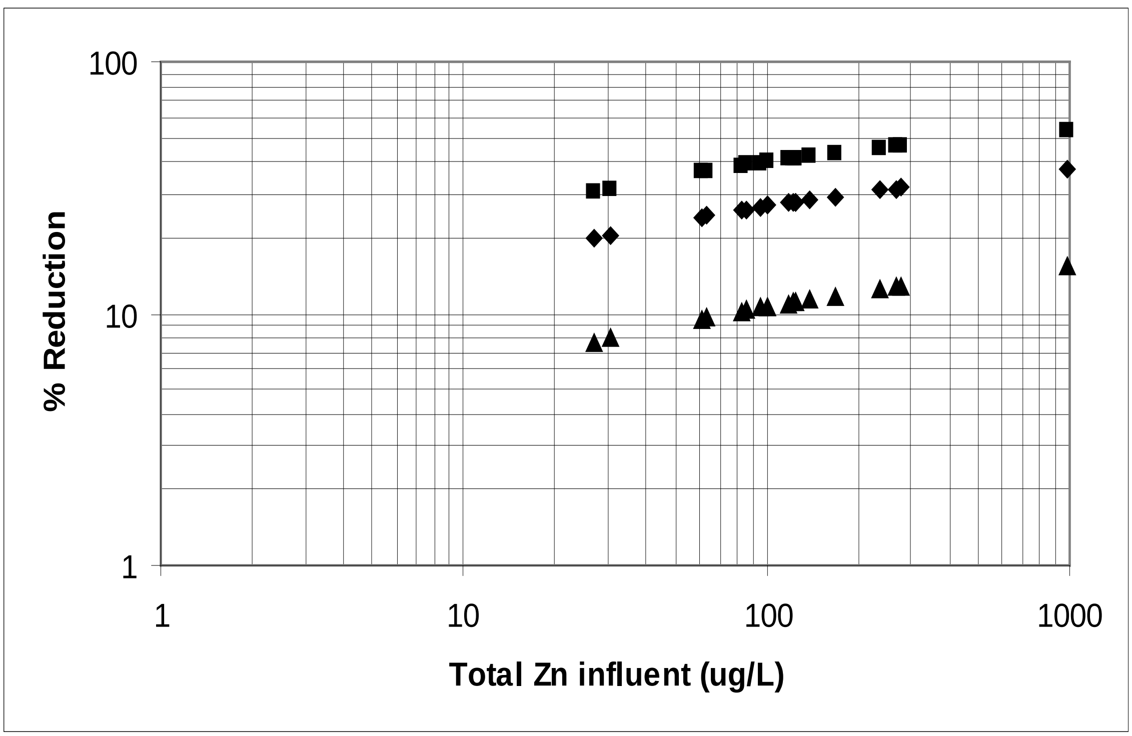
Fitted equation and data points for influent and effluent Total Zn.



Residual analyses of fitted equation for Total Zn influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

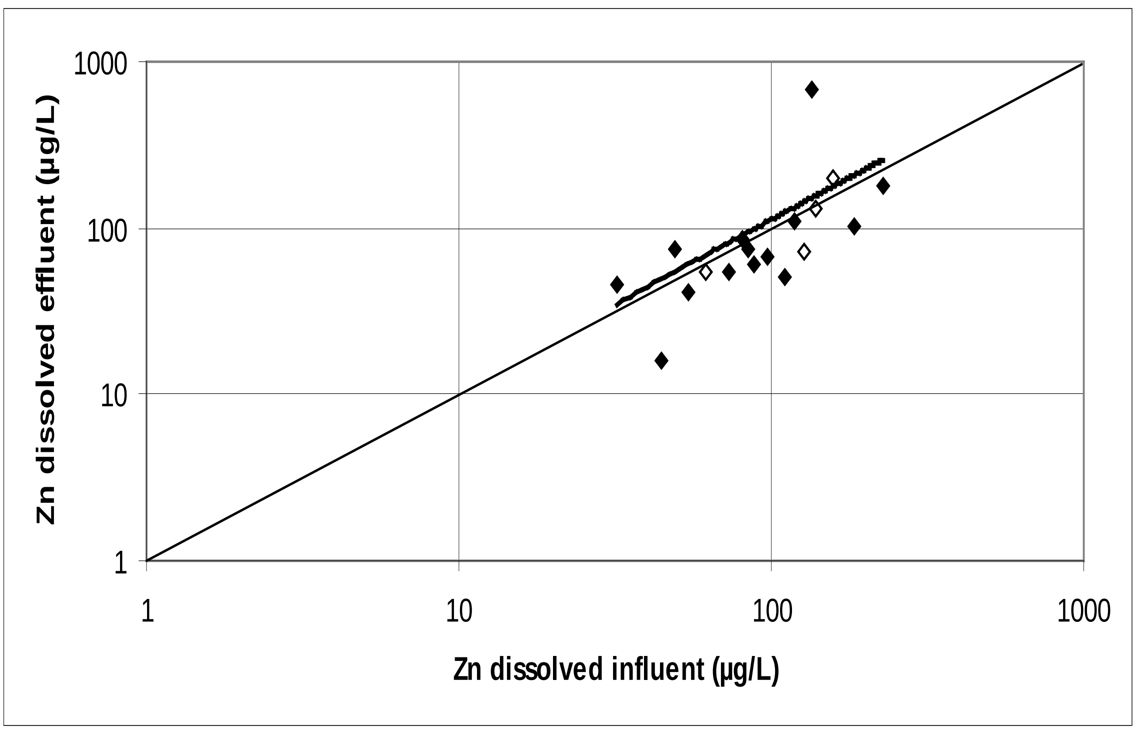


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

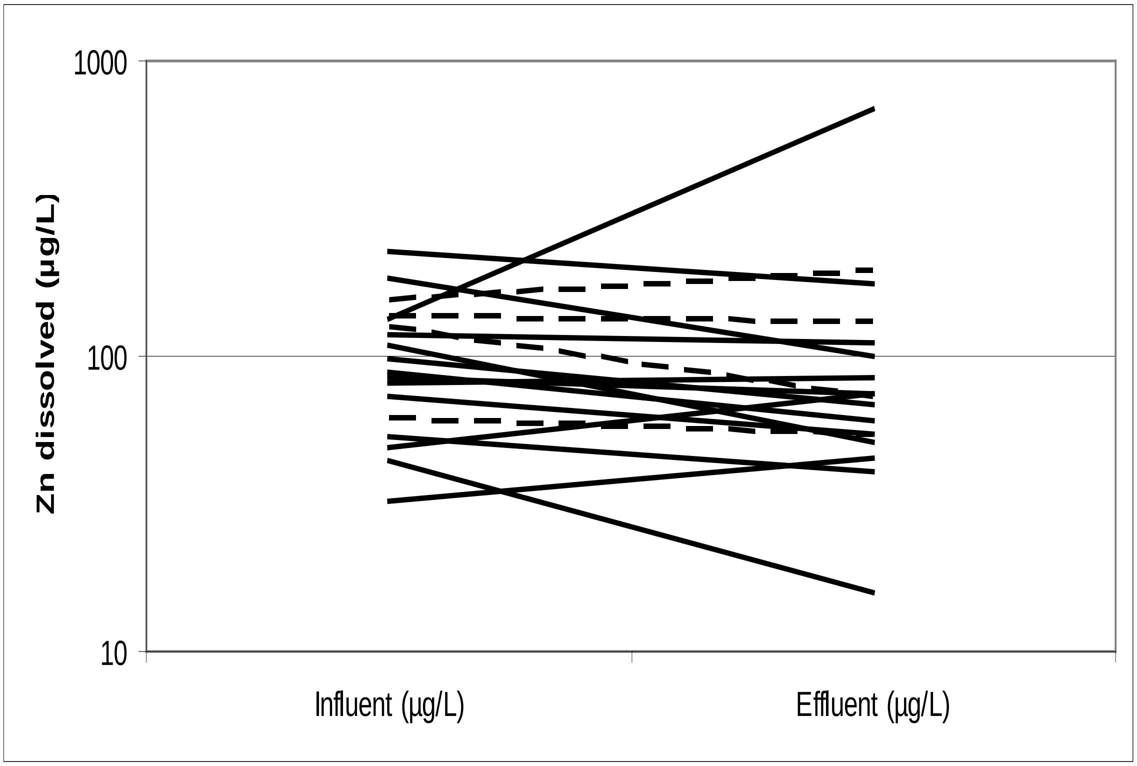
Dissolved Zinc

Observed Dissolved Zinc Concentrations

Sample Number	Influent (µg/L)	Effluent (µg/L)
1-1	84.8	73.9
5-1	227.4	177.8
5-2	137.9	132.0
6-1	97.3	68.3
6-2	109.8	50.9
6-3	80.3	84.6
6-4	87.5	60.6
6-5	73.0	54.8
7-1	118.2	110.8
7-2	127.0	72.8
7-3	61.8	54.3
7-4	53.9	40.7
7-5	49.4	74.8
7-6	44.2	15.8
8-1	135.4	679.6
8-2	156.5	196.57
9-1	32.0	45.1
10-1	183.7	100.7



Scatterplot of observed influent and effluent dissolved Zn concentrations.



Paired influent and effluent dissolved Zn concentrations (Sign test Significant P-value = 0.96).

Fitted Equation:

Effluent Dissolved Zn, log µg/L = 0.9734 * (Influent Dissolved Zn, log µg/L)

Regression Statistics on Observed Influent vs. Effluent Dissolved Zn, log µg/L

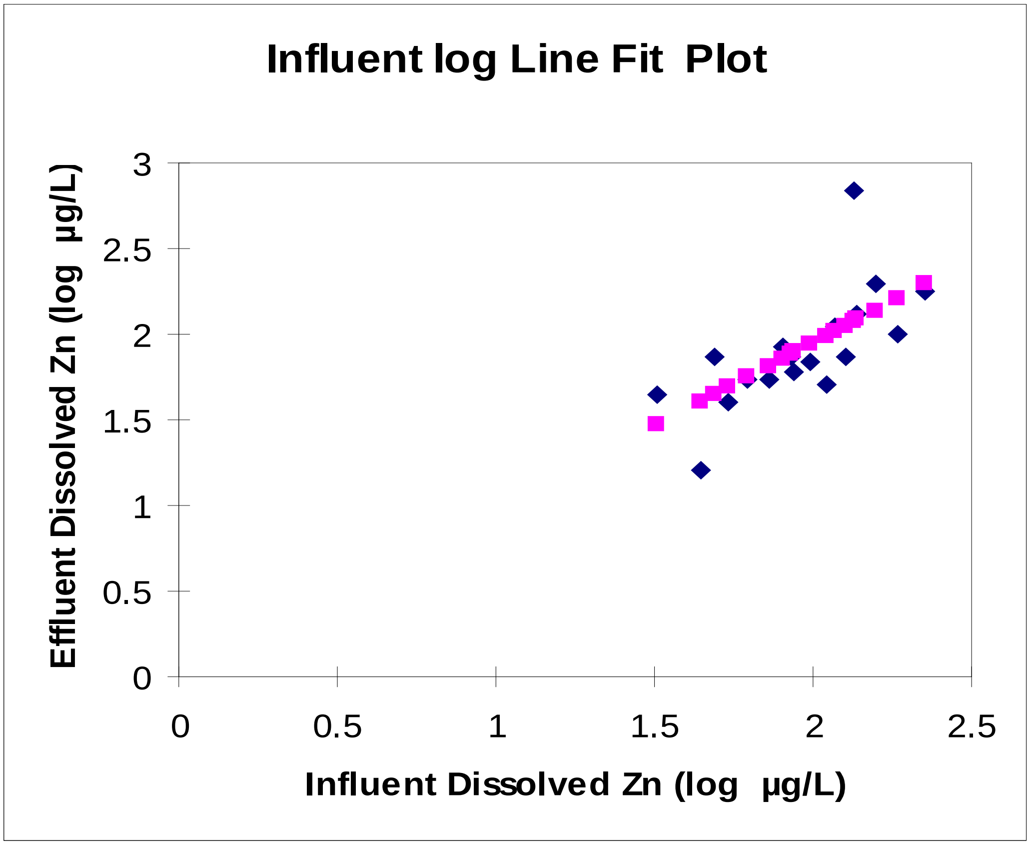
Multiple R	0.687
R Square	0.472
Adjusted R Square	0.413
Standard Error	0.248
Observations	18

ANOVA

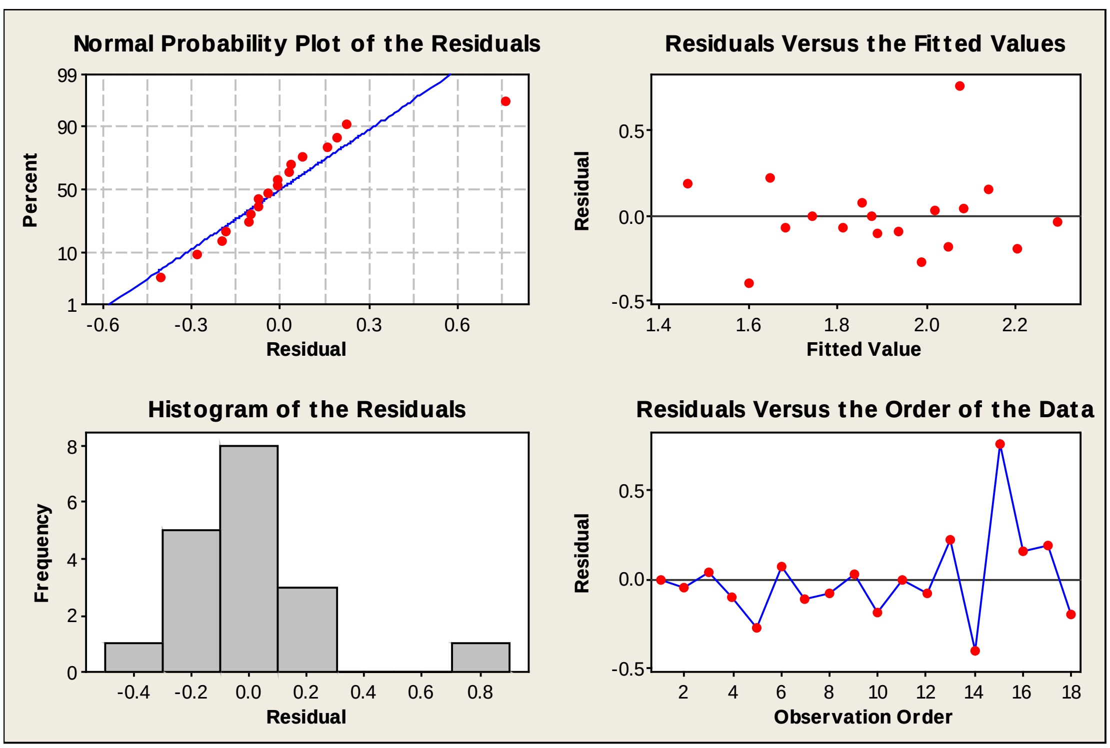
	df	SS	MS	F	Significance F
Regression	1	0.931	0.931	15.2	0.00129
Residual	17	1.04	0.061		
Total	18	1.97			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Influent Dissolved Zn	0.973	0.030	32.9	7.77E-17	0.911	1.04

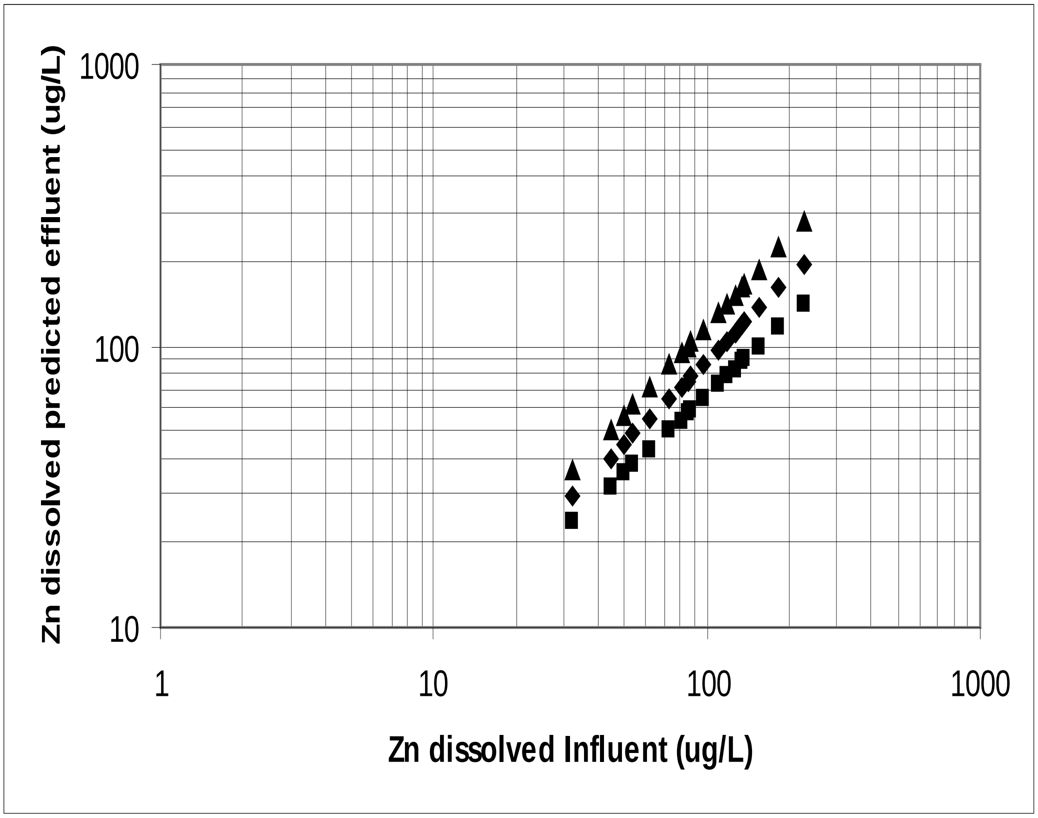
* the intercept term was determined to be not significant



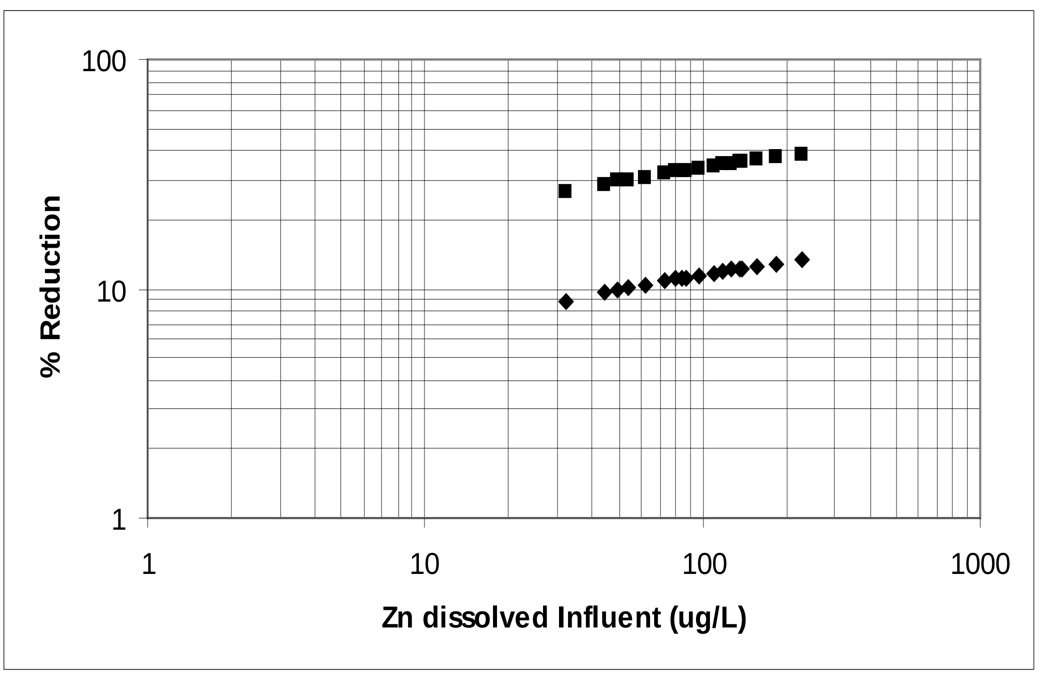
Fitted equation and data points for influent and effluent Dissolved Zn.



Residual analyses of fitted equation for Dissolved Zn influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

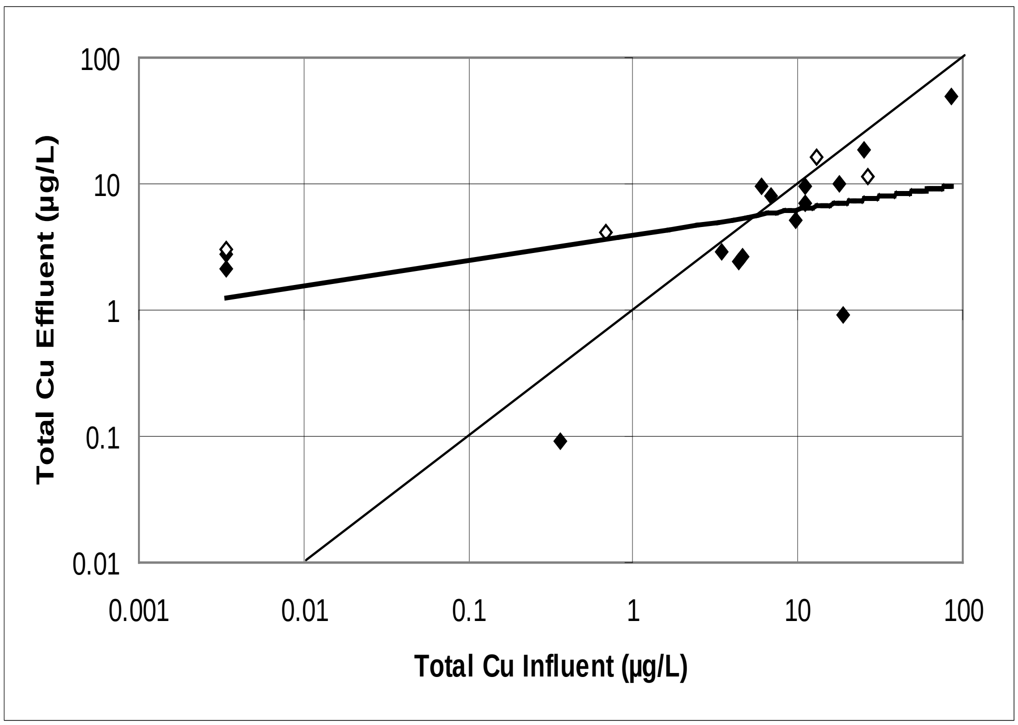


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

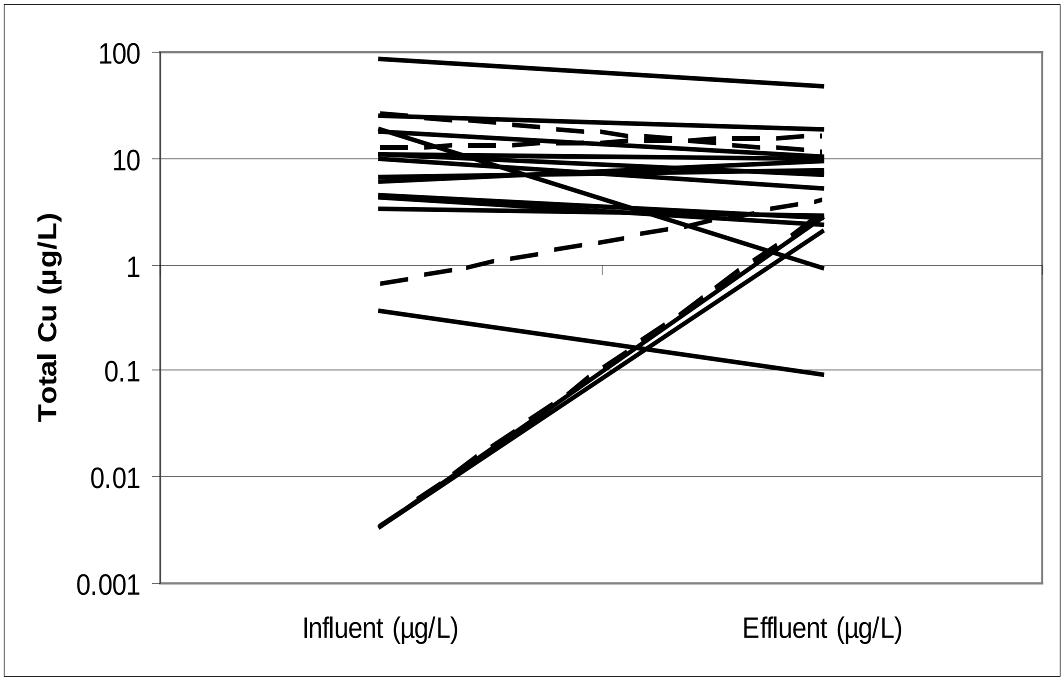
Total Copper

Observed Total Copper Concentrations

Sample Number	Influent (µg/L)	Effluent (µg/L)
1-1	4.6	2.7
2-1	9.7	5.2
5-1	25.5	18.9
5-2	13.0	16.5
6-1	18.0	10.2
6-2	6.8	7. 9
6-3	3.4	2.9
6-4	4.4	2.4
6-5	19.0	0.9
7-1	6.0	9.4
7-2	0.7	4.0
7-3	0.003	3.1
7-4	0.003	2. 1
7-5	0.003	2.7
7-6	0.4	0.1
8-1	85.8	48.2
8-2	26.3	11.5
9-1	11.2	7.0
10-1	11.0	9.7



Scatterplot of observed influent and effluent total Cu concentrations.



Paired influent and effluent total Cu concentrations (Sign test Significant P-value = 0.36).

Fitted Equation:

Effluent Total Cu, log µg/L = 0.598 + 0.1991 * (Influent Total Cu, log µg/L)

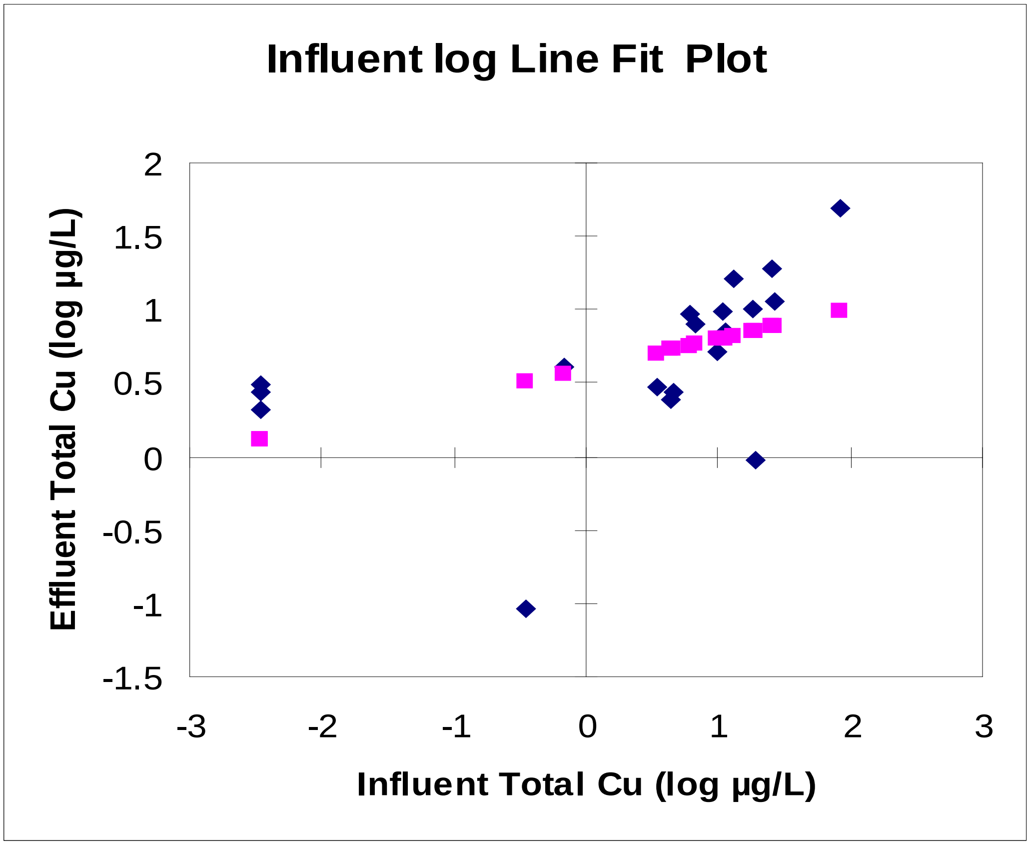
Regression Statistics on Observed Influent vs. Effluent Total Cu, log µg/L

Multiple R	0.470
R Square	0.221
Adjusted R Square	0.175
Standard Error	0.527
Observations	19

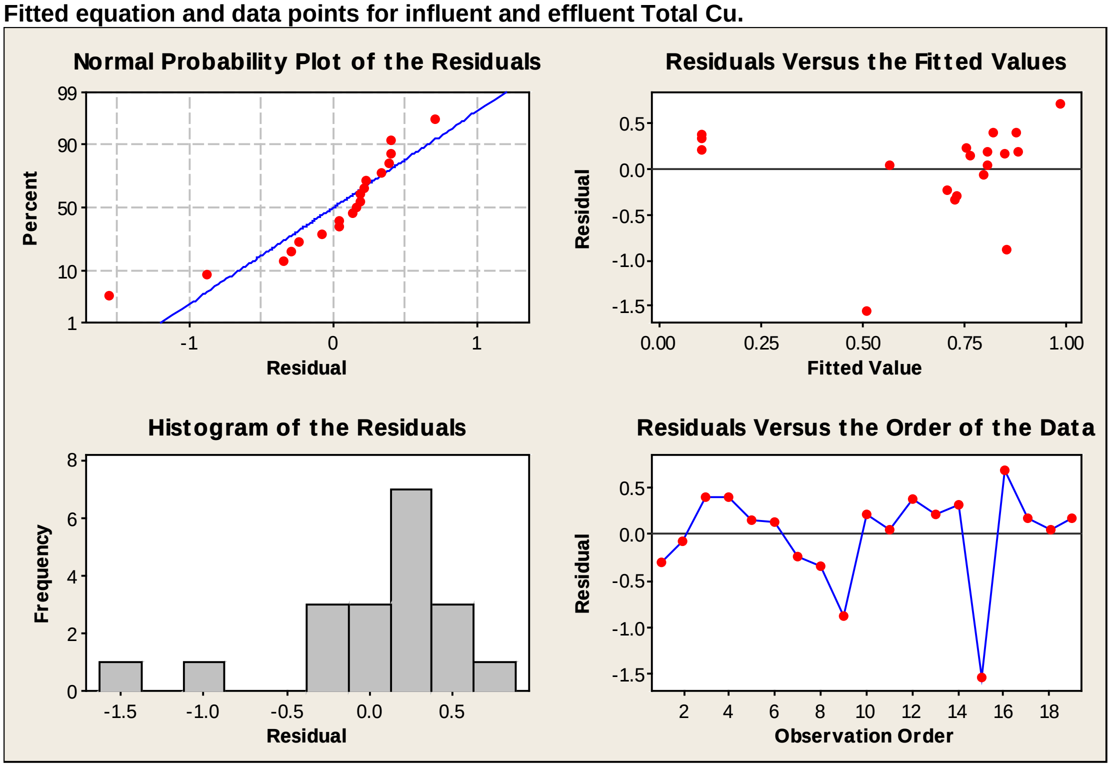
ANOVA

	df	SS	MS	F	Significance F
Regression	1	1.34	1.34	4.82	0.0424
Residual	17	4.72	0.278		
Total	18	6.06			

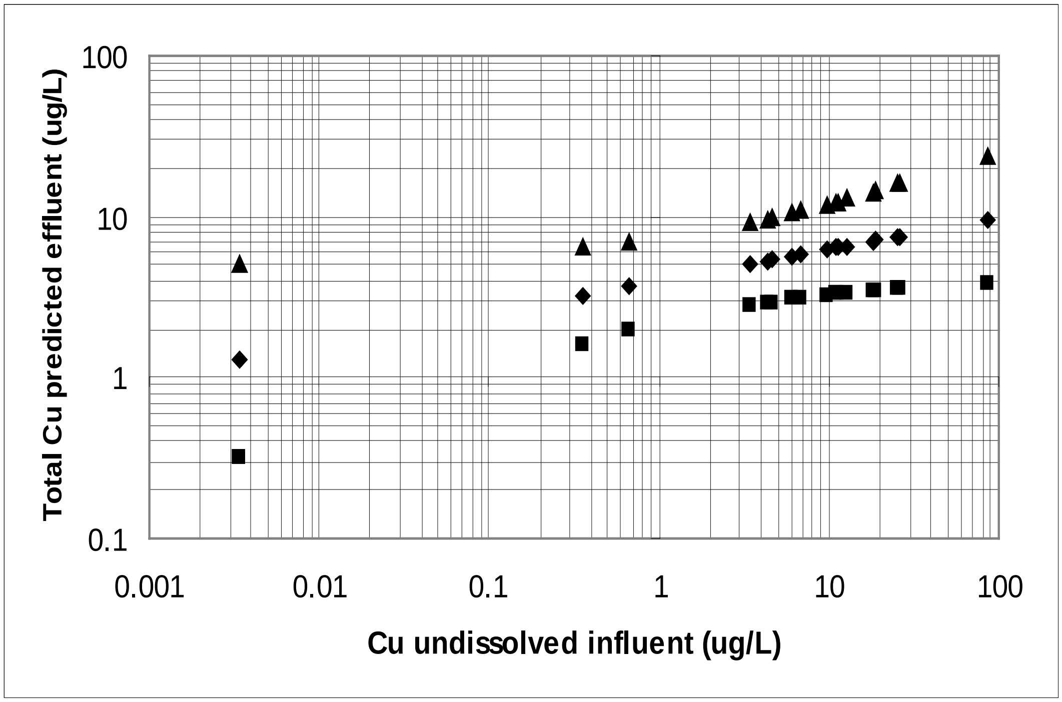
	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Intercept	0.597	0.125	4.76	0.000180	0.333	0.862
Influent Total Cu	0.199	0.0907	2.20	0.0423	0.00773	0.391



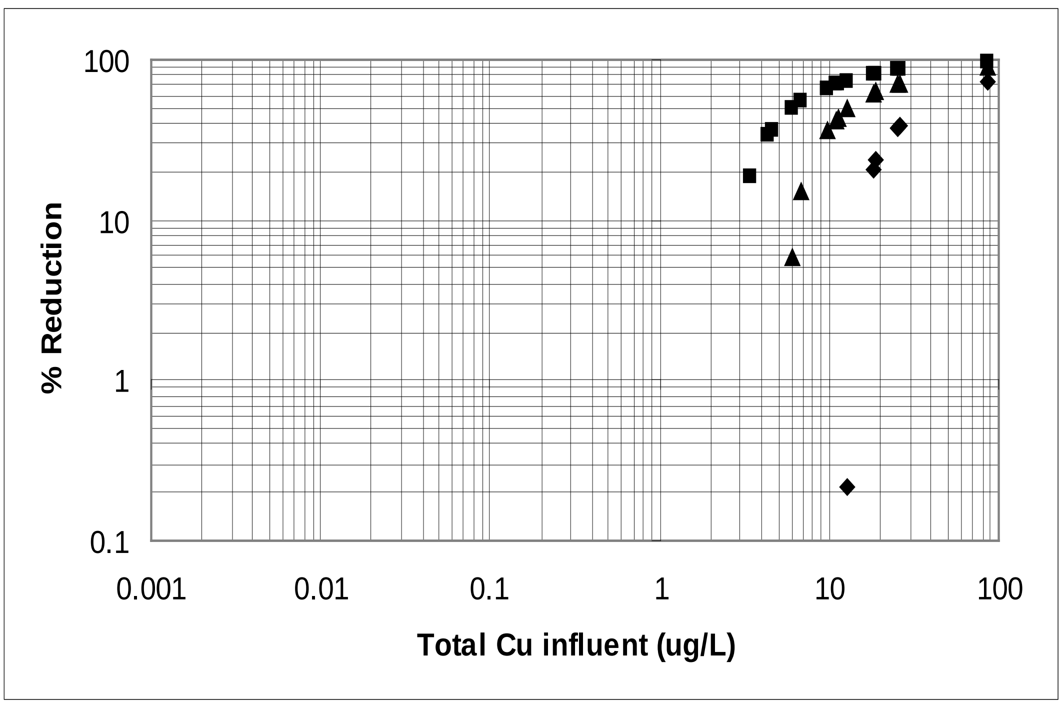
Fitted equation and data points for influent and effluent Total Cu.



Residual analyses of fitted equation for Total Cu influent vs. effluent.



Predicted effluent concentrations for different influent concentrations, with 95% confidence limits.

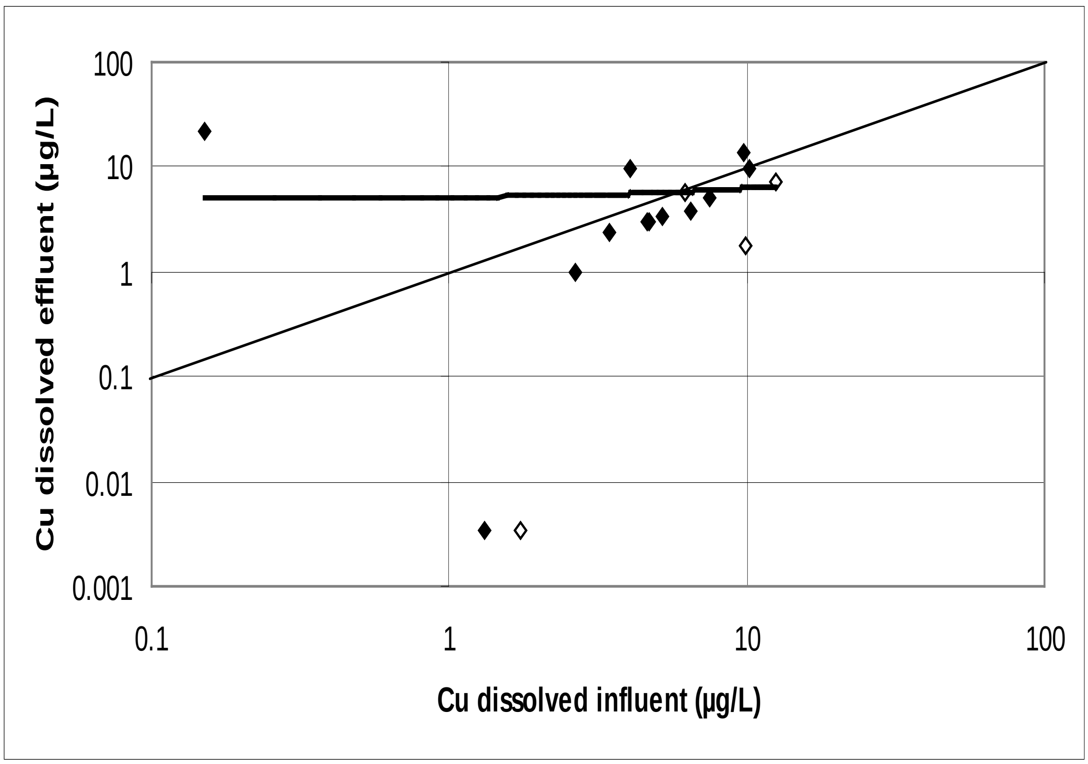


Percentage reductions as a function of influent concentrations, with 95% confidence limits.

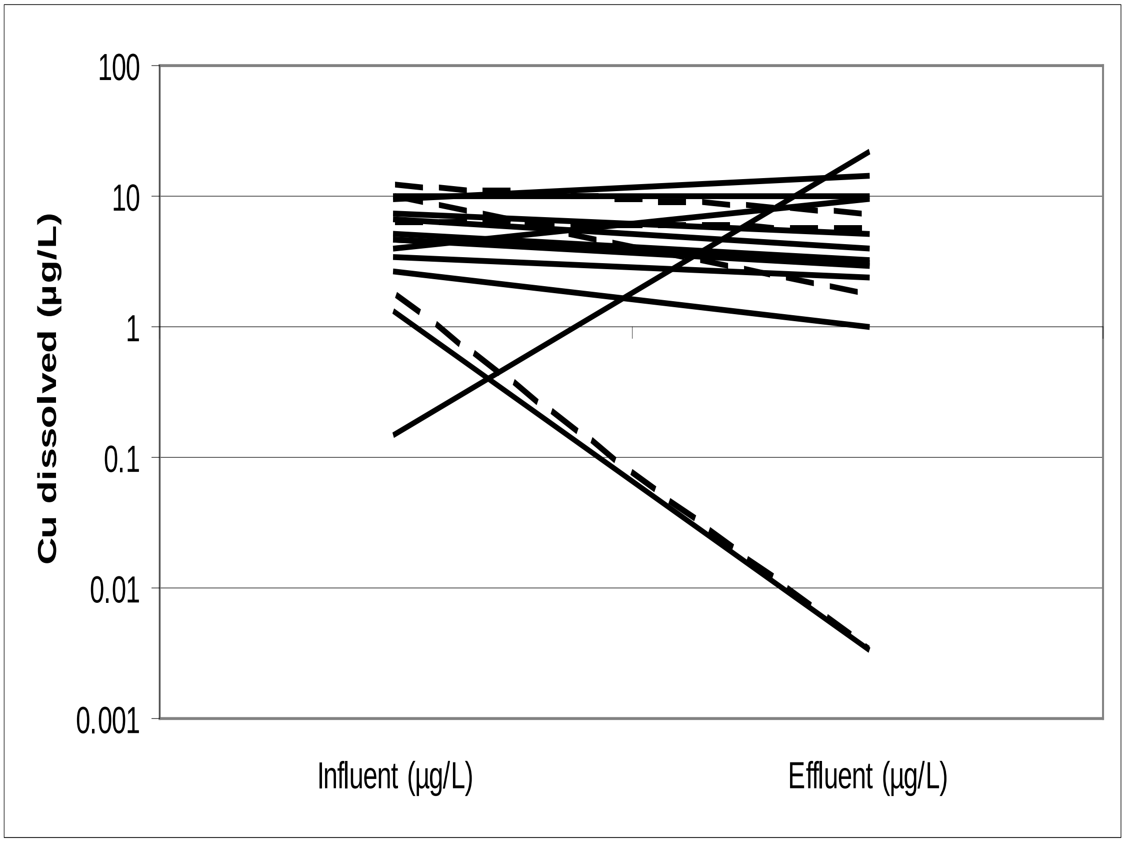
Dissolved Copper

Observed Dissolved Copper Concentrations

Sample Number	Influent (µg/L)	Effluent (µg/L)
1-1	3.43	2.35
5-1	9.73	14.07
5-2	12.50	7.27
6-1	10.18	9.83
6-2	6.49	3.90
6-3	5.20	3.32
6-4	4.72	3.00
6-5	2.67	0.98
7-1	4.08	9.44
7-2	9.85	1.77
7-3	1.73	0.0034
7-6	1.32	0.0034
8-1	0.15	22.07
8-2	6.24	5.76
9-1	4.63	3.02
10-1	7.55	5.20



Scatterplot of observed influent and effluent dissolved Cu concentrations.



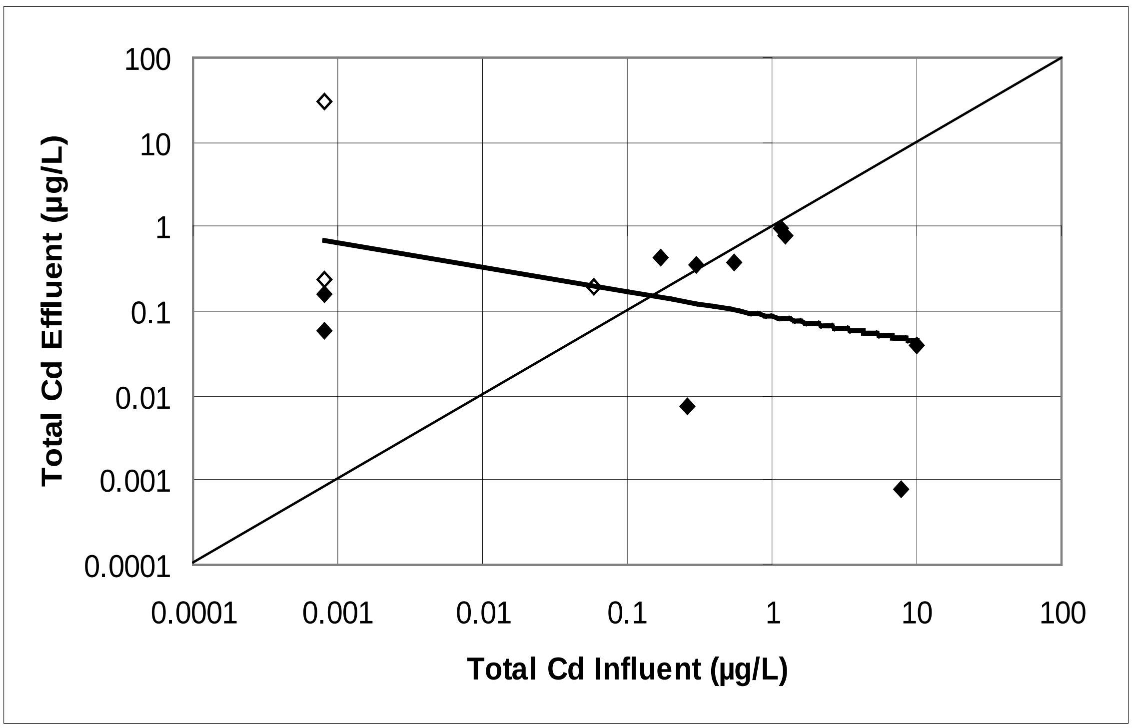
Paired influent and effluent dissolved Cu concentrations (Sign test significant P-value = 0.02).

The ANOVA evaluations of the alternative regression equations for dissolved copper did not show any significant equations or equation coefficients.

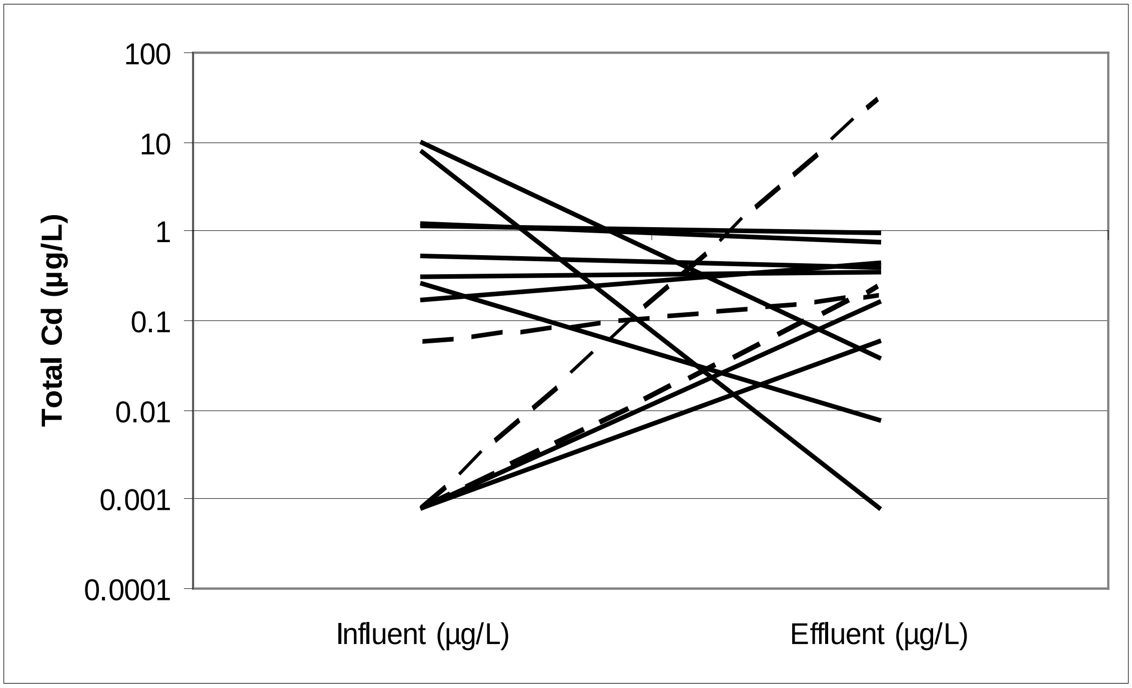
Total Cadmium

Observed Total Cadmium Concentrations

Sample Number	Influent (µg/L)	Effluent (µg/L)
1-1	1.23	0.76
2-1	0.54	0.39
5-1	10.0	0.039
5-2	0.0008	29.92
6-5	7.82	0.0008
7-1	0.0008	0.156
7-2	0.0008	0.24
7-5	0.0008	0.058
7-6	0.17	0.44
8-1	1.15	0.95
8-2	0.058	0.20
9-1	0.26	0.008
10-1	0.30	0.36



Scatterplot of observed influent and effluent total Cd concentrations.



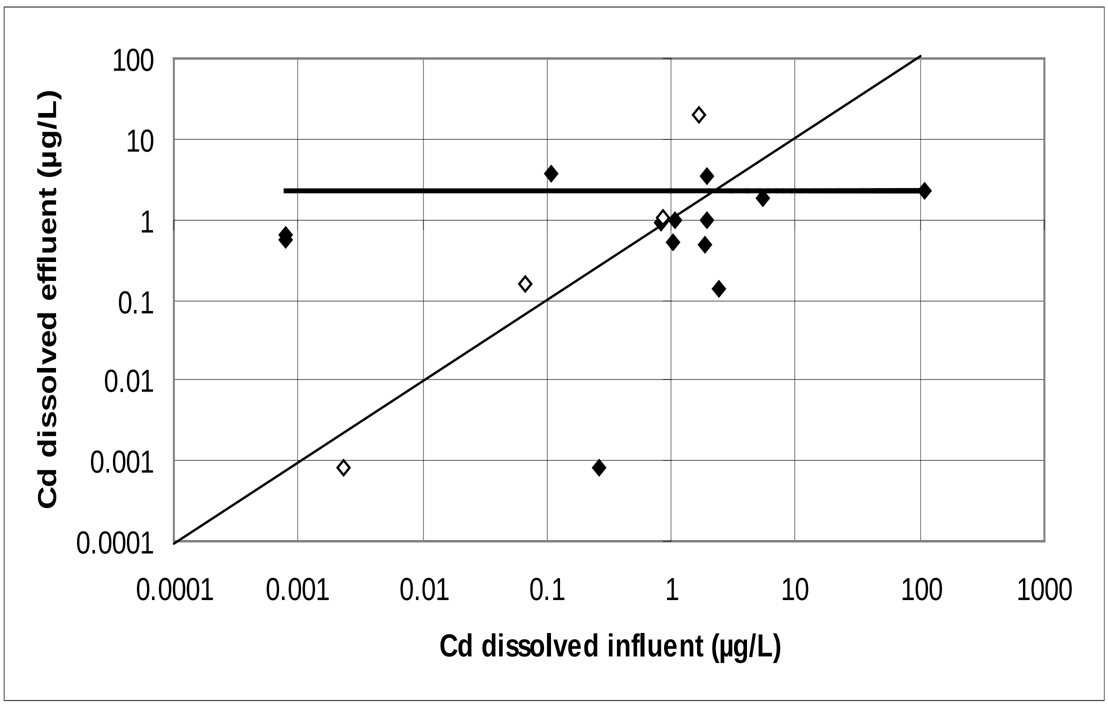
Paired influent and effluent total Cd concentrations (Sign test significant P-value = 1.0).

The ANOVA evaluations of the alternative regression equations for total cadmium did not show any significant equations or equation coefficients.

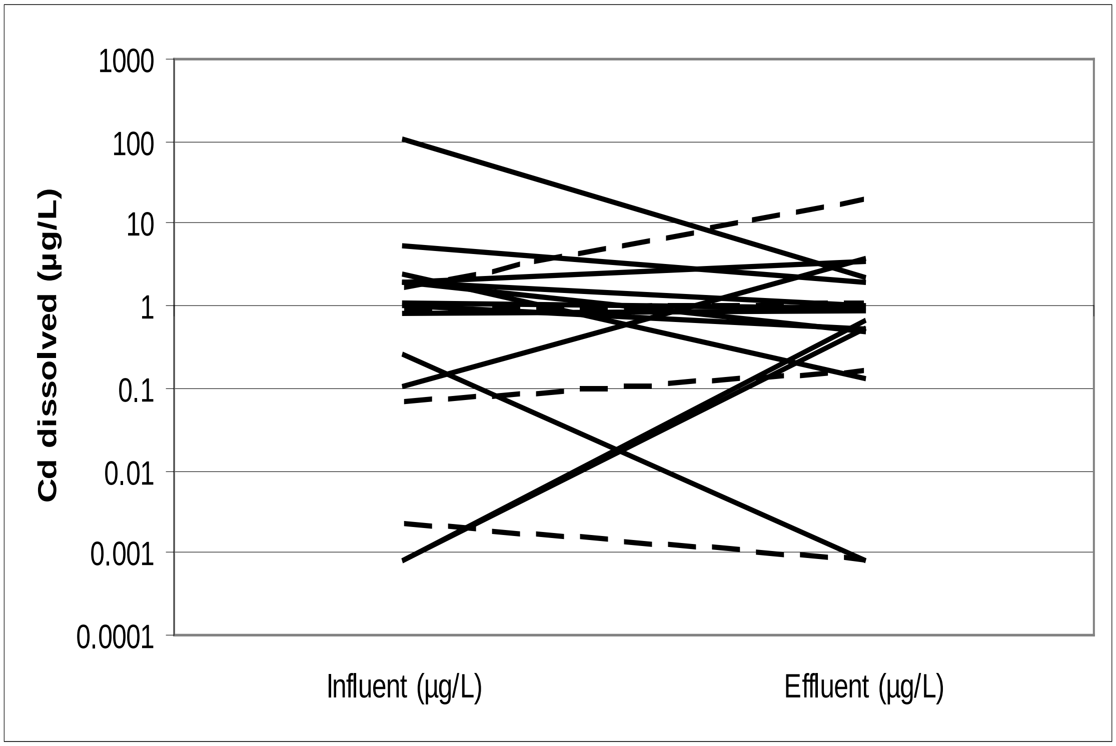
Dissolved Cadmium

Observed Dissolved Cadmium Concentrations

Sample Number	Influent (µg/L)	Effluent (µg/L)
1-1	1.06	0.96
5-1	1.88	0.49
5-2	0.0023	0.0008
6-2	0.26	0.0008
6-3	0.0008	0.54
6-4	1.03	0.53
6-5	0.0008	0.65
7-1	109.1	2.23
7-2	0.86	1.06
7-3	1.64	20.32
7-4	5.52	1.88
7-5	0.11	3.83
7-6	1.90	0.99
8-1	0.82	0.88
8-2	0.067	0.16
9-1	1.93	3.40
10-1	2.46	0.13



Scatterplot of observed influent and effluent dissolved Cd concentrations.



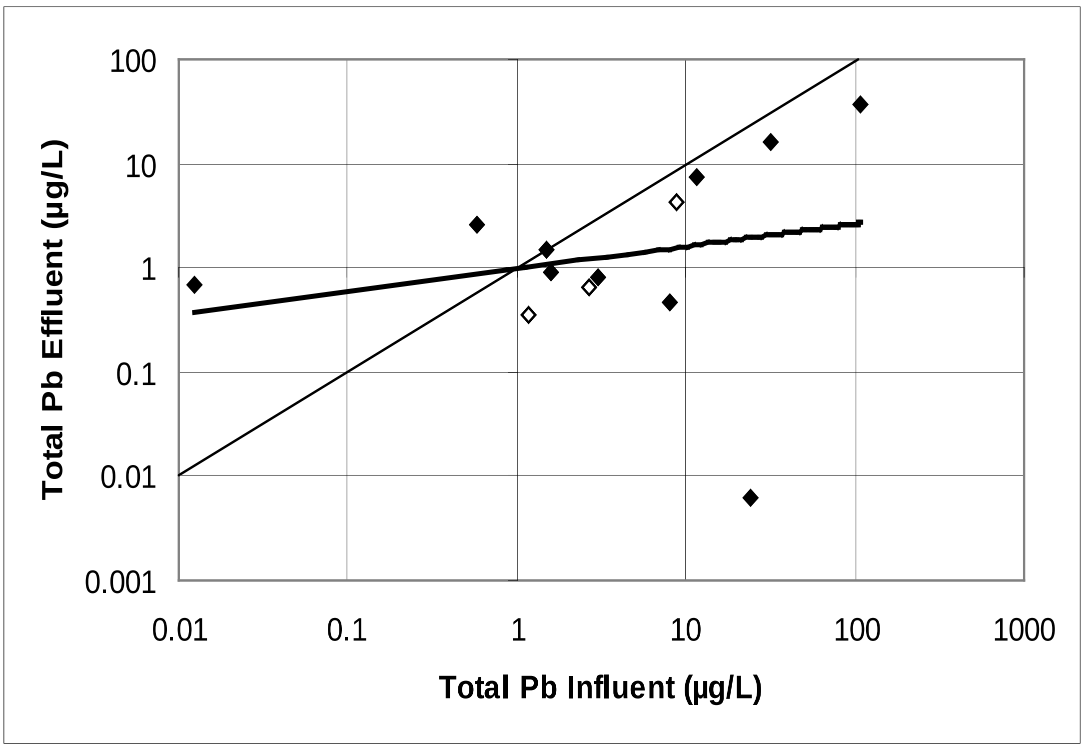
Paired influent and effluent dissolved Cd concentrations (Sign test significant P-value = 1.0).

The ANOVA evaluations of the alternative regression equations for dissolved cadmium did not show any significant equations or equation coefficients.

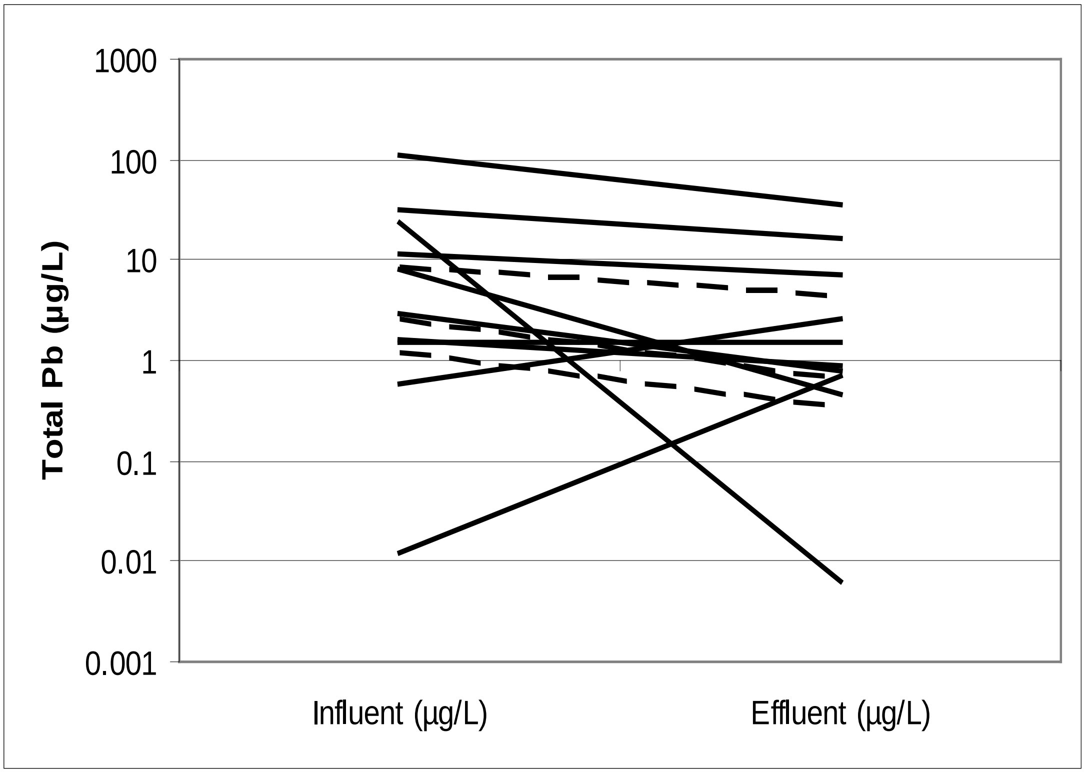
Total Lead

Observed Total Lead Concentrations

Sample Number	Influent (µg/L)	Effluent (µg/L)
1-1	31.68	16.17
2-1	11.44	7.26
5-1	24.16	0.0062
7-1	1.50	1.51
7-2	2.64	0.65
7-3	1.19	0.345
7-4	0.58	2.60
7-5	0.012	0.68
7-6	1.60	0.91
8-1	107.4	36.29
8-2	8.67	4.35
9-1	7.90	0.47
10-1	3.00	0.80



Scatterplot of observed influent and effluent total Pb concentrations.



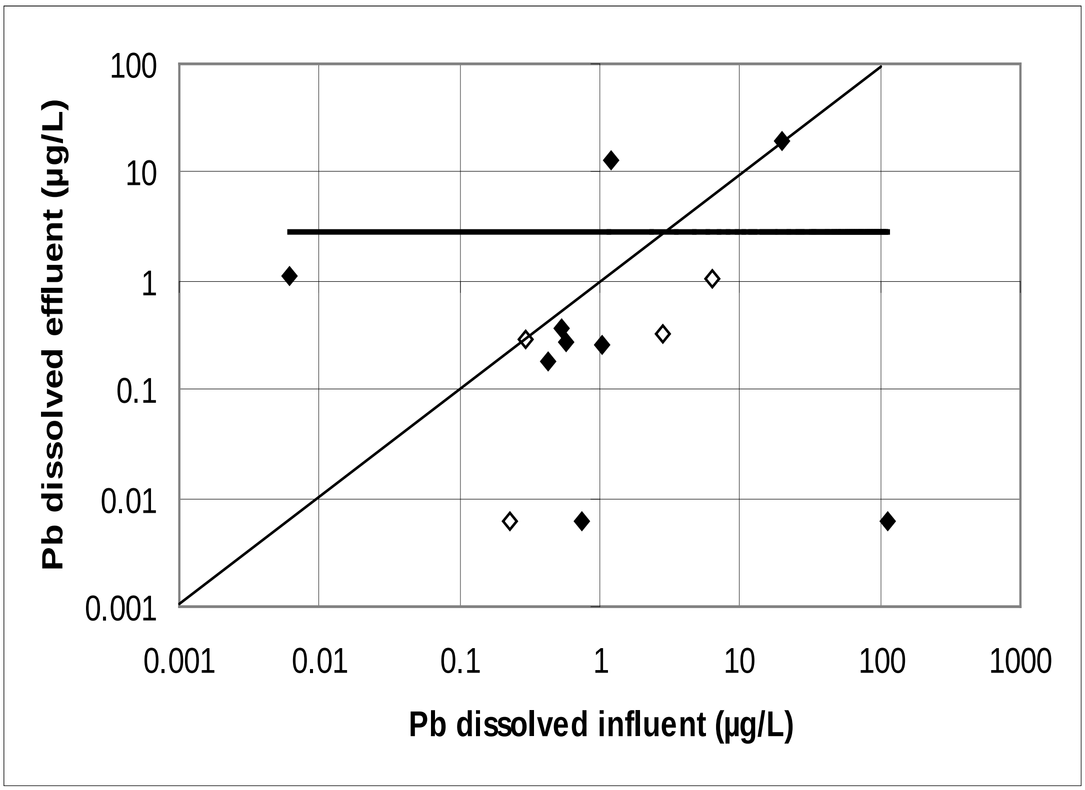
Paired influent and effluent total Pb concentrations (Sign test significant P-value = 0.09).

The ANOVA evaluations of the alternative regression equations for total lead did not show any significant equations or equation coefficients.

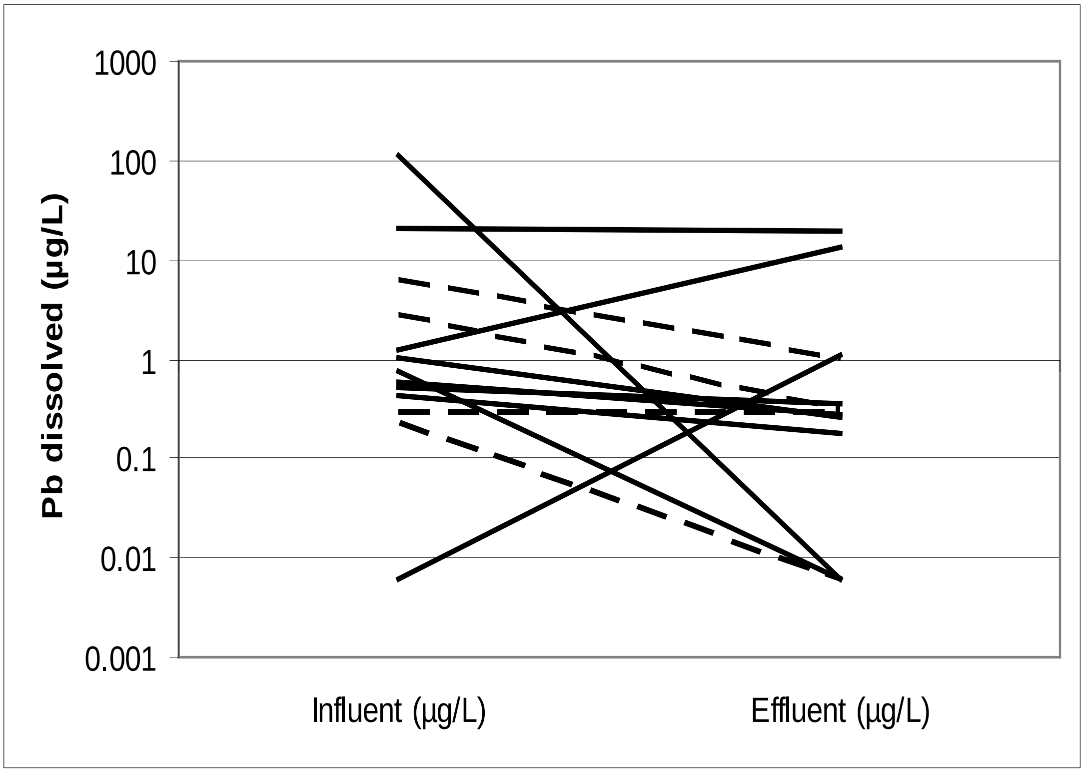
Dissolved Lead

Observed Dissolved Lead Concentrations

Sample Number	Influent (µg/L)	Effluent (µg/L)
1-1	20.3	19.5
5-1	112	0.0062
5-2	0.23	0.0062
7-1	0.0062	1.13
7-2	0.30	0.30
7-3	2.83	0.32
7-4	0.43	0.18
7-5	0.53	0.36
7-6	1.06	0.26
8-1	1.22	13.2
8-2	6.23	1.05
9-1	0.74	0.0062
10-1	0.58	0.28



Scatterplot of observed influent and effluent dissolved Pb concentrations.



Paired influent and effluent dissolved Pb concentrations (Sign test P-value = 0.023).

The ANOVA evaluations of the alternative regression equations for dissolved lead did not show any significant equations or equation coefficients.

Appendix D

Sonde Data

The data on sondes is not available of the 4 storms April 26, 05; August 29, 05; September 16, 05; September 25, 05.

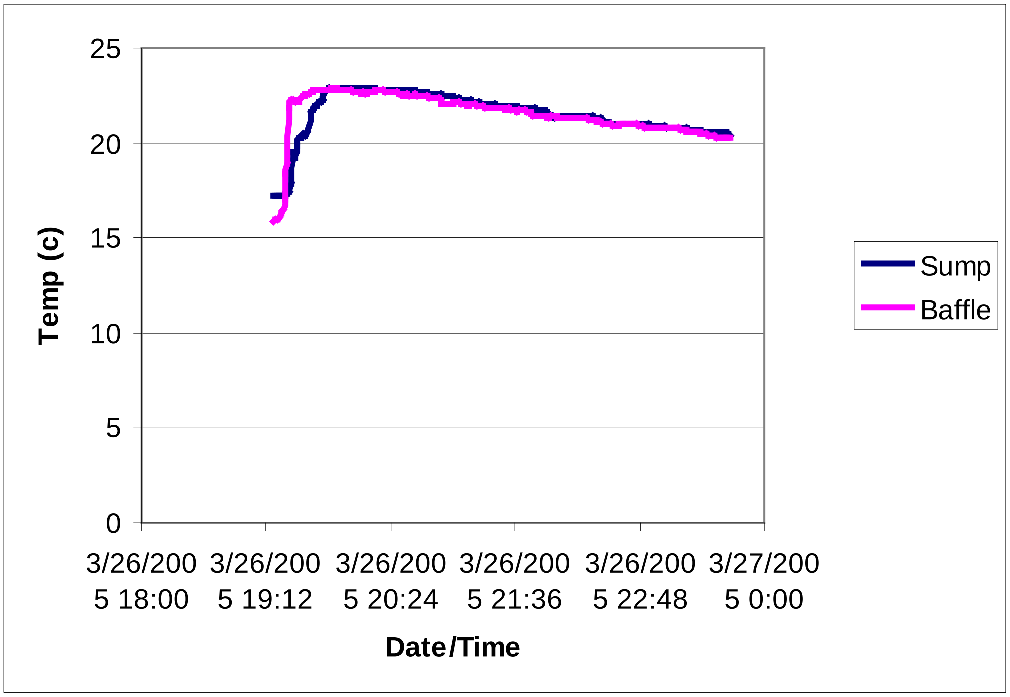


Figure 1: Comparison of Temperature for Storm Event on March 26, 05

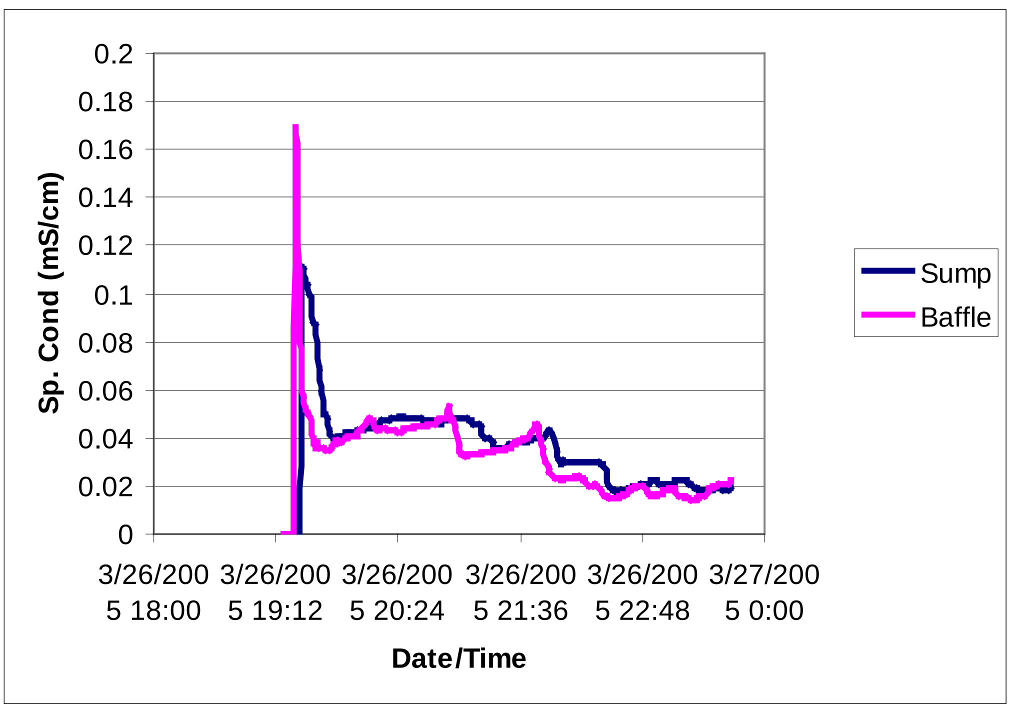


Figure 2: Comparison of Specific Conductivity for Storm Event on March 26, 05

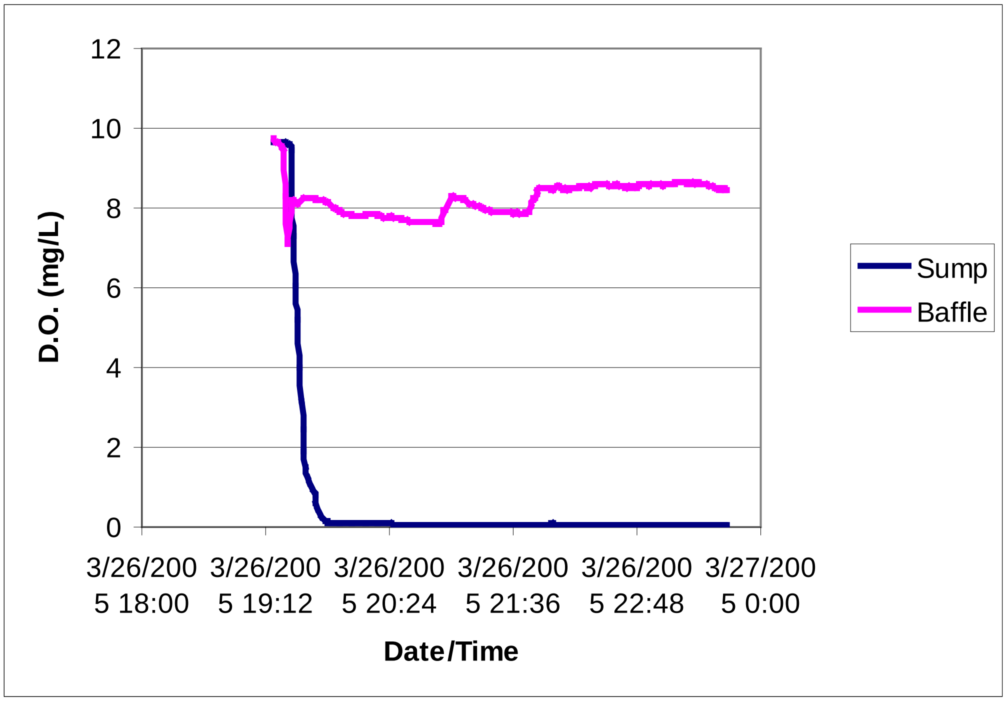


Figure 3: Comparison of Dissolved Oxygen for Storm Event on March 26, 05

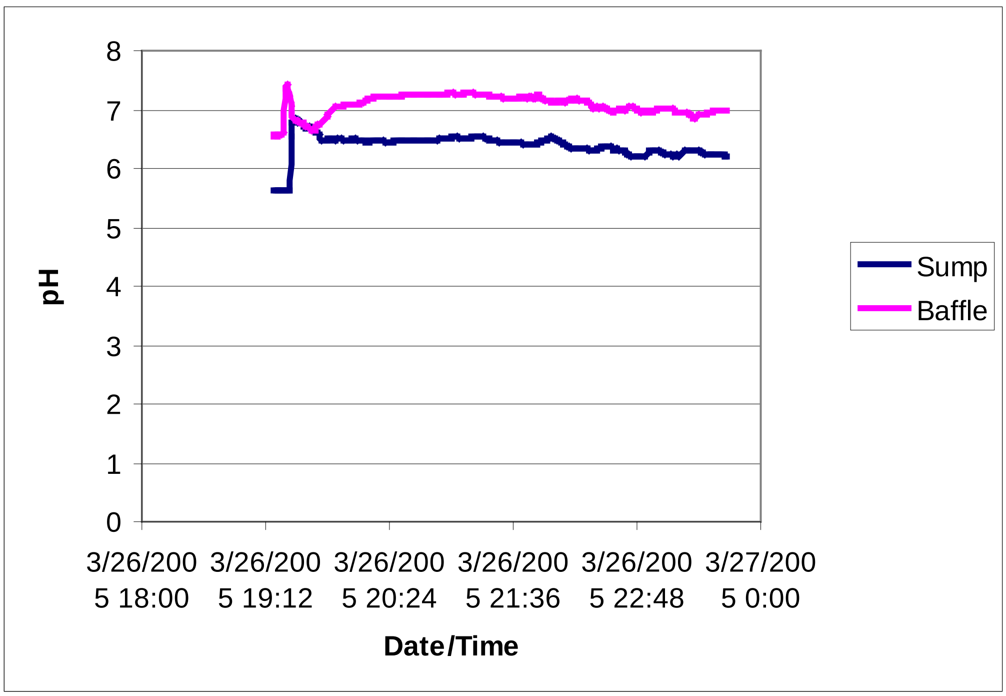


Figure 4: Comparison of pH for Storm Event on March 26, 05

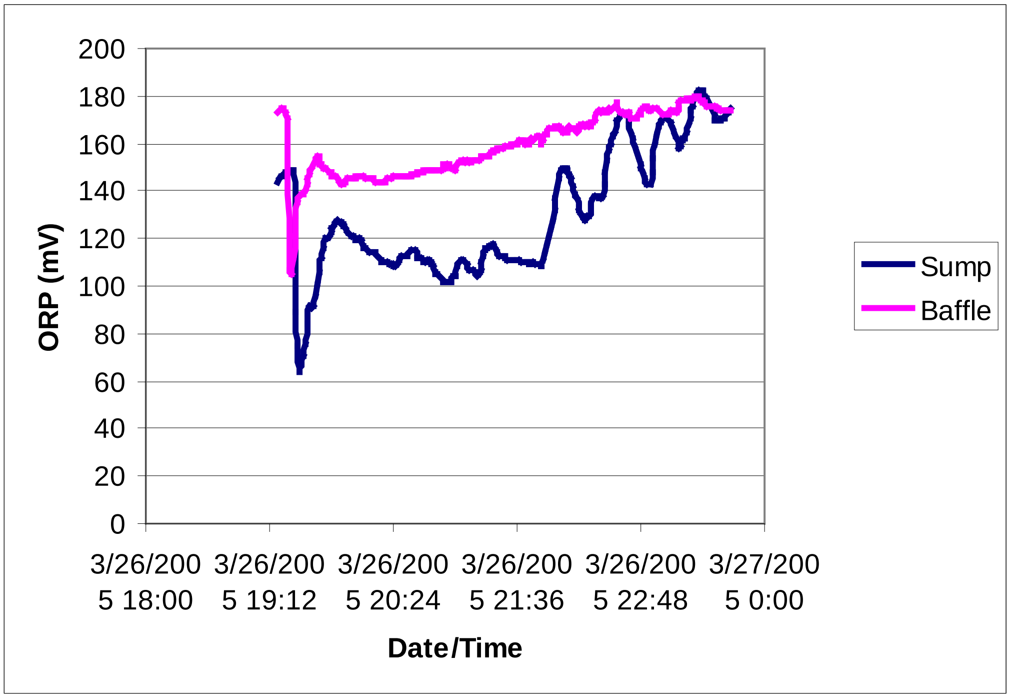


Figure 5: Comparison of ORP for Storm Event on March 26, 05

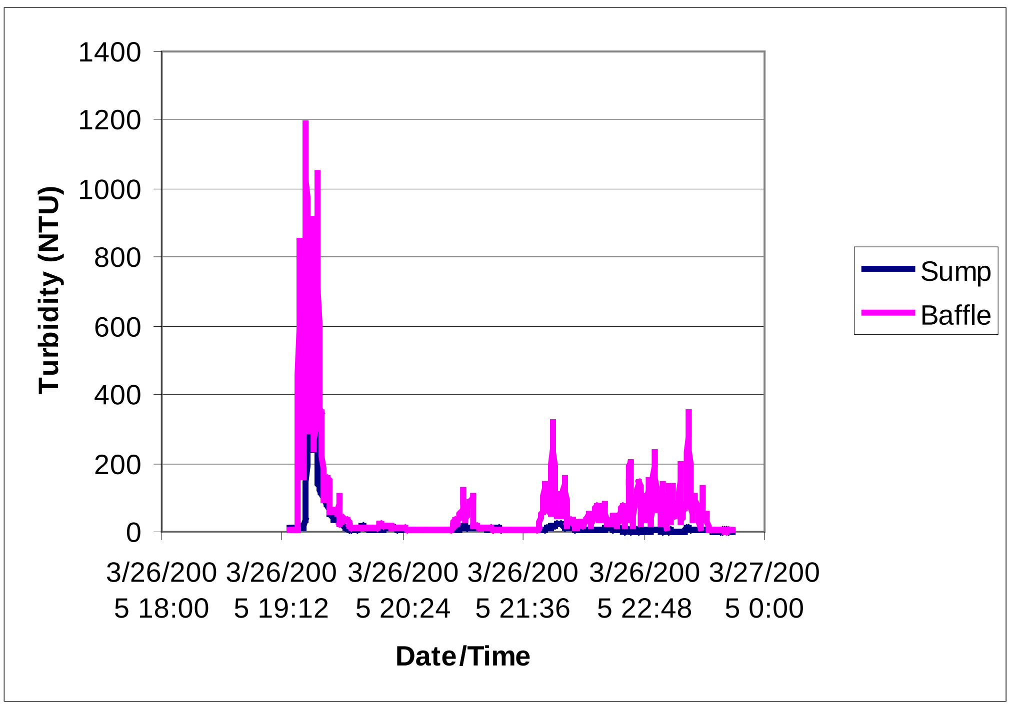


Figure 6: Comparison of Turbidity for Storm Event on March 26, 05

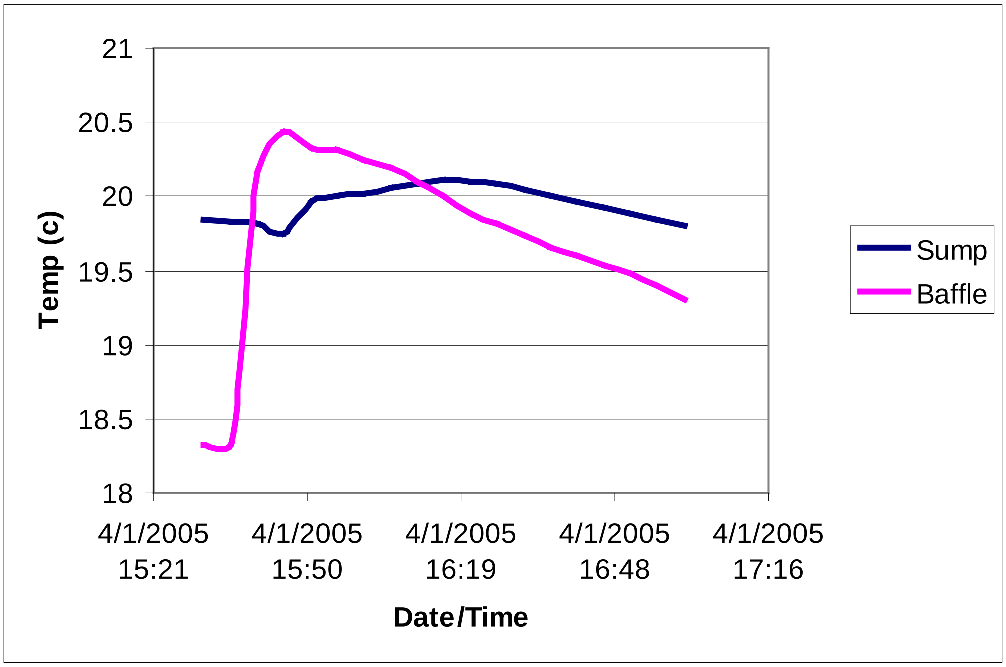


Figure 7: Comparison of Temperature for Storm Event on April 01, 05

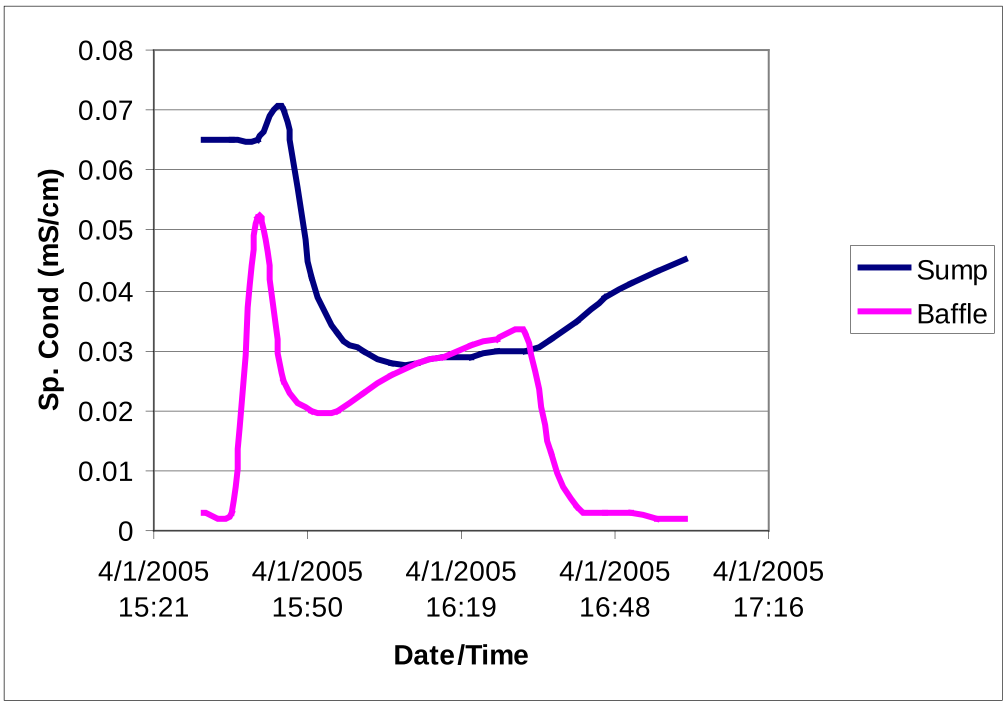


Figure 8: Comparison of Specific Conductivity for Storm Event on April 01, 05

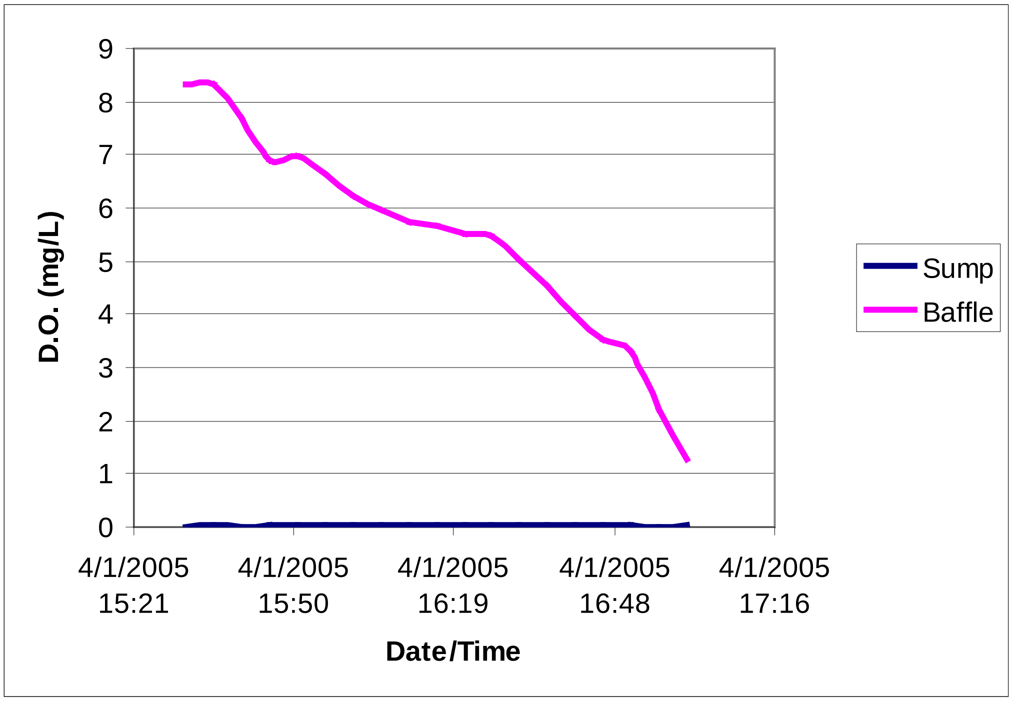


Figure 9: Comparison of Dissolved Oxygen for Storm Event on April 01, 05

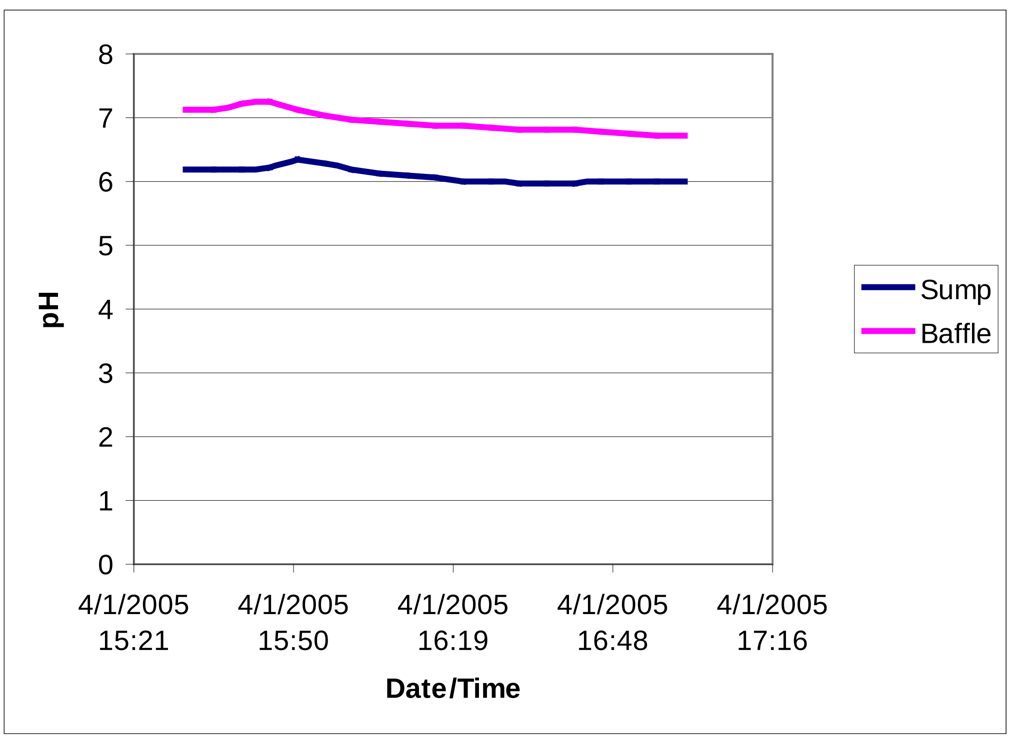


Figure 10: Comparison of pH for Storm Event on April 01, 05

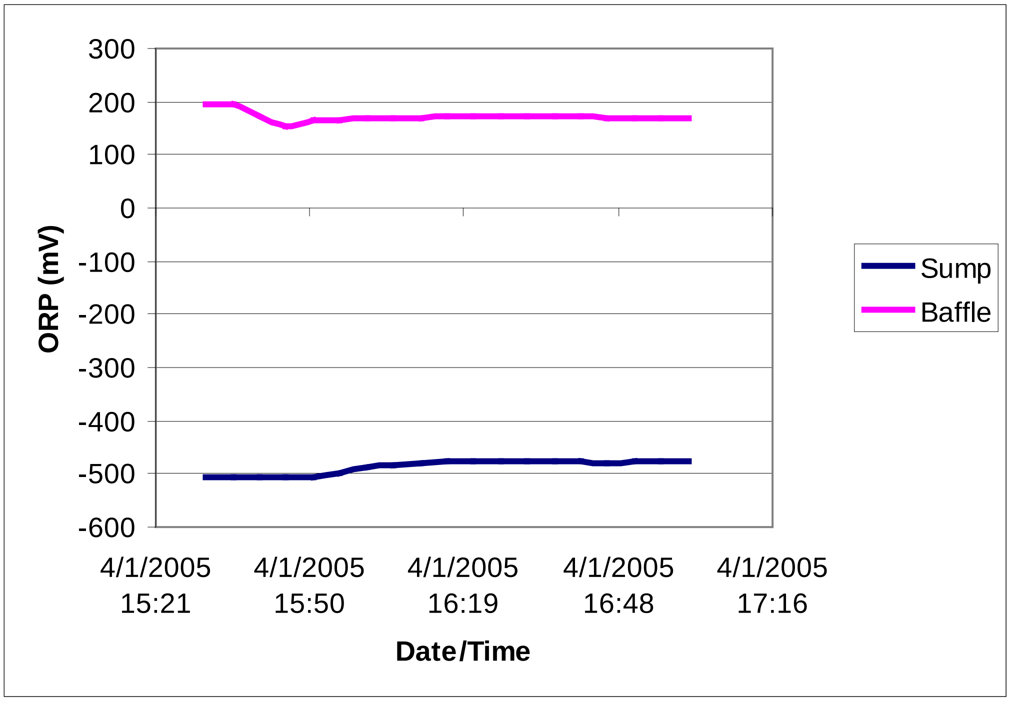


Figure 11: Comparison of ORP for Storm Event on April 01, 05

Appendix E: Sump Sediment Quality and Quantity Data

Observed Quantity and Quality of Sediment Collected from UpFlow Filter Sump

Sediment Size Range (µm)	total amount in sump (kilograms)	% of total amount is sump in size range	COD (mg/kg)	P (mg/kg)	Ca (mg/kg)	Mg (mg/kg)	Fe (mg/kg)	Cu (mg/kg)	Cr (mg/kg)	Pb (mg/kg)	Zn (mg/kg)
<75	1. 1	2.0	233,000	3,580	117	15,600	6,050	190	21.2	80.4	1,340
75-150	1.6	2.9	129,000	1,620	177	25,300	4,960	99.8	17.4	70.3	958
150-250	3.6	6.6	35,500	511	158	22,900	3,010	48.2	8.0	34.9	501
250-425	11.5	21.5	60,100	315	134	18,600	2,790	33.6	6.7	28.4	539
425-850	17.1	31.9	45,000	496	146	19,700	2,290	22.1	3.7	23.7	270
850-2,000	10.5	19.6	29,200	854	312	44,700	4,050	27.8	6.9	25.1	414
2,000-4,750	4.8	8.9	143,000	1,400	452	65,000	4,430	54.9	10.5	27.8	450
>4,750	3.5	6.5	251,000	1,700	134	8,390	7,000	48.7	9.3	59.6	564
	53.6	100.0									

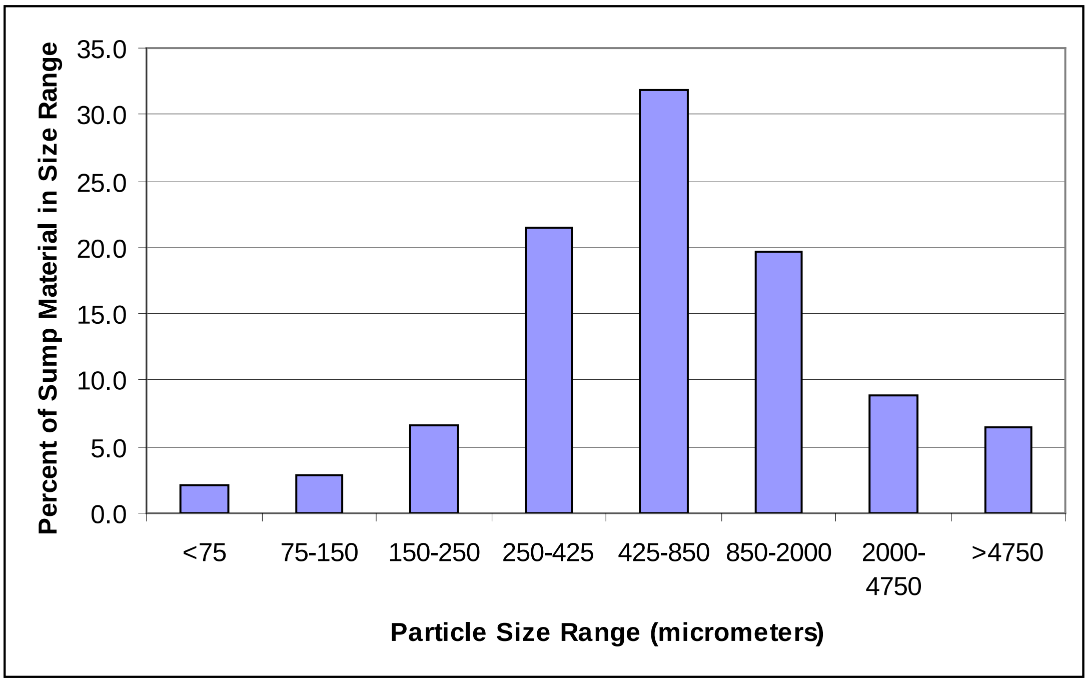
For reference, the following is a copy of Table 4 from the literature review section showing sediment quality from older Bellevue, WA, catchbasin and inlet tests. These values are comparable, with the expected differences for lead. Current lead sediment concentration values are about 1/10th of the older values.

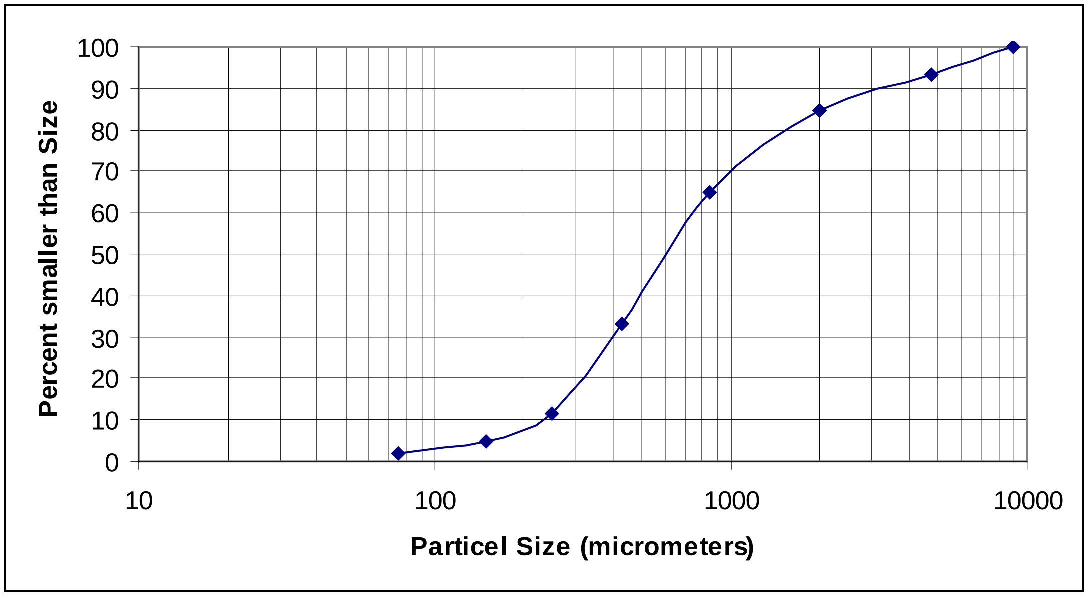
Table 4. Chemical Quality of Bellevue, WA, Inlet Structure Sediment (mg constituent/kg total solids) (Pitt 1985)

Particle Size (µm)	COD	TKN	TP	Pb*	Zn
<63	160,000	2,900	880	1,200	400
61-125	130,000	2,100	690	870	320
125-250	92,000	1,500	630	620	200
250-500	100,000	1,600	610	560	200
500-1,000	140,000	1,600	550	540	200
1,000-2,000	250,000	2,600	930	540	230
2,000-6,350	270,000	2,500	1,100	480	190
>6,350	240,000	2,100	760	290	150

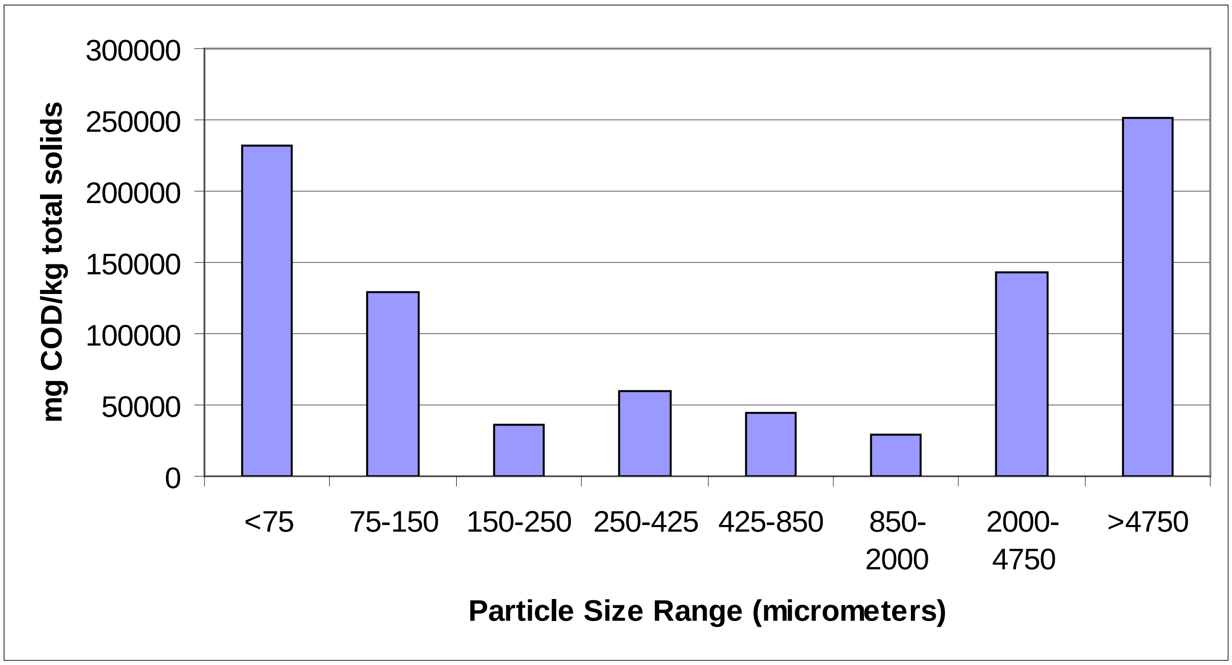
* these lead values are much higher than would be found for current samples due to the decreased use of leaded gasoline since 1981.

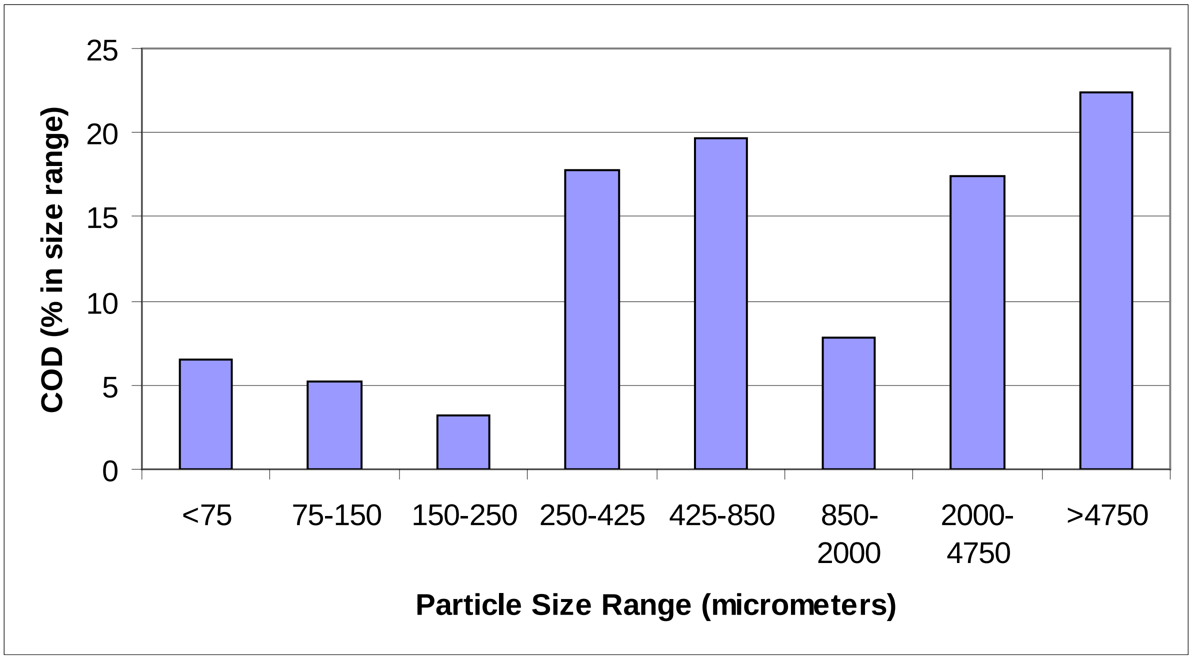
Total Solids in Upflow Filter Sump



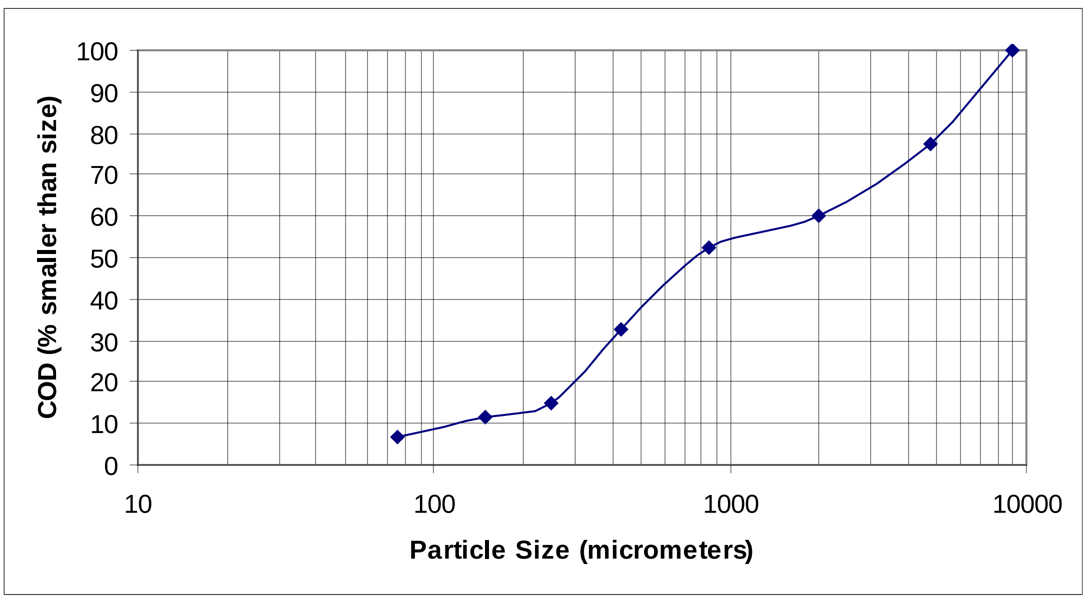

100
90
80
70
60
50
40
30
20
10
0
Percent smaller than Size
10
100
1000
10000
Particel Size (micrometers)

Chemical Oxygen Demand in Upflow Filter Sump



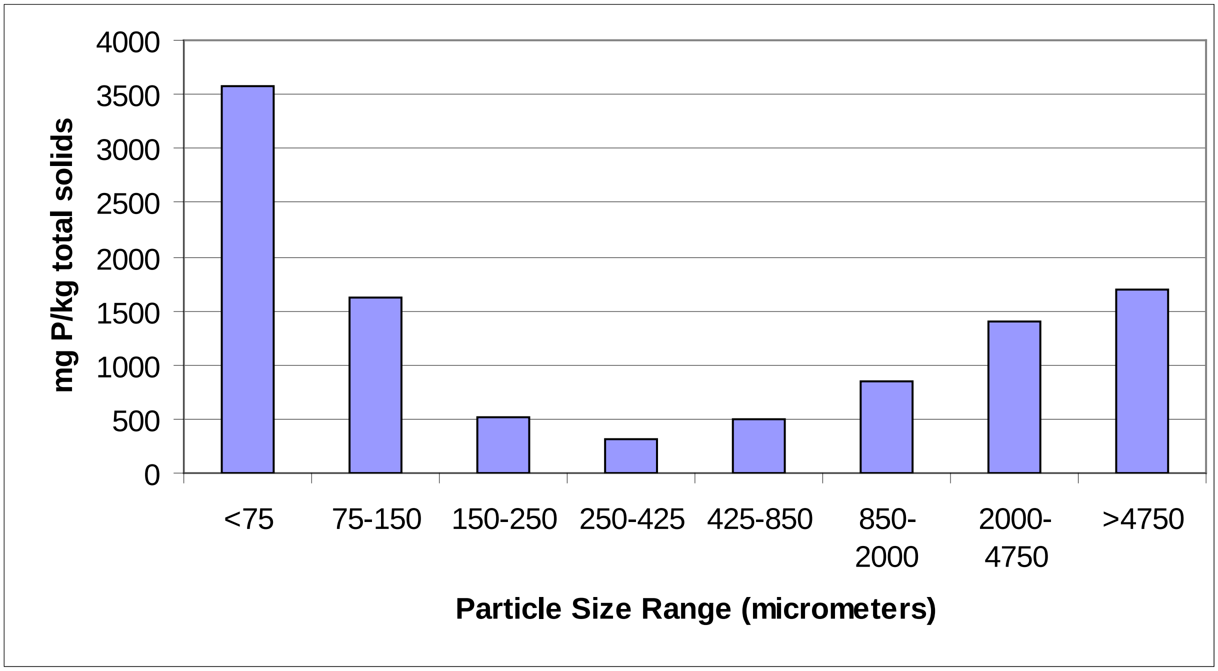


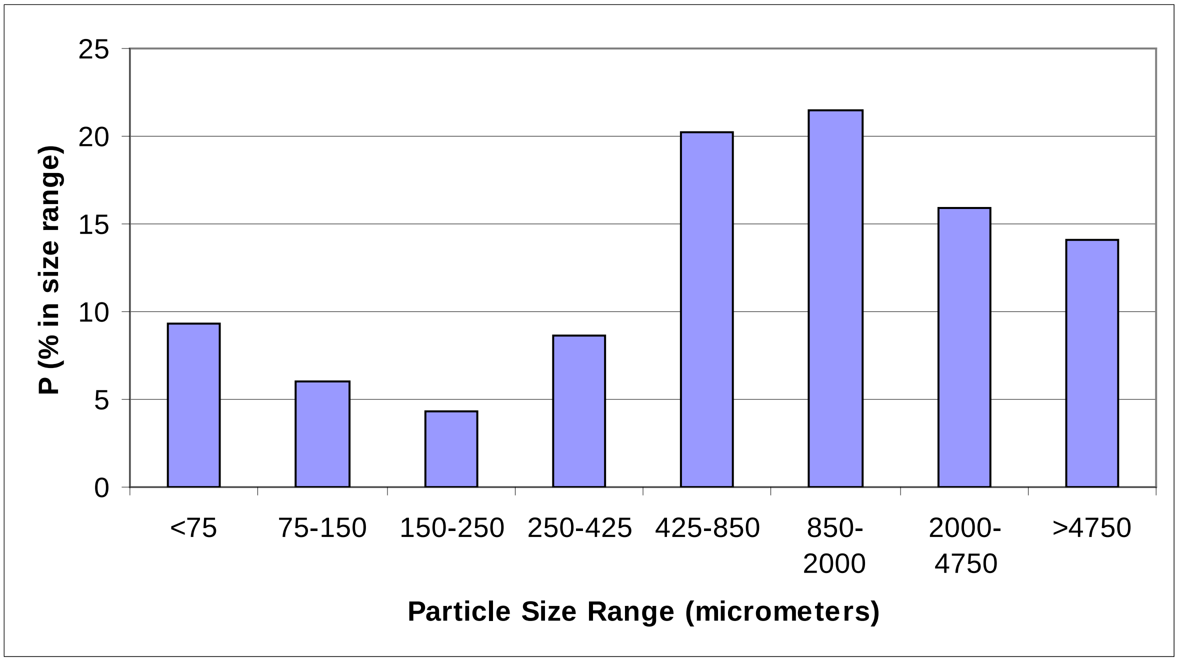
25
20
15
10
5
0
COD (% in size range)
<75
75-150
150-250
250-425
425-850
850-2000
2000-4750
>4750
Particle Size Range (micrometers)



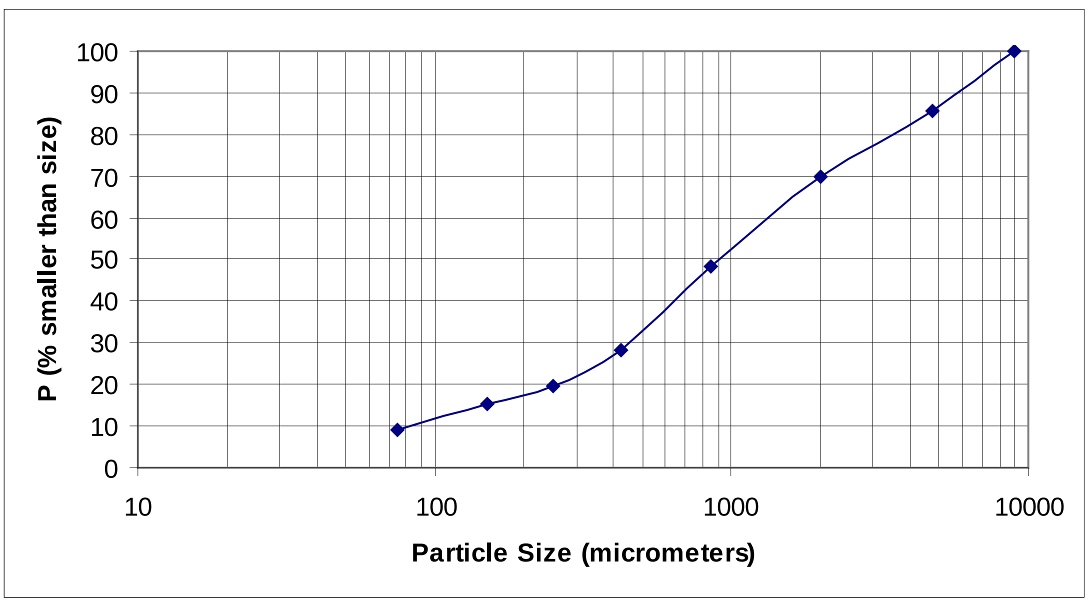
100
90
80
70
60
50
40
30
20
10
0
COD (% smaller than size)
10
100
1000
10000
Particle Size (micrometers)

Total Phosphorus in Upflow Filter Sump



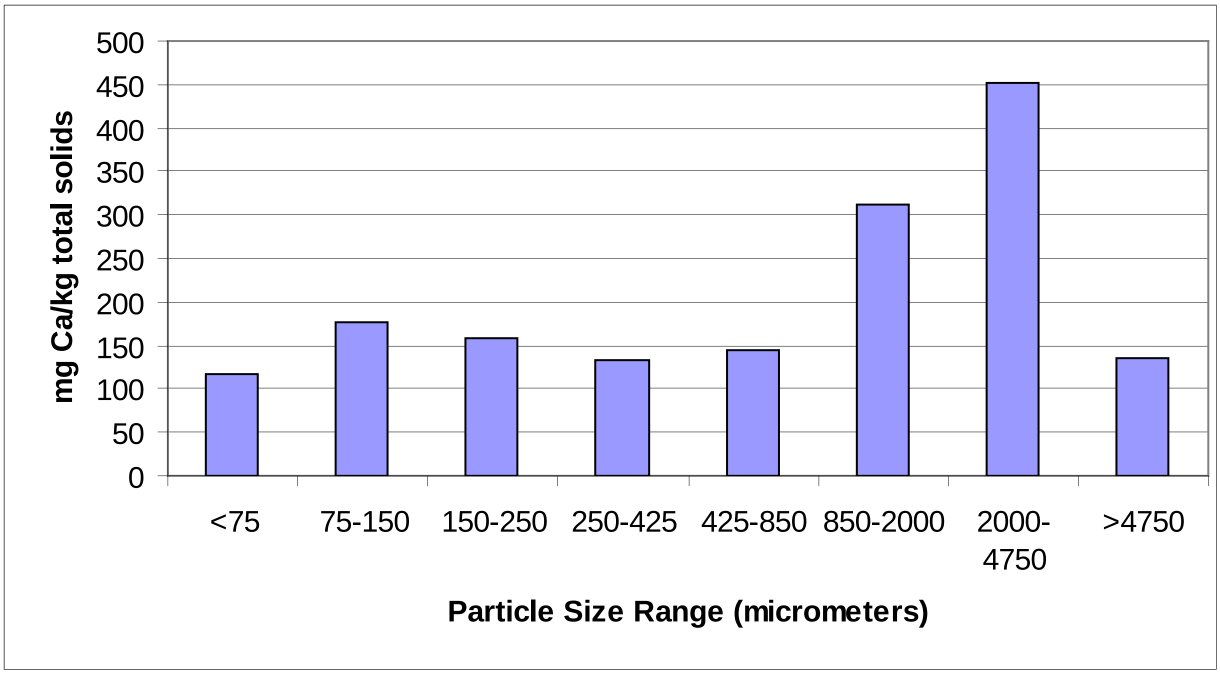


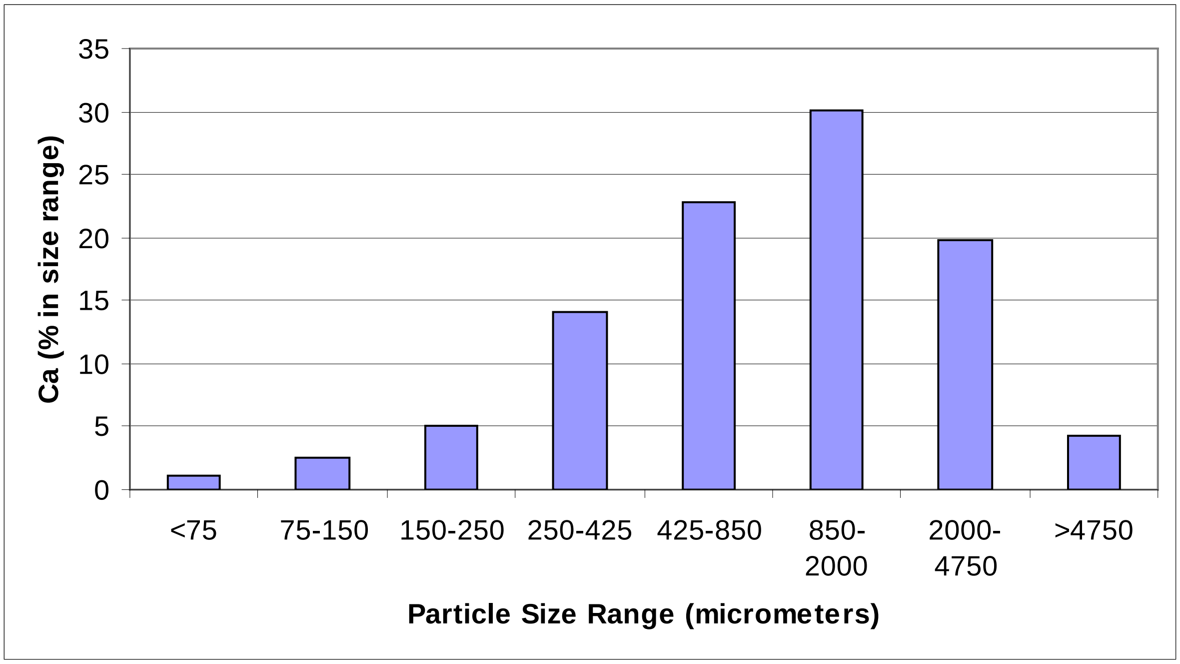
25
20
15
10
5
0
P (% in size range)
<75
75-150
150-250
250-425
425-850
850-2000
2000-4750
>4750
Particle Size Range (micrometers)

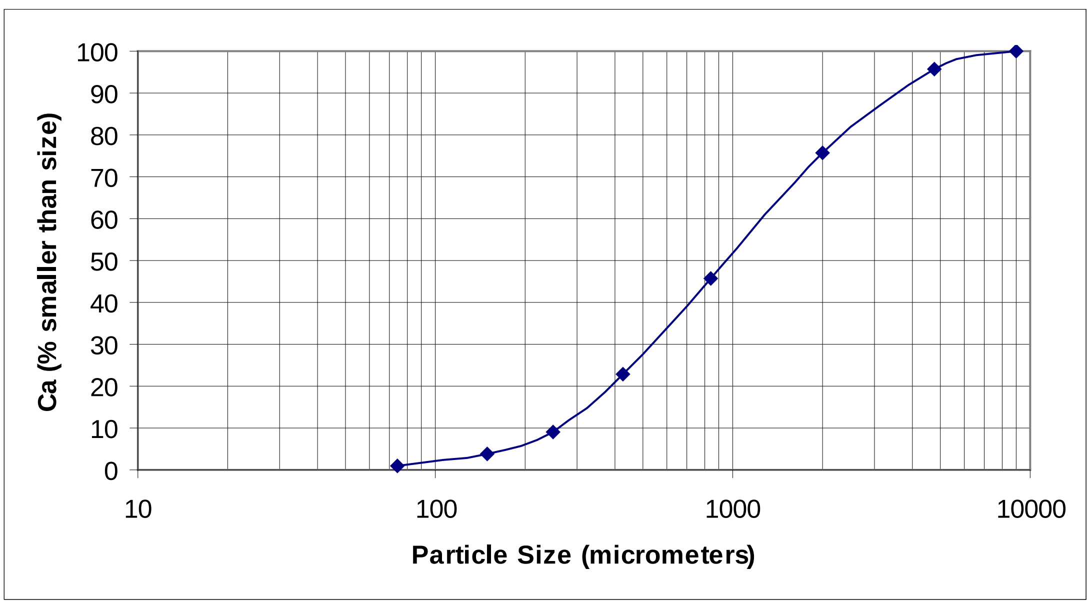


100
90
80
70
60
50
40
30
20
10
0
P (% smaller than size)
10
100
1000
10000
Particle Size (micrometers)

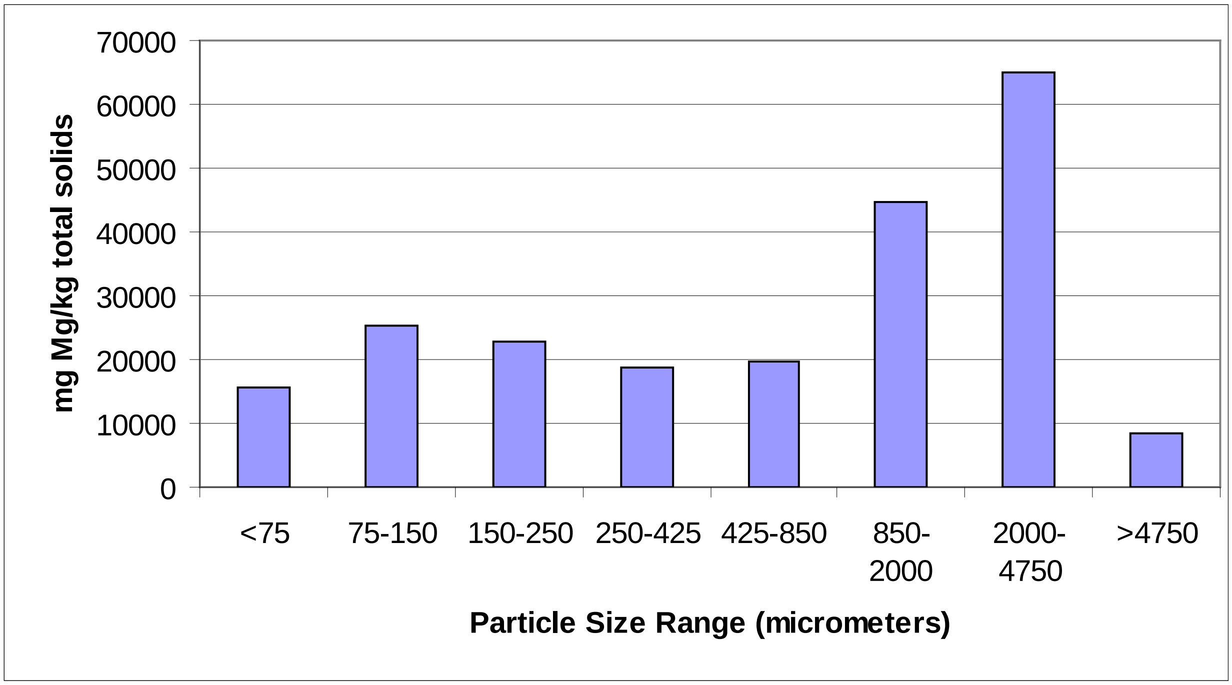
Calcium in Upflow Filter Sump

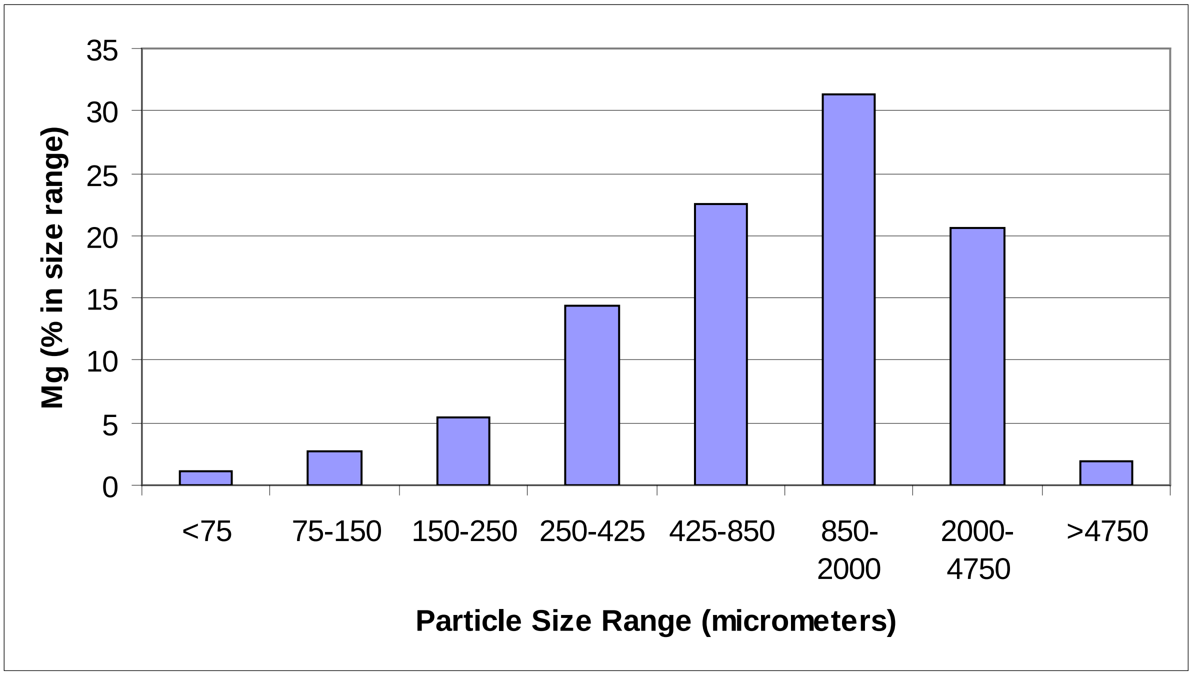



Ca (% in size range)
35
30
25
20
15
10
5
0
<75
75-150
150-250
250-425
425-850
850-2000
2000-4750
>4750
Particle Size Range (micrometers)

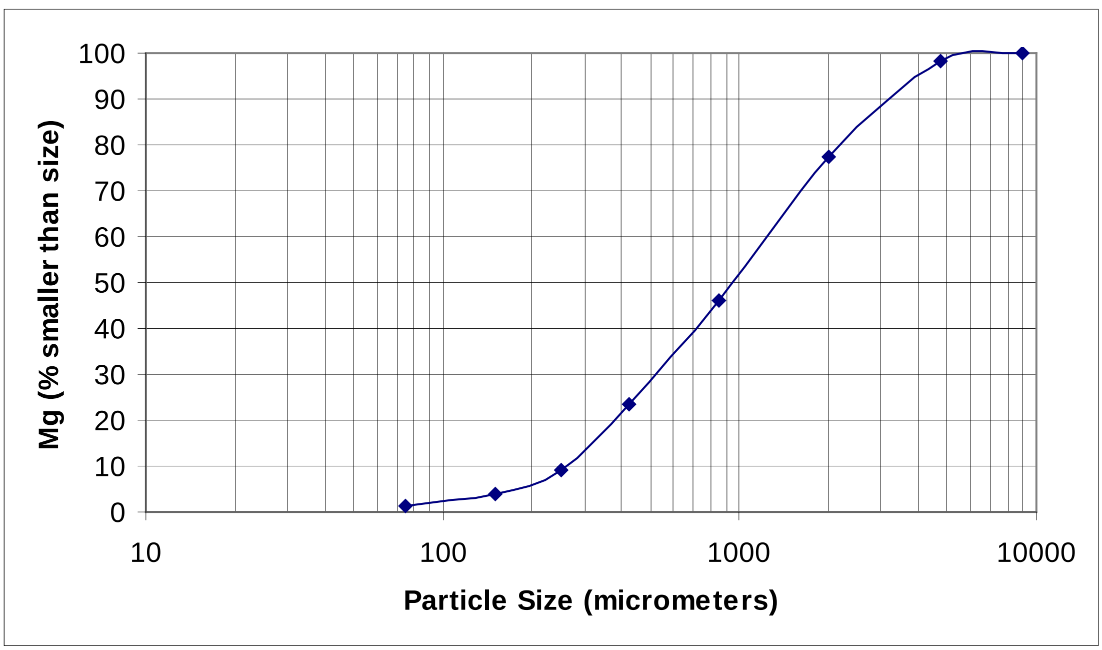

Ca (% smaller than size)
100
90
80
70
60
50
40
30
20
10
0
10
100
1000
10000
Particle Size (micrometers)

Magnesium in Upflow Filter Sump



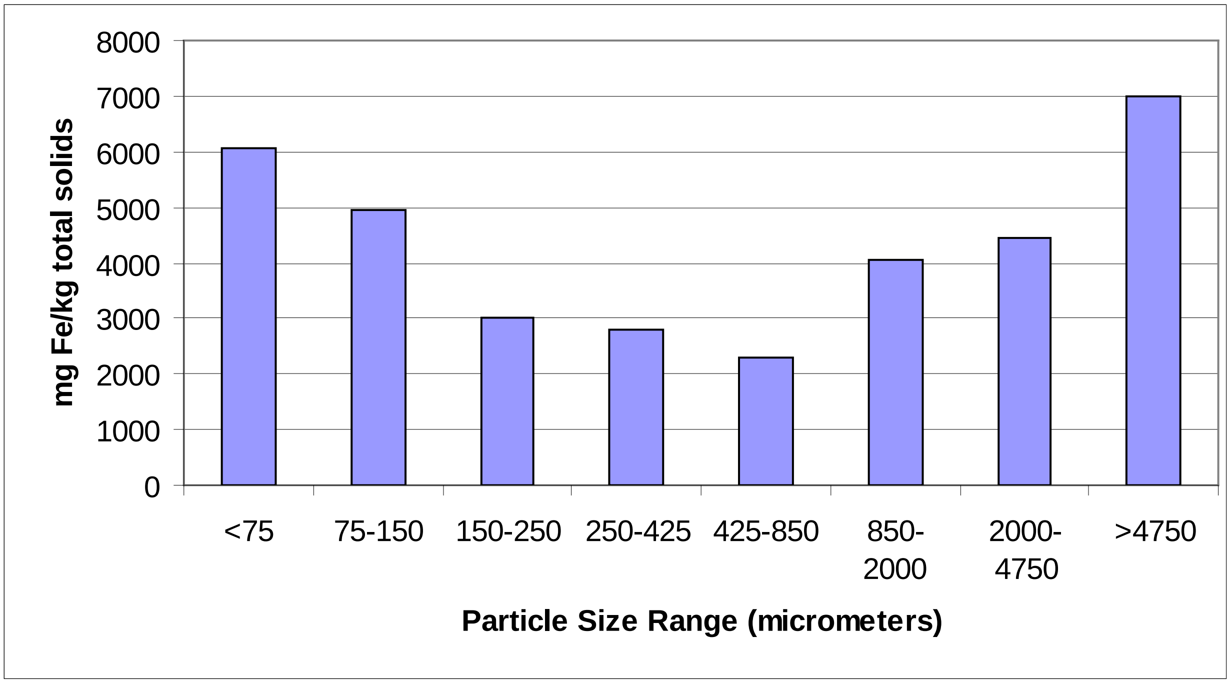


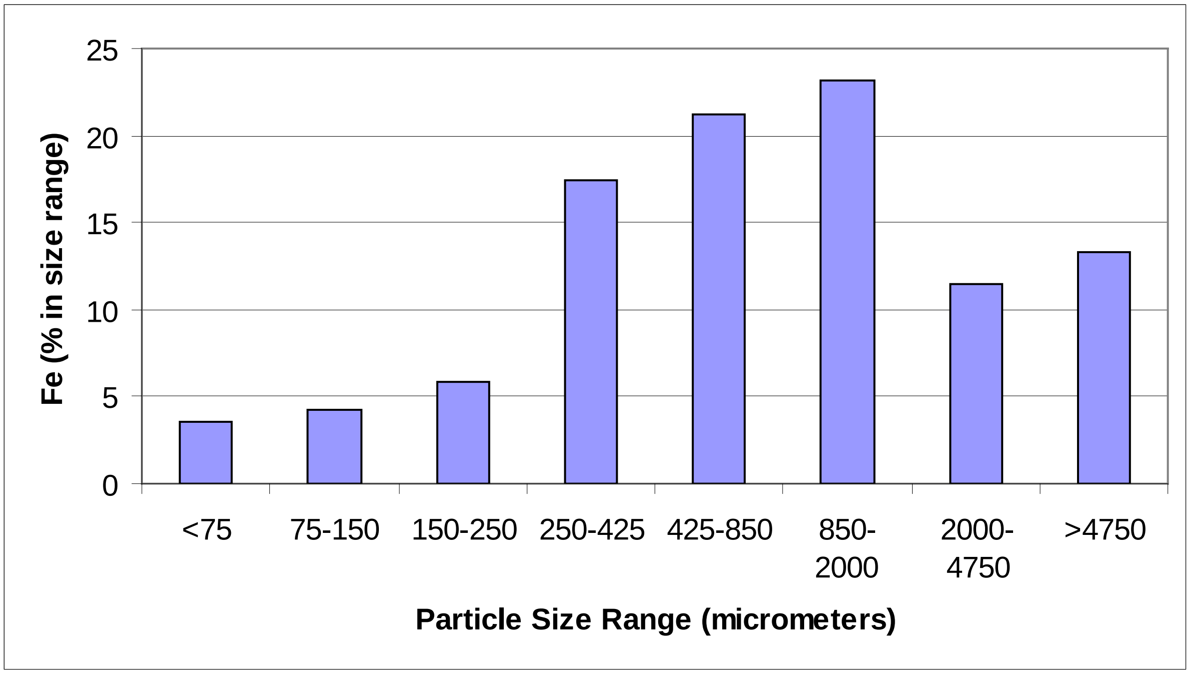
Mg (% in size range)
35
30
25
20
15
10
5
0
<75
75-150
150-250
250-425
425-850
850-2000
2000-4750
>4750
Particle Size Range (micrometers)



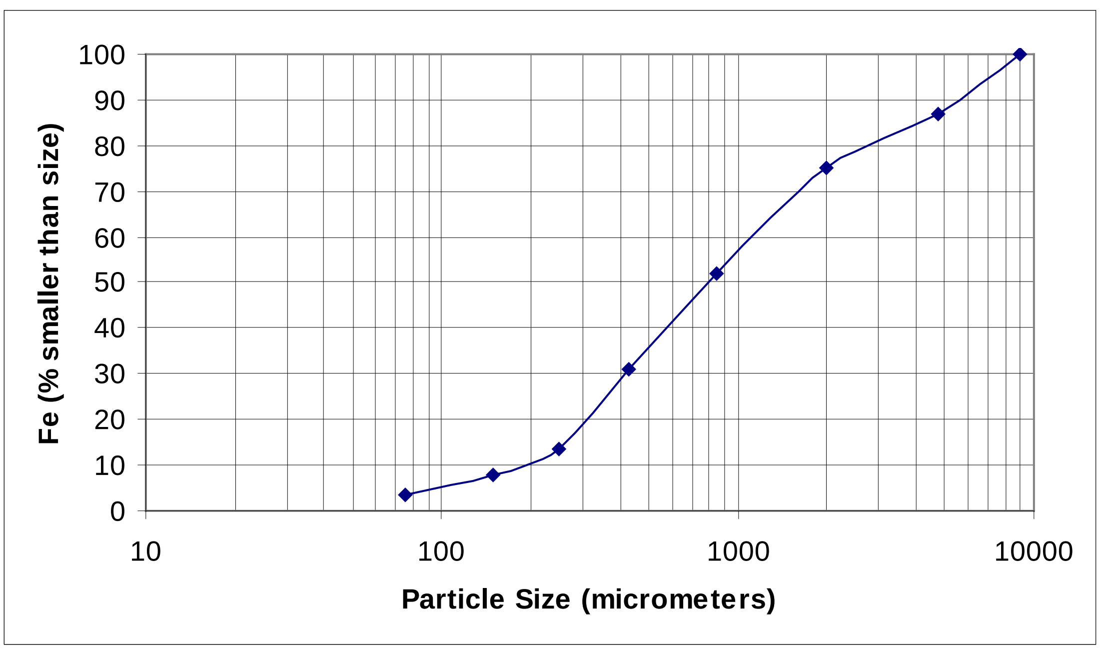
Mg (% smaller than size)
100
90
80
70
60
50
40
30
20
10
0
10
100
1000
10000
Particle Size (micrometers)

Iron in Upflow Filter Sump



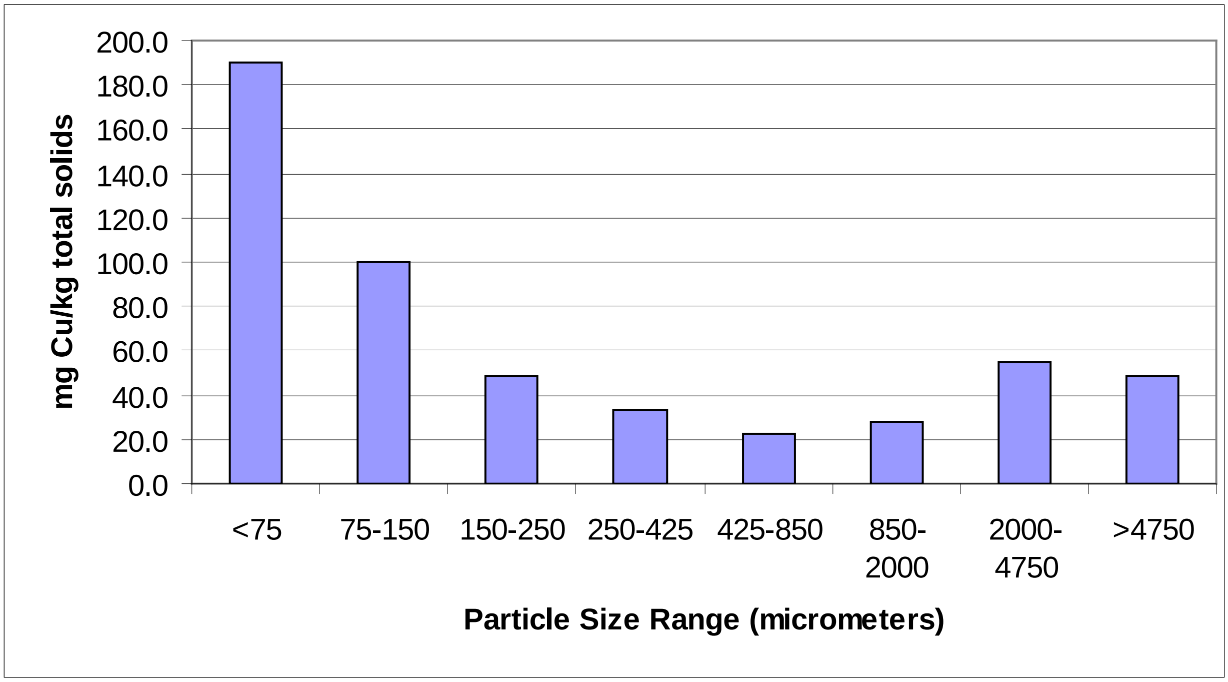


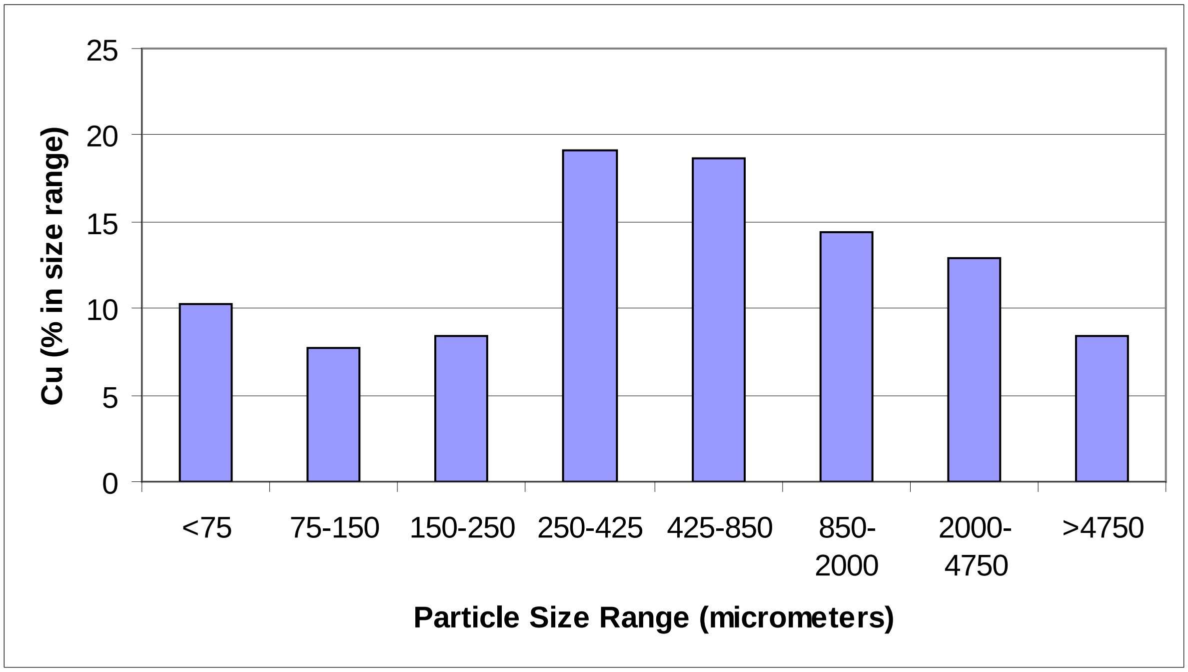
Fe (% in size range)
0
5
10
15
20
25
<75
75-150
150-250
250-425
425-850
850-2000
2000-4750
>4750
Particle Size Range (micrometers)

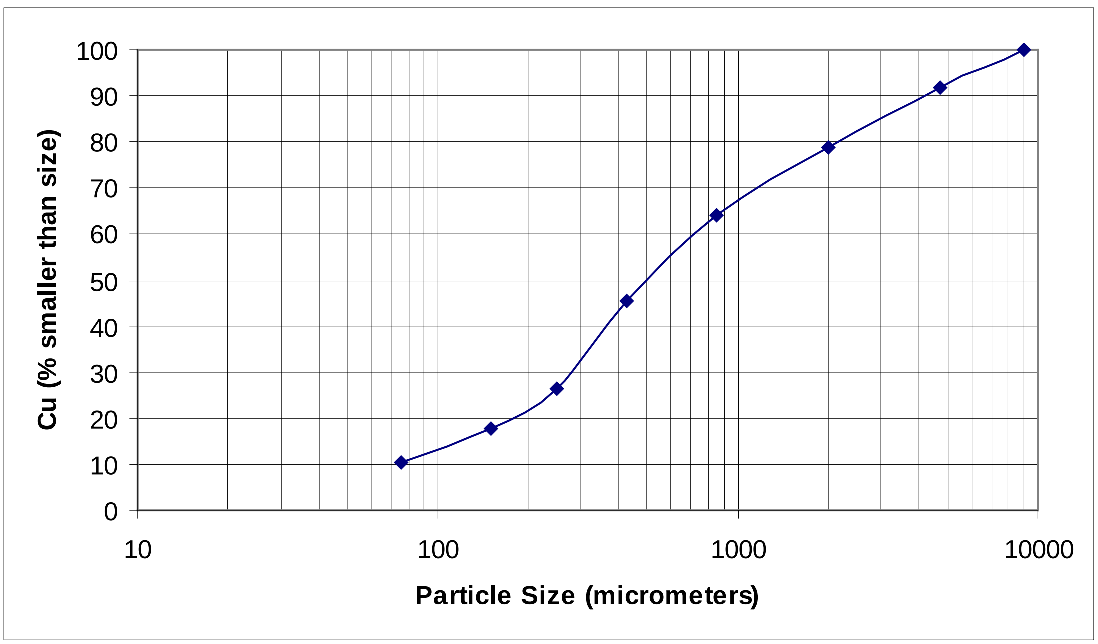


Fe (% smaller than size)
0
10
20
30
40
50
60
70
80
90
100
10
100
1000
10000
Particle Size (micrometers)

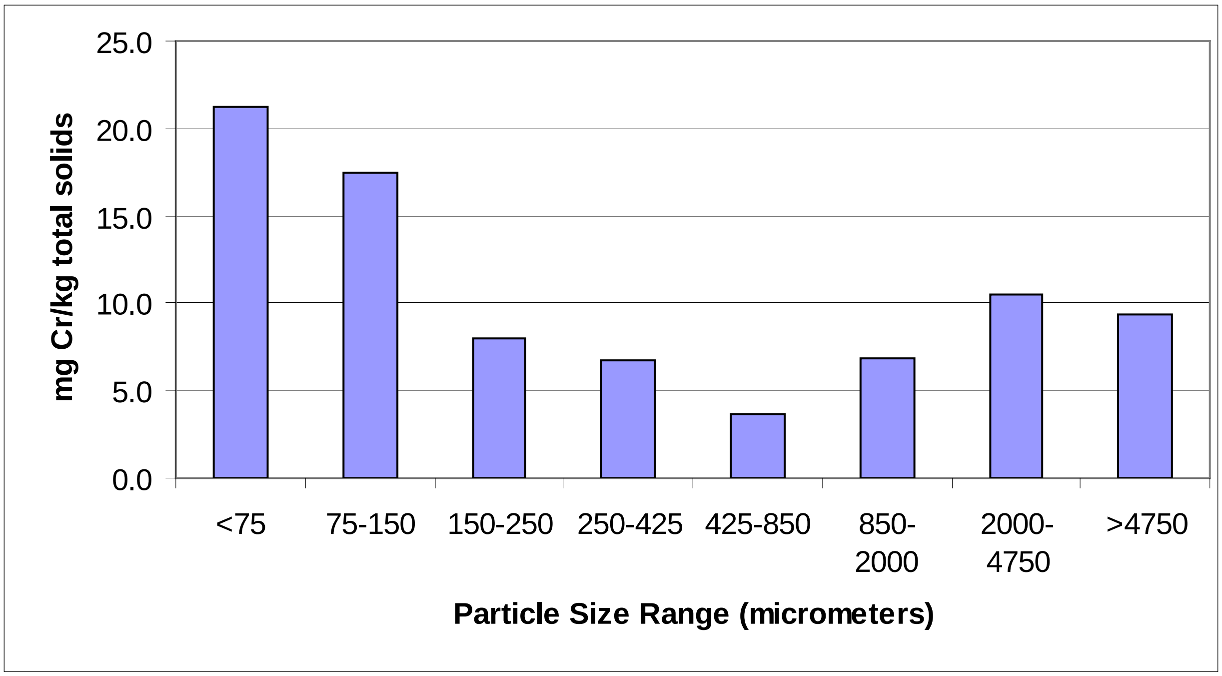
Copper in Upflow Filter Sump

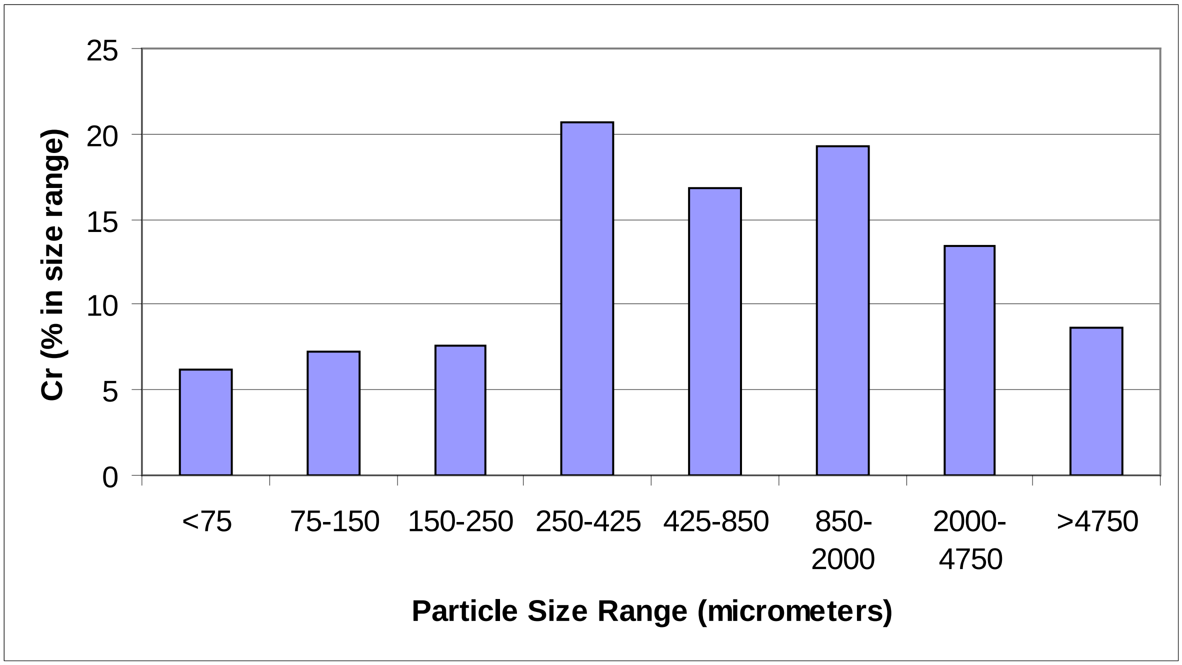


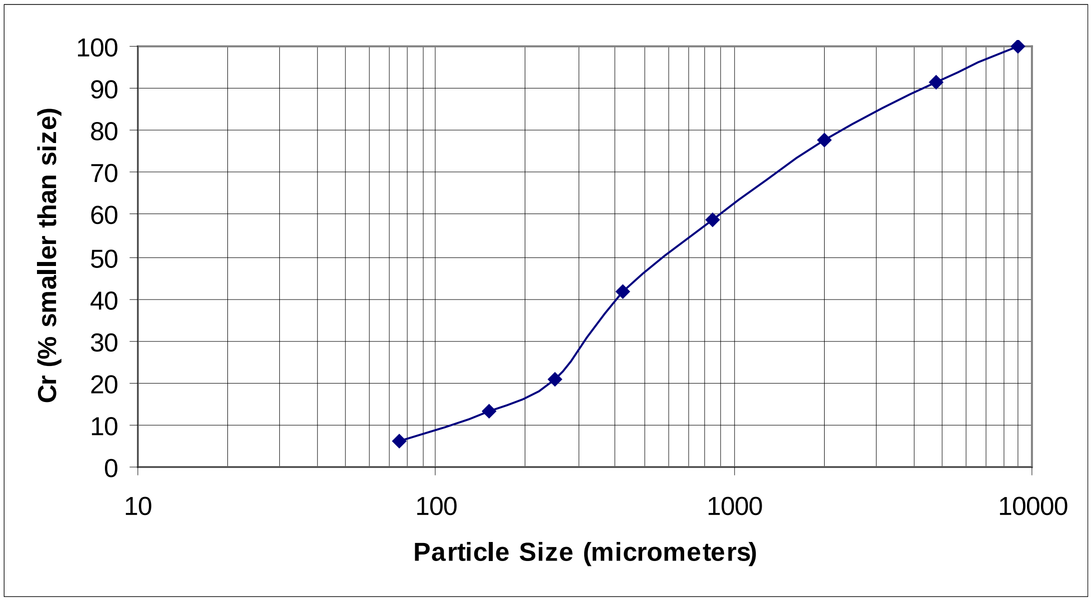

25
20
15
10
5
0
Cu (% in size range)
<75
75-150
150-250
250-425
425-850
850-2000
2000-4750
>4750
Particle Size Range (micrometers)


100
90
80
70
60
50
40
30
20
10
0
Cu (% smaller than size)
10
100
1000
10000
Particle Size (micrometers)

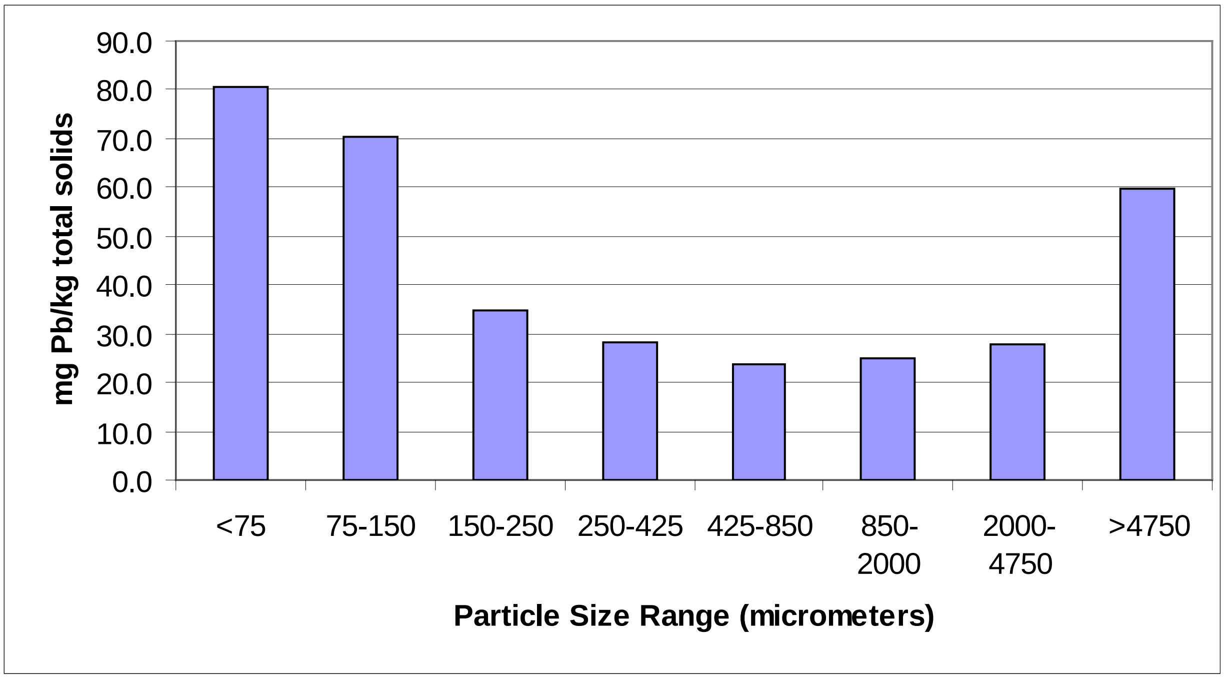
Chromium in Upflow Filter Sump

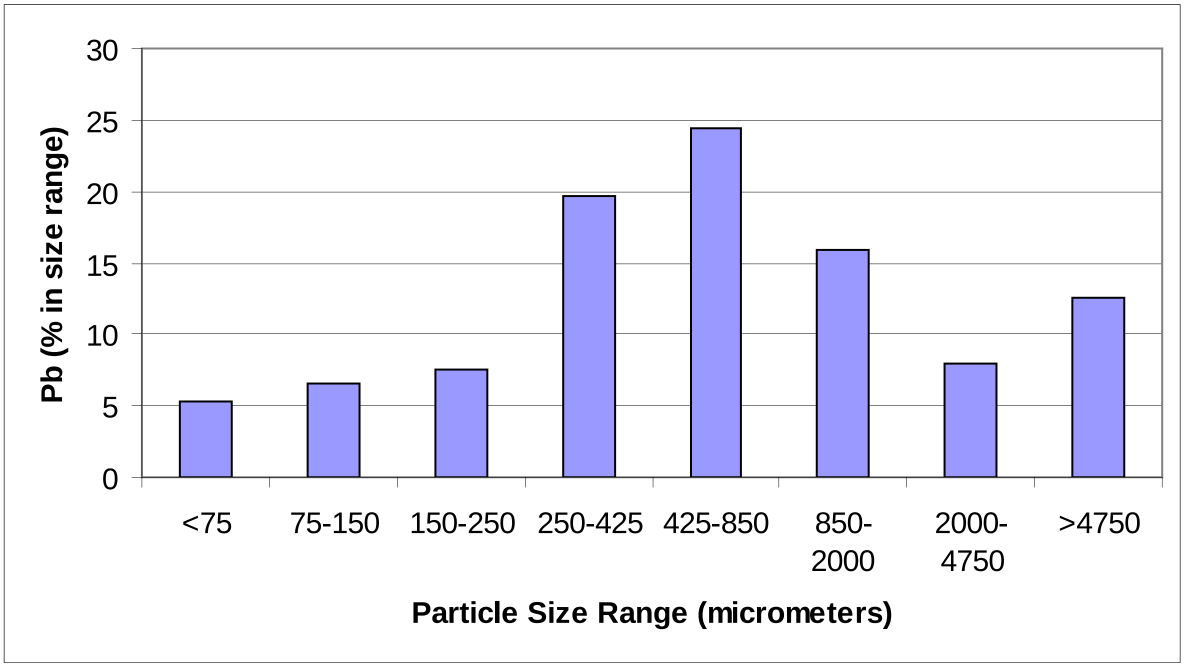



Cr (% in size range)
25
20
15
10
5
0
<75
75-150
150-250
250-425
425-850
850-2000
2000-4750
>4750
Particle Size Range (micrometers)

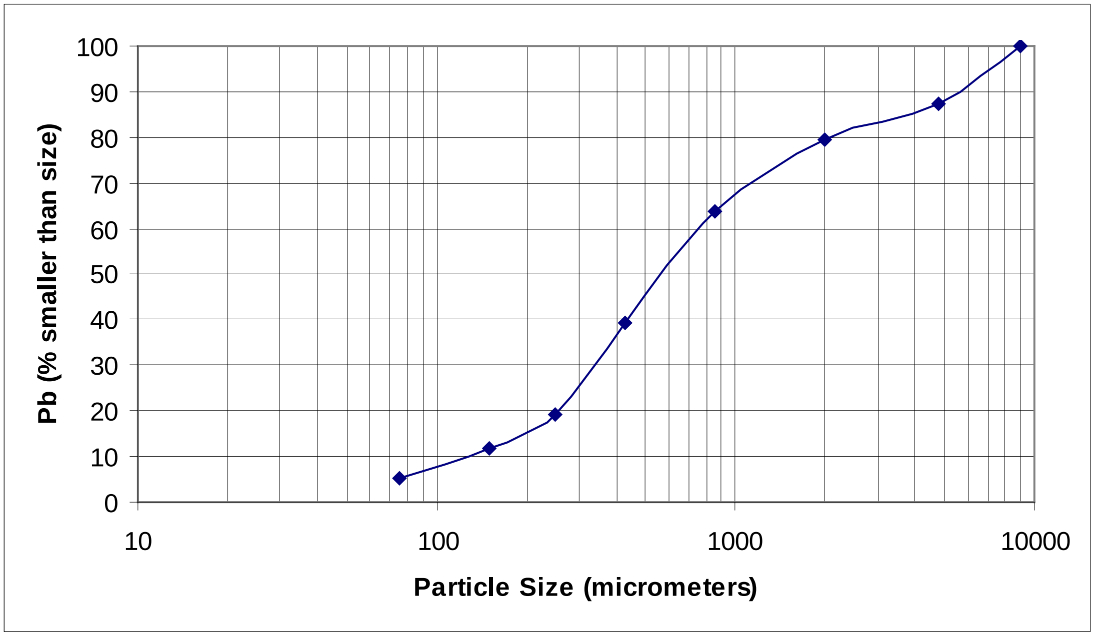

Cr (% smaller than size)
100
90
80
70
60
50
40
30
20
10
0
10
100
1000
10000
Particle Size (micrometers)

Lead in Upflow Filter Sump



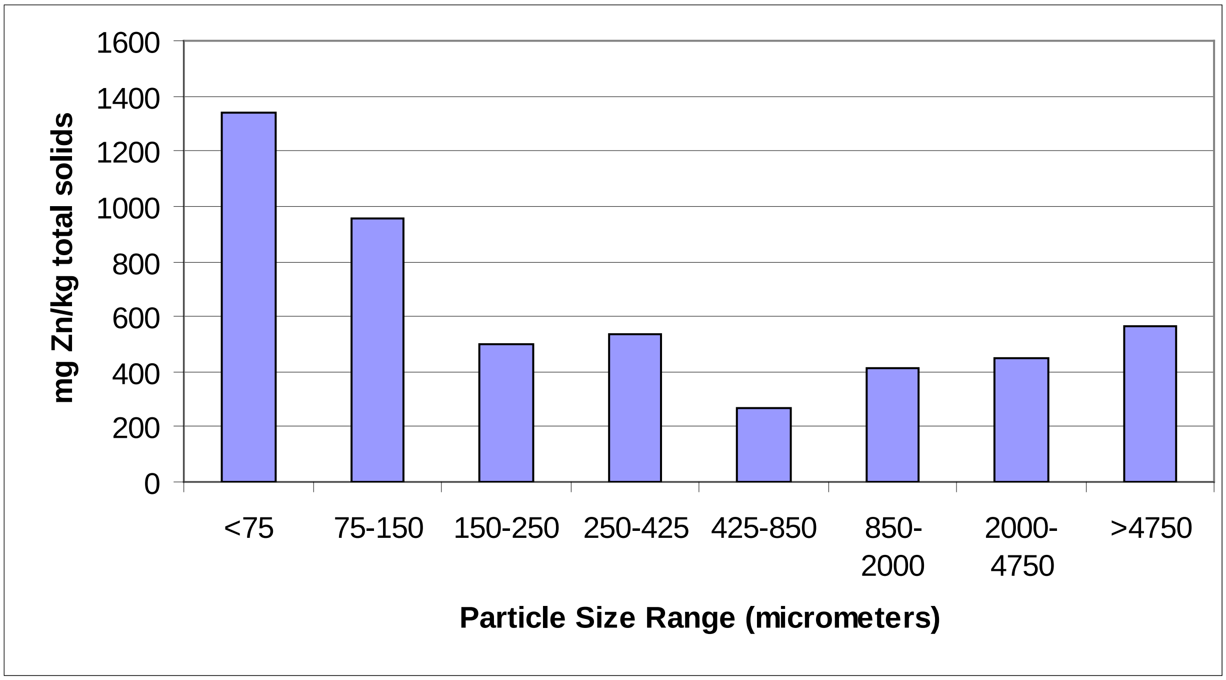


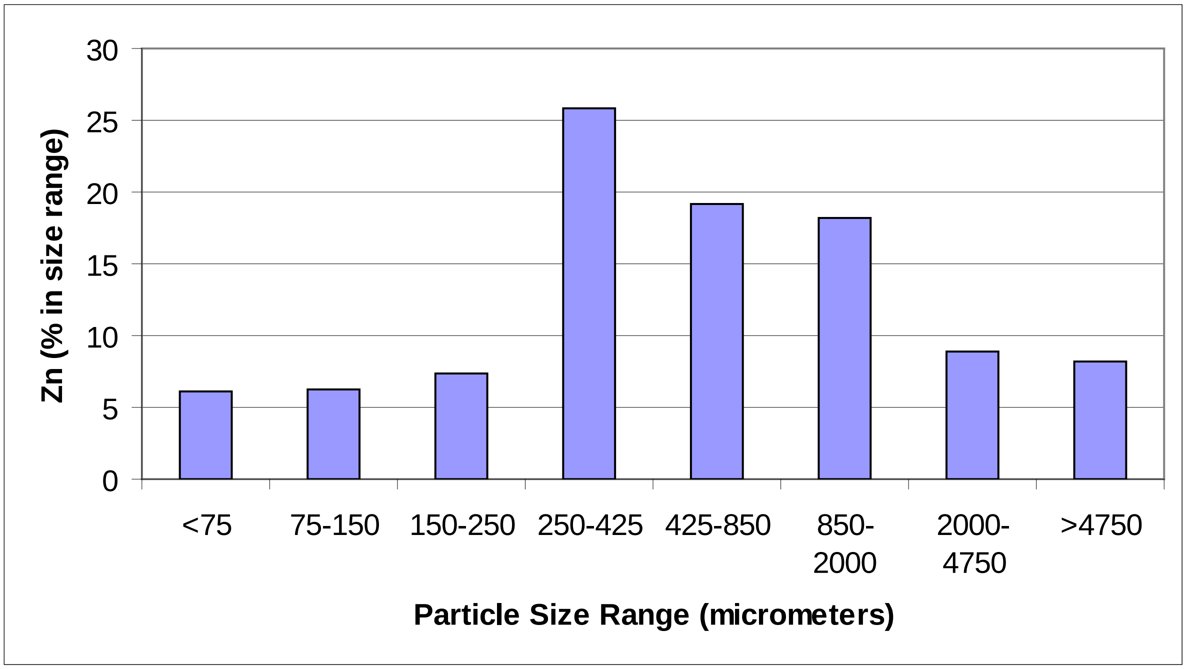
Pb (% in size range)
30
25
20
15
10
5
0
<75
75-150
150-250
250-425
425-850
850-2000
2000-4750
>4750
Particle Size Range (micrometers)



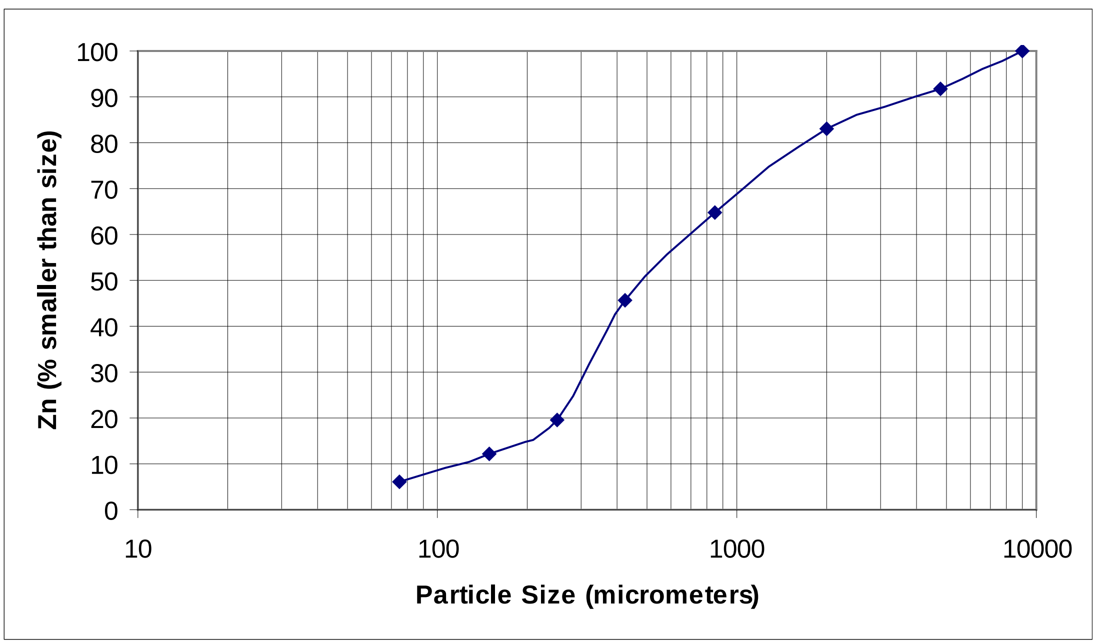
Pb (% smaller than size)
100
90
80
70
60
50
40
30
20
10
0
10
100
1000
10000
Particle Size (micrometers)

Zinc in Upflow Filter Sump





Zn (% in size range)
30
25
20
15
10
5
0
<75
75-150
150-250
250-425
425-850
850-2000
2000-4750
>4750
Particle Size Range (micrometers)



Zn (% smaller than size)
100
90
80
70
60
50
40
30
20
10
0
10
100
1000
10000
Particle Size (micrometers)