4076

THE EFFECTIVENESS OF A SLOW-SAND FILTER

AT A ROAD MAINTENANCE FACILITY

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We certify that we have read this thesis and that, in our opinion, it is satisfactory in scope and quality as a thesis for the degree of Master of Science in Civil Engineering.

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ABSTRACT

The Ada County Highway District (ACHD) in Boise , Idaho routinely treats and disposes of substantial amounts of storm water and related solids (sediment, leaves, and miscellaneous debris) collected by its street sweepers and vacuum trucks. In 2002 ACHD engaged the services of CH2M HILL to design a new water treatment / road waste disposal system at its Cloverdale maintenance yard in Boise. I served as the junior project engineer, providing design services, construction oversight, and the initial sampling of the treated water. This thesis covers the design and construction of the new facility and evaluates its performance. The primary focus is the overall efficiency of the slow sand filter used to treat the water at the facility.

Water discharged at the facility is treated and either reused for truck washing, street cleaning, or other road maintenance operations, or it is discharged from the facility to Evan's Drain, an irrigation canal adjacent to the Cloverdale maintenance yard.

Regulatory compliance was met through application of an effluent water quality monitoring plan. Water quality data was collected monthly after the facility began operation in May of 2003. This data collected is presented in this paper and has been used to accurately characterize the facility's effluent. Overall, the effluent water quality is shown to be sufficiently treated for reuse in road maintenance operations. Solids concentrations are relatively low alleviating concerns over reusing the water in maintenance equipment; e. coli levels are low easing some health and safety concerns; in general all pollutants monitored are acceptably removed in the system.

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CHAPTER 1
INTRODUCTION

The Ada County Highway District (ACHD) municipality in Boise, Idaho maintains public roads and highways throughout the 1,060 square mile county. The ACHD's primary functions are to provide planning, design, construction, reconstruction, maintenance and traffic supervision for all urban streets, rural roadways and bridges under its jurisdiction.

Each year, ACHD must dispose of thousands of cubic yards of water, sediment, leaves, and miscellaneous debris collected by its road sweepers and vacuum trucks. To properly handle this waste, ACHD engaged the services of CH2M HILL to design a new stormwater management facility at their Rural Maintenance Yard at 440 North Cloverdale Road. This facility is designed to treat all non-hazardous road-wastes collected by ACHD's sweepers and vacuum trucks. I served as the junior project engineer, providing design services, construction oversight, and the initial sampling of the treated water. This thesis will cover the design and construction of the new facility and evaluate its performance.

At the Cloverdale treatment facility, the road-wastes are first dumped onto a large concrete pad that is designed to drain excess water away from the solids. Once drained, the solids are removed from the pad and stockpiled on the ground adjacent to the concrete pad for eventual disposal. The pad is specially designed to drain all water to a 3,200 ft^2 settling basin that is divided into four cells. The water is slowly routed through each of the four cells to facilitate maximum settling of fine solids. Settled water is pumped from the fourth cell of the settling basin to a 2,800 ft^2 sand filter that is designed to remove residual oil, grease, nutrients, bacteria, and suspended sediments from the water. The sand filter then drains to a second sump where the clean water is pumped to an aboveground 17,000 gallon storage tank for reuse.

A major benefit of the treatment facility is that ACHD is able to incorporate water recycling into the disposal and treatment of the road-wastes. To recycle the water, the treated water in the 17,000 gallon storage tank is utilized as either rinse water for the street sweepers and vacuum trucks or as a fresh water supply for filling the storage tanks on ACHD's sweeper, vacuum, and water trucks; thus greatly reducing the need for use of the City's water supply. All recycled water that is used to rinse the trucks drains back to the stilling basin for retreatment. During wet weather when total collected water may exceed recycled water use, excess treated water is drained to the Evans Drain which is owned and operated by the Nampa and Meridian Irrigation District (NMID).

The goal of this project was to create a facility that was easy to operate and maintain and one that was environmentally sound, alleviating the need for use of the City sewer and reducing the dependence on the City water supply. One of the primary challenges met during the design phase was understanding and complying with local and federal regulations.

First, water reuse in the state of Idaho. Water reuse regulations are administered by the Idaho Department of Environmental Quality (IDEQ) in the state of Idaho. Only the reuse of wastewater, municipal and industrial, is regulated. The water being treated at the Cloverdale Facility does not fall into either of these categories and is considered storm water by local regulators and therefore does not fall under these regulations. Only NPDES regulations are applicable here. (See next paragraph).

Second, regulations for discharging treated water to surface waters (i.e. Evans Drain). A license agreement between ACHD and NMID was established permitting the discharge to Evans Drain. The license limits discharge to not more than .16 cubic feet per second, and requires water quality monitoring in compliance with ACHD's existing National Pollutant Discharge Elimination System (NPDES) permit. NPDES requirements for the ACHD at this time are to monitor pollutants during throughout the calendar year and to report on the findings in order to accurately characterize the quality and quantity of pollutants discharged. There are currently no numerical limits set on any particular pollutants.

This thesis focuses primarily on the effectiveness of the sand filter at the Cloverdale Facility, analyzing its design as well as the influent and effluent water quality parameters. These results are compared to those of sand filters used in other areas of the country as well as in other places around the world.

CHAPTER 2
LITERATURE REVIEW

02.01 Introduction to Slow Sand Filtration

Water filtration has been used for centuries, some might even argue for millennia, to purify water for drinking. The first recorded use, however, of the slow sand filter occurred in 1804, when Mr. John Gibb designed and built a slow sand filter to provide clean water to his bleachery in Paisley, Scotland. His filter was so successful that Gibb was able to provide the entire town of Paisley as well as the neighboring town of Glasgow with clean water. The success of the sand filter for drinking water purification led to more widespread use and more intense scrutiny from the scientific community. Today there exists a plethora of information on sand filter effectiveness for purification of drinking water.

A slow sand filter utilizes a large surface area to filter a relatively "slow" flow of water as defined by its loading rate (flow per square foot of area). In contrast, a rapid sand filter works with a much smaller surface area and a much higher loading rate. Intermittent sand filters are essentially slow sand filters used in wastewater treatment. For comparison purposes in this paper – all data available on sand filtration will be used to measure the efficiency of the slow sand filter employed at the Cloverdale site. To date, most data available is on rapid sand filters; rapid sand filters have been a more popular choice, more often than not, because of their small land space requirements.

Sand filters are commonly used to reduce quantities of contaminants such as suspended solids, turbidity and microorganisms. Microorganism removal is accomplished through the presence of what is termed the *Schmutzdecke*. According to

Wikipedia online dictionary, *Schmutzdecke* is German for 'grime or filth cover', and is a biological layer consisting of bacteria, fungi, protozoa, as well as a range of aquatic insect larvae. This layer of a sand filter is critical for treatment in potable water systems since the microorganisms present can cause disease.

"Slow sand filters were built to serve communities in North America both before and after 1900, but the advent of effective coagulation, sedimentation, and rapid rate filtration resulted in a declining interest in slow sand filtration in North America in the early part of the twentieth century. This situation changed during the latter part of the twentieth century when slow sand filtration was evaluated for removal of viruses, Giardia cysts, and Cryptosporidium oocysts... microorganisms unknown in the 1800s and early 1900s (Logsdon et al. 2002)."

The use of sand filtration in general (slow, rapid, and intermittent) is now is becoming a popular option in non-potable urban storm water best management practices. Sand filters are encouraged by the United States Environmental Protection Agency (EPA) for use in removing pollutants from storm water runoff (EPA 1999).

02.01.01 Process Theory – How it works

In an EPA Technology Assessment, Damann Anderson et al. (1985) describe the process theory behind sand filtration, reporting that:

Contaminants are removed from the influent water through the processes of "straining, sedimentation, inertial impaction, interception, adhesion, flocculation, diffusion, adsorption and biological activity...

- Straining involves a mechanical sieve action as well as a lodging of particles in crevices.
- Sedimentation occurs as gravity settling takes place in the interstices of the media.
- Inertial impaction, interception, and adhesion occur as particles moving through the filter strike media granules and are removed.
- Particles moving through the pores will also collide and flocculate causing subsequent removal by other mechanisms.
- Diffusion is important in the removal of very small particles such as viruses, and occurs because of the small interstices in porous media and the fact that laminar flow exists.
- Physical adsorption of pollutants takes place on media surfaces due to electrostatic, electrokinetic, and van der Waals forces while chemical adsorption occurs due to bonding and chemical interaction between wastewater constituents and the filter media.
- Biological activity on the filter media results in removal of polluting materials by biological assimilation and biosynthesis" (Anderson et al. 1985).

Biological processes play an important role in the successful functioning of slow sand filters, especially in the removal of turbidity and biological particles. "The biological action is first encountered at the surface of a slow sand filter in the slimy surface referred to as the *Schmutzdecke* (Logsdon et al. 2002)."

02.01.02 Operation and Maintenance

In 1999 the EPA published a Stormwater Technology Fact Sheet on Sand Filters. The Fact Sheet presents basic operation and maintenance recommendations to ensure design level performance is achieved. The fact sheet states that, "Sand filters should be inspected after all storm events to verify that they are working as intended... Typically, sand filters begin to experience clogging problems within 3 to 5 years...A record should be kept of dewatering times... to determine if maintenance is necessary (EPA 1999)."

According to an article in the *Journal of Environmental Engineering and Science* by Gary Logsdon et al. (2002), "Slow sand filtration performs best when the filtration rate is constant, so frequent rate increases must be avoided. Especially to be avoided is the opening and closing of effluent valves on a frequent basis to maintain a desired water production rate... Stopping and starting a slow sand filter may seriously impair filtrate quality."

Logsdon (2002) continues, "Routine maintenance... is not complicated. When terminal head loss develops [due to excessive clogging], slow sand filters are scraped to remove the *Schmutzdecke* and 1 to 2 cm of sand and thus restore the filtration capacity."

02.02 Water Reuse / Alternative Sources of Water

The stormwater management facility analyzed in this thesis makes use of water treatment sufficient to reuse all of the water deposited and collected at the site. The site was designed with no connection to the sanitary sewer system. The reuse of treated storm and domestic waste water is a growing trend worldwide (Smith et al. 1995).

Across the globe increasing water demands due to the population growth have prompted municipalities to respond by investigating alternative sources of non-potable water. Water used for irrigation, vehicle washing, and street cleaning are the most common uses targeted for supply through such alternate sources (Smith et al. 1995).

The United Kingdom has implemented the use of treated wastewater effluent as an alternate source of non-potable water. In the article, *Innovative Treatment Technologies for Non-Potable Wastewater Reuse*, A.J. Smith et al. (1995) note that "The prime requirement of any wastewater effluent reuse program is to ensure that the application does not compromise public health or affect the environment. With the potential impact on public health there is an element of risk associated with effluent reuse. In order to minimize this risk we must define both the quality standard and the treatment requirement appropriate to the final use. The cost of achieving the required quality must be compared with the costs associated with other options for augmenting water resources. In general terms, the higher the standards, the lower the risk, and the higher the cost."

Colorado Springs Street Division's stormwater catch basin cleaning operations reportedly result in approximately 2,500 cubic yards of solid material and 50,000 gallons (gal) of water annually that require disposal (King 1996).

To address this situation in an environmentally responsible and friendly way, Division employees designed and constructed a dewatering facility. The facility is a 20-foot by 50-foot by 4-foot deep structure which "provides space for trucks to dump their wastes in to a containment area... the water flows through a series of two 2,500 gal enclosed concrete tanks allowing solids to settle to the bottom. The water exiting the final tank has been tested and found not to be hazardous. The water is pumped to a system of sprinklers that spray adjoining compost piles... Solids left behind are screened and placed in the compost piles for later use (King 1996)."

After eight months of operation the facility was effectively maintaining an exceptional efficiency rate by continuing to recycle 100% of catch basin operation "wastes."

02..03 Sand Filter Design Parameters

A slow sand filter utilizes a large surface area to filter a relatively "slow" flow of water, hence the term *slow* sand filter. Typical application rates range from 0.015 gallons per minute of flow per square foot of sand filter surface area (gpm/ft$^{2)}$ to 0.16 gpm/ft^2. The applied water flows by gravity through a layer of sand (typically 3 or 4 feet) and a support layer of gravel and is then collected by an underdrain system and transferred to a storage area.

Further, "Slow sand filters have traditionally been designed with a bed of sand about 1 meter in depth... the effective size (D_{10}) of filter sand ranges from 0.15 to 0.35 mm... the uniformity coefficient (D_{60}/D_{10}) should be less than 5, preferably less than 3. Filtration rates are typically in the range of 0.1 to 0.3 m/h [meters per hour] (Logsdon et al. 2002)."

The filter media in the intermittent sand filters reported in Anderson's assessment are exclusively sand of medium to very coarse grain size (0.25 – 2.00 mm). This is similar to the sand used at the Cloverdale site (the subject of this paper) conforming to ASTM C-33, which is typically fine sand on the order of less than 5 mm in diameter. The intermittent sand filters were used to treat domestic wastewater; however, it seemed useful to show the results for contaminant removal in comparison to results (for treating urban stormwater) obtained at the Cloverdale site to show overall the capabilities of the slow sand filter with only sand used as filter media.

Barrett's article on Austin sand filters (rapid sand filters) provided the following information on the sand used in that study; "The sand used in the Los Angeles and San Diego filters had slightly different characteristics [from eachother]. In Los Angeles, the diameter of the median particle size (D_{50}) was about 0.6 mm with a coefficient of uniformity (C_u) of 2.1. The San Diego filters contained sand with a slightly finer particle (D_{50} = 0.4 mm) which was better sorted (C_u = 1.6). These media have a median sand size that is comparable to the ASTM C33 concrete sand required by the City of Austin, but better sorted (Barrett 2003)."

02.04 Sand Filter Performance - Data Analysis

In Barrett's study of Austin sand filters in Los Angeles and San Diego the results were carefully analyzed by different methodologies and some unique and interesting conclusions were drawn.

"When performance is expressed as a percent reduction in load or concentration, a relationship between the concentrations of treated and untreated runoff is implied (Barrett 2003)

However, this is not always the case, and further examination revealed the discovery of a significant relationship concerning the performance of the sand filter. For certain parameters, such as Total Suspended Solids (TSS) and Total Metal concentrations, the effluent concentration remained relatively constant regardless of influent concentration.

In these cases the effluent quality was not directly tied to the influent quality, thus the performance could not be directly measured by using the percent of pollutant removal. Rather the efficiency of the sand filter should be directly related to expected effluent water quality - not the expected percent removal.

02.05 Sand Filter Performance - Study Results

02.05.01 Sand Filters and Storm Water Quality Enhancement

A general range of field performance measures is presented in Urbonas's article, *Design of a Sand Filter for Storm Water Quality Enhancement*. These parameters are shown in the following table taken directly from the Urbonas (1999) article.

The expected performance for sand filter removal rates is provided here for the following contaminants: TSS, Total Phosphorous (TP), Total Nitrogen (TN), Total Kjeldahl Nitrogen (TKN), Total Copper (TC_u), and Total Zinc (TZ_n).

TABLE 2.1 (Urbonas 1999)

Field Measured Performance Ranges of Sand Filters

Constituent	In/out	Concentration (mg/L) Low	High	Mean	Percent Removed Low	High	MCR*
TSS	In	12	884	160			
	Out	4	40	16	67%	95%	80 - 90%
TP	In	0.05	1.4	0.52			
	Out	0.035	0.014	0.11	30%	99%	50 - 75%
TN	In	2.4	30	8			
	Out	1.6	8.2	3.8	33%	73%	30 - 50%
TKN	In	0.4	28	3.8			
	Out	0.2	2.9	1.1	50%	90%	60 - 75%
Total Copper	In	0.03	0.135	0.06			
	Out	0.016	0.035	0.025	47%	74%	20 - 40%
Total Zinc	In	0.04	0.89	0.2			
	Out	0.008	0.059	0.033	80%	93%	80 - 90%

* MCR = most common data range

The 1999 EPA Fact Sheet reports on three main sand filter designs, all of which are the rapid sand filter type (the Austin sand filter, the Washington, DC sand filter, and the Delaware sand filter). The Fact Sheet states that "Sand filters are able to achieve high removal efficiencies for sediment, biochemical oxygen demand (BOD), and fecal coliform bacteria. Total metal removal, however, is moderate, and nutrient removal is often low (EPA 1999)."

The following table shows percent removals of the common pollutants [Fecal coliforms, BOD, TSS, Total Organic Carbon (TOC), TN, TKN, Nitrate-nitrogen (NO_3-N), TP, Iron (Fe), Lead (Pb), and Zinc (Zn)]. The results given are average values for various sand filters serving drainage areas of several different sizes in the Austin area (EPA 1999).

TABLE 2.2 (EPA 1999)

Typical Pollutant Removal Efficiency

Pollutant	Percent Removal
Fecal Coliform	76
Biochemical Oxygen Demand (BOD)	70
Total Suspended Solids (TSS)	70
Total Organic Carbon (TOC)	48
Total Nitrogen (TN)	21
Total Kjeldahl Nitrogen (TKN)	46
Nitrate as Nitrogen (NO_3 - N)	0
Total Phosphorous (TP)	33
Iron (Fe)	45
Lead (Pb)	45
Zinc (Zn)	45

02.05.02 Sand Filters as Storm Water Best Management Practices

Chris Dunn, et al. (1995) presents a comparison of several types of best management practices (BMPs) for urban storm water treatment and their respective efficiencies. The results presented in the article, *Current Water Quality Best Management Practices Design Guidance,* for sand filters utilized in Austin, Texas are shown in the following table excerpted from Table 1 (Pollutant Removal Comparison for Various Urban BMP Designs) of Dunn's article.

The following table shows average percent removals for suspended sediment, TP, TN, trace metals, and bacteria expected from Austin sand filters using a non-peat media (i.e. sand only).

TABLE 2.3 (Dunn et al. 1995)

Sand Filters - Austin, Texas (Non-Peat)	
Pollutant	**Pollutant Removal Efficiency (%)**
Suspended Sediment	85
Total Phosphorous	40
Total Nitrogen	35
Oxygen Demand	Unknown
Trace Metals	50 - 70
Bacteria	40
Overall Removal Capability	Moderate

02.05.03 Performance Results of Austin Sand Filters

The results from Barrett's study are presented by giving both the influent and effluent event mean concentrations (EMCs) as well as the percent removal for the following pollutants:

TSS, NO_3-N, TN, Ortho-phosphate, TP; dissolved Cu, Pb and Zn; total Cu, Pb and Zn; Total Petroleum Hydrocarbons (TPH); and fecal coliforms.

The results of Barrett's study are shown in the following table:

TABLE 2.4 (Barrett 2003)

Performance of Austin Sand Filters

Constituent	Average influent EMC[a]	Average effluent EMC[a]	Reduction (%)
Total Suspended Solids (TSS) (mg/L)	90	8.6	90
Nitrate - N (mg/L)	0.63	1.1	-74
TKN (mg/L)	3.02	1.48	51
N total (mg/L)	3.72	2.91	22
Ortho-Phosphate (mg/L)	0.17	0.16	6
P total (mg/L)	0.41	0.25	39
Cu dissolved (μg/L)	8.9	8.4	6
Pb dissolved (μg/L)	2	<1	39
Zn dissolved (μg/L)	94	36	62
Cu total (μg/L)	21	10	50
Pb total (μg/L)	21	3	87
Zn total (μg/L)	236	48	80
Total Petroleum Hydrocarbons (TPH) - oil (mg/L)[b]	1	0.7	30
Total Petroleum Hydrocarbons (TPH) - diesel (mg/L)[b]	0.8	0.6	25
Fecal coliform[b] (MPN/100 mL)	11200	3900	65

[a]Event Mean Concentration
[b]TPH and coliform are collected by grab method and may not accurately relect removal.

Barrett (2003) notes that, "The data indicate that modest removal of TN does occur, or some conversion of ammonia nitrogen (NH_4-N) or organic nitrogen to nitrate must be occurring in the filter bed. TN concentrations are calculated as the sum of NO_3 and TKN."

"The distinction between a constant effluent quality and a percent reduction is extremely important to recognize, if the results are to be used to estimate effluent quality from sand filters installed at other sites with different influent concentrations or for estimating compliance with water quality standards for storms with high concentrations of particulate constituents. If the conventionally derived removal efficiency (90%) were used to estimate the TSS concentrations in the treated runoff from storms with high influent concentrations, the estimated effluent concentration would be too high (Barrett 2003)." The expected TSS removal should instead be reported by the expected effluent concentration - in this case 7.8 mg/L (+/- 1.2 mg/L).

"Sand filters are generally expected to have limited removal ability for dissolved constituents, yet for the dissolved copper and other metals these data indicate significant reduction in concentration when the influent concentrations were sufficiently high (Barrett 2003)." Again, it is recommended that percent removal rates not be used to predict sand filter performance when discussing dissolved metals - rather, examination of a somewhat constant effluent quality can be expected regardless of influent concentrations.

02.05.04 Intermittent Sand Filters and Domestic Wastewater Treatment

The intermittent sand filters discussed in Anderson's assessment were all used to treat domestic wastewater, however, their effectiveness in contaminant removal is relevant here because it shows typical capabilities of slow sand filtration.

The following table presents data showing BOD_5, TSS, NH_4-N, NO_3-N, and Fecal coliform (FC) removals versus filter loading rates (and corresponding flow rates). The results are illustrated in the table below:

TABLE 2.5 (Anderson et al. 1985)

Type	Actual Flow (gpd)	Filter Loading (gpd/ft^2)	Influent / Effluent Quality: BOD_5 (mg/L) In	BOD_5 (mg/L) Out	TSS (mg/L) In	TSS (mg/L) Out	NH_4-N (mg-N/L) In	NH_4-N (mg-N/L) Out	NO_3-N (mg-N/L) In	NO_3-N (mg-N/L) Out	P (mg-P/L) In	P (mg-P/L) Out	FC (Log #/L) In	FC (Log #/L) Out	Data Period
RSF[a]	30000	2.7	218	7	79	7	27.9	4.8	1	27	13.4	8.9	7.1	5.7	3/77 - 10/77
RSF	20000	5	48	2	38	11									1/83 - 9/83
ISF[b]	70000	3	148	4	62	5	22.4	0.7	0.7	24.4	8	7.2	7.2	5.5	3/77 - 10/77
ISF	45000	1 1		10		10									12/82 - 11/83
RSF				10		12		1.5 - 4						3.6	
ISF	22000	1.7		10 - 30		10 - 30								5.2	
ISF	70000	13.5	30	11	10	2									2/83 - 4/83

[a] RSF = Rapid Sand Filter
[b] ISF = Intermittent Sand Filter

02.05.05 *Slow Sand Filtration and Microorganism Removal*

Logsdon's (2002) research focused primarily on microorganism removal. Removal percentages are given for the following microorganisms; poliovirus, total coliform bacteria, *Giardia*, and *Cryptosporidium* oocysts. The results are presented in the following table:

TABLE 2.6 (Logsdon et al. 2002)

Microorganism Removal by Slow Sand Filtration

Reference	Organism	Filtration Rate (m/h)	Temperature (°C)	Removal Percentage
Poynter and Slade (1977)	Poliovirus	0.2	16 to 18	99.997 average
Poynter and Slade (1977)	Poliovirus	0.4	16 to 18	99.865 average
Poynter and Slade (1977)	Poliovirus	0.2	5 to 8	99.68 average
Poynter and Slade (1977)	Total coliform bacteria	0.5	5 to 8	98.25 average
Bellamy et al. (1985*b*)	Total coliform bacteria	0.12	17	97 average
Bellamy et al. (1985*b*)	*Giardia*	0.12	5	87 average
Bellamy et al. (1985*a*)	*Giardia*	0.12	5 to 15	99.994 average
Bellamy et al. (1985*a*)	*Giardia*	0.4	5 to 15	99.981 average
Bellamy et al. (1985*b*)	*Giardia*	0.12	17	>99.93 to >99.99
Bellamy et al. (1985*b*)	*Giardia*	0.12	5	>99.92 to >99.99
Pyper (1985)	*Giardia*	0.08	0.5	93.7
Pyper (1985)	*Giardia*	0.08	0.5 to 0.75	99.36 to 99.91
Pyper (1985)	*Giardia*	0.08	7.5 to 21	99.98 to 99.99
Ghosh et al. (1989)	*Giardia*	0.3	4.5 to 16.5	>99.99
Ghosh et al. (1989)	*Giardia*	0.4	4.5 to 16.5	99.83 to 99.99
Ghosh et al. (1989)	*Cryptosporidium* oocysts	0.15 to 0.40	4.5 to 16.5	>99.99
Hall et al. (1994)	*Cryptosporidium* oocysts	0.2	Not stated	99.8 to 99.99
EES and TWU (1996[a])	*Cryptosporidium* oocysts	0.29	12 to 14	>99.99

[a]Economic and Engineering Services, Inc. and Thames Water Utilities. 1996. Salem slow sand filtration pilot study microbiological challenge test results. Unpublished report.

"The biological condition of the sand bed is very important, as removal is more effective when the biota have become established in a ripened bed. Fresh sand in a newly built filter is not very effective (Logsdon et al. 2002)."

Additionally, the following two points are noted as important process characteristics: "cold water (below 1 °C) inhibits biological processes; and microorganism removal improves with lower filtration rates as well as with smaller sand size in the filter bed (Logsdon et al. 2002)."

As might be expected "control of turbidity improves after filter ripening (Logsdon 2002)."

02.06 Summary

For the site being analyzed in this thesis, a slow sand filter has been selected for design and implementation. The use of a slow sand filter will provide a treatment system which can be expected to provide effluent water quality acceptable for non-potable reuse, and a system which requires relatively little maintenance.

Based on the literature review, expected removals are high for sediment (70 – 90%) and BOD (70%), with effluent concentrations expected in the ranges of 5 – 20 mg/L and 2 – 30 mg/L respectively. Fecal coliform removals are expected to be moderate to high (65 -75%), with anticipated effluent concentrations being around 3900 Most Probable Number (MPN) per 100mL. Anticipated total metal removal is moderate – though a consistent effluent concentration of each metal is expected; averaging 10 μg/L for copper, 3 μg/L for lead, and 48 μg/L for zinc. Nutrient removal is anticipated to be moderate; total phosphorous about 30 - 75%, total nitrogen about 20 - 50% and total Kjeldahl nitrogen about 45 - 75%.

CHAPTER 3
SCOPE OF STUDY

03.01 Background

The Ada County Highway District (ACHD) municipality in Boise, Idaho maintains public roads and highways throughout the 1,060 square mile county. Each year, ACHD must dispose of thousands of cubic yards of water, sediment, leaves, and miscellaneous debris collected by its road sweepers and vacuum trucks.

When this project began in 2002 the county's three vacuum trucks, five vacuum street sweepers, and twelve mechanical sweepers were all discharging their collected waste at a small facility located at ACHD's maintenance yard in Garden City, Idaho.

The ACHD decanting system consisted of a large decant basin separated into two basins by a small concrete wall. The water entering the basin was directed from the larger basin through a 2-inch aggregate wall-filter into the smaller basin. From the smaller basin the water was discharged to the sanitary sewer system.

The approximately 2000 square foot system could not adequately address the needs of the ACHD for several reasons. First, the system was insufficiently sized for handling the large number of loads discharged in a given day. With this system as the only option for trucks to decant to, the debris within the basins regularly became problematic. The piling up of debris was excessive and difficult to manage because the basin was slow to drain and very slow to dry. Second, maintenance of the system was very difficult. The basins were hard to clean since they did not dry adequately making the debris soggy, heavy and very messy to deal with. Also, this soggy mess of debris had to be shoveled out by hand because trucks or front-end loaders could not get into the basins to clear them out. Third, all discharge of the processed water drained directly to the sanitary sewer creating a large load on the City of Boise's treatment plant and overall sanitary sewer collection system.

Figures 3.1 through 3.5 illustrate the ACHD decanting system at the Adams Street facility.

To address the problems associated with the Adams Street Facility, ACHD engaged the services of CH2M HILL to design a new system at their Cloverdale maintenance yard in Boise.

The focus of this thesis is to study the effectiveness of the sand filter and assess the overall quality of the water after treatment at the new Cloverdale facility.

03.02 Design Objectives

The design requirements for the new system had several objectives::

- adequate hydraulic capacity for anticipated loads.
- sufficient area for trucks to decant, wash, and fill.
- easy operations and maintenance.
- eliminate use of the sanitary sewer system; reuse process effluent.

The space available at the Cloverdale maintenance yard was approximately 175 feet by 150 feet or approximately 26,000 square feet. The total area is included in the facility design. The design includes a large concrete decant/sediment basin, a slow sand filter, a storage tank for the treated water and wash, decant, and fill areas for the trucks. The facility's final design layout is shown in Figure 3.6.

The treatment process is intended to work as follows: The decanted water will be routed through four sediment basin chambers. Settleable solids will settle out in the chambers and the water will then be pumped onto a slow sand filter for treatment. Water will pass through the sand filter; filtering out solids and pathogens; some removal of organics and nutrients may also be expected, if they are attached to solid particles, are precipitated into solid form, or are themselves solid particles. Finally, the water will be collected at the bottom of the sand filter and carried to a sump from where it will be pumped into a storage tank for re-use from the wash building.

If more water flows from the sand filter than the tank has capacity for, the water will be discharged to a storm drain system.

03.02.01 Design Objective 1, Hydraulic Capacity

The estimated required load that the new system would need to handle was determined based on the following information provided by ACHD:

- All three of the County's vacuum trucks, two of the five vacuum sweepers and none of the mechanical sweepers would be assigned to decant, wash, and fill at the Cloverdale site.
- The trucks operate between 250 and 300 days per year.
- Each truck is expected to decant at the site at least once per day and not more than five times per day.
- The volume of water and solids carried by a full vacuum truck is 3,000 gallons. The volume carried by a full vacuum sweeper is 1,500 gallons – though vacuum sweepers sometimes decant when only 2/3 full.

- Of the volume carried by the trucks to the decant facility the percent water in the vacuum truck loads varies from 50 to 95%. In the vacuum sweepers water content varies from 10 to 95%.

Combining these factors gives an average daily loading of approximately 11,500 gallons (ranging from 1,600 gal/day to 21,375 gal/day). The calculations are illustrated in Table 3.1.

Table 3.1 (Tim Mosko/CH2MHILL 2002)

Hydraulic Load Calculations

Type Vehicle			Truck Dumps @Cloverdale/ Day/Vehicle		Water + Solids Volume/Load		% Water/Load			Water Discharged at Cloverdale Site		
	Days/ Year Average	No. of Vehicles	Low (no.)	High (no.)	Low (gal)	High (gal)	Low (%)	High (%)	Daily Low (gal)	Daily High (gal)	Daily Average (gal)	Annual Total (gal)
Vacuum Trucks	275	3	1	5	3,000	3,000	50	95	1,500	14,250	7,875	2,165,625
Vacuum Sweepers	275	2	1	5	1,000	1,500	10	95	100	7,125	3,613	993,438
Total									1,600	21,375	11,488	3,159,063

Sizing of the sediment basin is discussed in detail in Section 2 of Chapter 4 of this thesis. The basin was sized maximizing the space available for construction and making sure to have adequate capacity based on the above figures. The resulting basin is sufficiently larger than required.

03.02.02 Design Objective 2, Sufficient Area

The space available at the Cloverdale maintenance yard is approximately 175 feet by 150 feet. This is more than twice the size of the available room at the Adams Street yard. In addition to adequate room for truck decanting, washing, and filling - sufficient room must be provided for stockpiling of debris for drying prior to final disposal. Using the same calculations shown in Table 3.1, Table 3.2 shows how the debris pile can be significant - averaging 7,225 gallons (965 cubic feet).

Table 3.2 (Tim Mosko/CH2MHILL 2002)

Solid Load Calculations												
Type Vehicle			Truck Dumps @Cloverdale/ Day/Vehicle		Water + Solids Volume/Load		% Solid/Load		Solids Disposed of at Cloverdale Site			
	Days/ Year Average	No. of Vehicles	Low (no.)	High (no.)	Low (gal)	High (gal)	Low (%)	High (%)	Daily Low (gal)	Daily High (gal)	Daily Average (gal)	Annual Total (gal)
Vacuum Trucks	275	3	1	5	3,000	3,000	5	50	150	7,500	3,825	1,051,875
Vacuum Sweepers	275	2	1	5	1,000	1,500	5	90	50	6,750	3,400	935,000
Total									200	14,250	7,225	1,986,875

The site was designed such that more than 15,000 SF of concrete area is provided for the truck operations of decanting, washing, and filling as well as space for sediment dumping and stockpiling. Given that the facility receives, on average, 7225 gallons per day, or 965 cubic feet per day, of wet debris, and assuming that the debris is piled approximately five feet high – roughly 200 SF of space will be utilized, on average, per day. Piles will dry within two or three days, after which they will be relocated and disposed of in the county landfill. 15,000 SF is more than enough room for operations at this facility.

03.02.01 Design Objective 3, Operations and Maintenance

The major frustration of operators with maintenance at the Adams Street system, and thus the primary O & M design objective for the Cloverdale facility, was cleaning out the basin. The Cloverdale site would need to provide access for cleaning equipment to easily enter and exit the treatment facilities.

The site was designed such that there is at least an 11-foot path/drive area between all of the system components. The basin walls and slopes were designed with ease of cleaning in mind. The basins are easily drained and a front-end loader may easily enter and exit the basins.

03.02.02 *Design Objective 4, Treat Process Effluent for Reuse*

In order to treat the water sufficiently for it to be suitable for use in operations it would have to meet certain standards. The treated water needs to comply with National Pollutant Discharge Elimination System (NPDES) requirements since excess water (quantity beyond that which could be stored for reuse) was to be discharged to surface waters rather than to the city sewer system. NPDES requirements for the ACHD at this time are simply to monitor pollutants during throughout the calendar year and to report on the findings in order to accurately characterize the quality and quantity of pollutants discharged. There are currently no numerical limits set on any particular pollutants.

Other criteria important to the facility include the following:

- System and equipment protection: The water should be sufficiently filtered so that it can be used in the sweeper and vacuum trucks without clogging the trucks' nozzles with fine particles.
- Health and safety: Since there are no specific water reuse regulations applicable to this site, guidelines for health and safety were established by ACHD and CH2M HILL using the NPDES permit and past experience as guidelines. Pollutants to watch for include coliforms, heavy metals, hydrogen sulfide, and volatile organic compounds, among others.
- Odor control: Odors from the large amounts of water decanted need to be kept under control. This is accomplished by keeping the water moving through the system and keeping it aerated through the sprinkling process of filter application.

In order to know if the water is being treated to suitable standards a sampling plan was developed. The sampling plan is discussed in detail in the following section on *Water Quality Monitoring*.

03.03 Water Quality Monitoring

As mentioned previously, the primary water quality concerns include worker health and safety, system protection and odor control. In addition, the sampling plan employed at this site was designed to evaluate the Best Management Practices utilized as part of ACHD's storm water management program as required by the NPDES permit, and to monitor for continued design performance of the system. After operations at the facility began in May 2003, sampling was performed monthly. Parameters monitored under this plan are discussed in the following sections.

03.03.01 General Monitoring

pH and *Water Temperature* were sampled and tested for to monitor the overall system performance. pH was monitored to verify that the water collected and processed was not unusually acidic or basic. Typical ranges for pH in stormwater are between 7.0 and 8.0. Water temperature can be considered a pollutant in streams and rivers; if temperatures get too high aquatic life may be threatened; also, if temperatures drop too low, the sand filter will become ineffective at removing microorganisms.

03.03.02 Solids Monitoring

Substances can exist in water in one of three classifications - suspended, colloidal, or dissolved.

Suspended solids can be removed from water by physical methods such as sedimentation and filtration.

Total Suspended Solids (TSS) was sampled and tested for to monitor system performance (efficiency of sediment basin and of sand filter), and to ensure system protection (avoid nozzle clogging in trucks).

Dissolved substances are homogeneously dispersed in the liquid and cannot be removed from the liquid without accomplishing a phase change through a process such as precipitation, adsorption, distillation, or extraction.

Total Dissolved Solids (TDS) was sampled and tested for to ensure system protection. The primary concern with dissolved solids is corrosion.

Conductivity is another general indicator of water quality. Conductivity is a measure of the water's ability to conduct electricity, and is determined by the amount of solids that are dissolved in the water. It can tell us how much solids are dissolved in the water, but not what kind of dissolved solids.

Conductivity was measured to ensure system protection. The primary concern with dissolved solids is corrosion.

Colloidal particles are in the size range between dissolved substances and suspended particles and are too small to be removed by sedimentation or normal filtration processes. Turbidity is used as a relative measure of these particles. Turbidity was not monitored in this particular case.

03.03.03 Phosphorous Monitoring

"Phosphorous serves as a vital nutrient for the growth of algae. If the phosphorous availability meets the growth demands of the algae, there is an excessive production of algae. When the algae die, they become an oxygen-demanding organic material as bacteria seek to degrade them. This oxygen demand frequently overtakes the dissolved oxygen supply of the water body and, as a consequence, causes fish to die (Davis et al. 1998)."

Soluble dissolved reactive phosphorous is the most available form of phosphorous to plants.

Total Phosphorous (P) and *Dissolved Reactive Phosphorous (DRP)* was sampled and tested for to monitor the performance of the facility and to ensure compliance with local and federal regulations for discharging to the storm drain system.

The United States Environmental Protection Agency (USEPA) established recommended total phosphorus limits for streams that enter lakes of 0.05 mg/L and for total phosphorus in flowing waters of 0.10 mg/L (WATERSHEDSS 2006).

03.03.04 Nitrogen Monitoring

Like phosphorous, nitrogen is a vital nutrient required for plant growth. There are several reasons why we must monitor the amount of nitrogen being released into a receiving body of water. The top three are presented here:

- In high concentrations, NH_3-N (Nitrogen-Ammonia) is toxic to fish.
- NH_3, in low concentrations, and NO_3^- serve as nutrients for excessive growth of algae.

- The conversation of NH_4^+ to NO_3^- consumes large quantities of dissolved oxygen.

Total Kjeldahl nitrogen is a measure of total organic and ammonia nitrogen in the water – gives a measure of the availability of nitrogen for building cells, as well as the potential nitrogenous oxygen demand that will have to be satisfied (Davis et al. 1998).

Total Kjeldahl Nitrogen (TKN) and *Nitrate + Nitrite (NO_3 + NO_2)* were sampled and tested for to monitor system performance and to ensure compliance with local and federal regulations for discharging to the storm drain system.

03.03.05 Total Organic Carbon (TOC)

The third primary nutrient required for plant growth is carbon. Carbon compounds serve as food source for microorganisms and can result in large blooms if not monitored and kept under control. Measuring for total organic carbon (TOC) provides a quick and convenient way of determining the degree of organic contamination within the water.

Total organic carbon (TOC) was sampled and tested for to monitor overall system performance.

03.03.06 Available Oxygen / Oxygen Demand

Adequate dissolved oxygen is necessary for good water quality. Oxygen is a necessary element to all forms of life. Natural stream purification processes require adequate oxygen levels in order to provide for aerobic life forms. As dissolved oxygen levels in water drop below 5.0 mg/L, aquatic life is put under stress. The lower the concentration, the greater the stress.

Dissolved Oxygen (DO) was sampled and tested for to monitor system performance, ensure system protection, and alleviate health and safety and odor control concerns.

In addition to the amount of DO in the water, the oxygen demand present in the water was also measured. This oxygen demand can be thought of as substances that fish and other natural aquatic life will have to compete with for the available oxygen present. There are two methods of testing for oxygen demand that are included in the sampling plan; Biochemical Oxygen Demand (BOD) and Chemical Oxygen Demand (COD).

BOD testing is an indirect measure of organic matter in the water. The test actually measures the change in DO caused by the microorganisms as they degrade the organic matter (Davis et al. 1998).

COD test is used to determine the oxygen equivalent of the organic matter that can be oxidized by a strong chemical oxidizing agent in an acid medium (Davis 1998). The five day *Chemical Oxygen Demand (COD_5)* and *Biochemical Oxygen Demand (BOD_5)* were sampled and tested for to monitor system performance and to ensure compliance with local and federal regulations for discharging to the storm drain system.

03.03.07 Oil and Grease

Oil and Grease was sampled and tested for in addition to noting any *Observable Floating Oil and Grease* to monitor system performance, ensure health and safety and odor control measures are being met, and to ensure compliance with local and federal regulations for discharging to the storm drain system.

03.03.08 Monitoring Total Metals

Heavy metals are those metals, when present in significant concentrations in water, which may pose detrimental health effects. Heavy metals include arsenic (As), barium (Ba), cadmium (Cd), chromium (Cr), copper (Cu), lead (Pb), mercury (Hg), nickel (Ni), selenium (Se), silver (Ag), tin (Sn), and zinc (Zn). The heavy metals have a wide range of effects. They may be acute poisons (As and Cr^{6+} for example), or they may produce chronic disease (Pb, Cd, and Hg for example) (Davis et al. 1998).

Total metals Copper, Lead, and Zinc (Cu, Pb, Zn) were sampled and tested for to monitor system performance and to ensure compliance with local and federal regulations for discharging to the storm drain system.

These three particular metals were selected by ACHD based upon their experience in the county, as these have shown up previously in areas served by the sweeper and vacuum trucks that use the new Cloverdale Facility. These metals were monitored to ensure to accurate characterization of the quality and quantity of pollutants discharged from the facility.

03.03.09 Monitoring Pathogens with Coliforms

To test for the presence of pathogenic (disease causing) organisms, indicator organisms are used. Pathogenic organisms themselves are few and difficult to isolate and identify. Therefore, the coliform organism is used as an indicator of pathogenic organisms. If coliforms are present it is an indicator that specific disease producing organisms may be present. Some of these originate with the fecal discharges of infected individuals; others are from the fecal discharge of animals.

The coliform group includes two genera: *Escherichia coli (E-coli)* and *Aerobacter aerogenes*. The reasoning behind the origination of the Total Coliform test is described by Davis and Cornwell as follows:

- The coliform group of organisms normally inhabits the intestinal tracts of humans and other mammals. Thus, the presence of coliforms is an indication of fecal contamination of the water.
- Even in acutely ill individuals, the number of coliform organisms excreted in the feces outnumber the disease-producing organisms by several orders of magnitude. The large numbers of coliforms make them easier to culture than disease-producing organisms.
- The coliform group of organisms survives in natural waters for relatively long periods of time, but does not reproduce effectively in this environment. Thus, the presence of coliforms in water implies fecal contaminations rather than growth of the organism because of favorable environmental conditions. These organisms also survive better in water than most of the bacterial pathogens. This means that the absence of coliforms is a reasonably safe indicator that the pathogens are not present.
- The coliform group of organisms is relatively easy to culture. Thus, laboratory technicians can perform the test without expensive equipment (1998).

Idaho Administrative Code Section 58.01.02 Water Quality Standards provides standards for acceptable e coli levels in waters with recreational use designations. Section 251.01 states the following:

> Waters designated for primary or secondary contact recreation are to contain E coli bacteria in concentrations exceeding a geometric mean of 126 e coli organisms per 100 mL based on a minimum of 5 samples taken every three to seven days...
>
> For waters designated as secondary contact recreation, a single sample maximum of 576 e coli organisms per 100 mL; or
>
> For water designated as primary contact recreation, a single sample maximum for 406 e coli organisms per 100 mL; or
>
> For areas within waters designated as primary contact recreation that are additionally specified as public swimming beaches, a single sample maximum of 235 e coli organisms per 100 mL. Single sample counts above this value should be used in considering beach closures.

Based on this information, ANY e coli present in water can present some safety concerns. The water used at the Cloverdale facility does not, however, fall under the category of waters for recreational use. The water is not likely to be ingested, and all taps into the treated water supply are marked "Non- Potable. Do Not Drink!"

Total coliforms, Fecal coliforms, and E-coli were sampled and tested for to monitor system performance, and to ensure health and safety precautions are being met as well as to ensure compliance with local and federal regulations for discharging to the storm drain system.

03.03.10 Monitoring Hydrogen Sulfide (H_2S)

Sulfide is found throughout the environment as a result of both natural and industrial processes. Most sulfide found in nature was produced biologically (under anaerobic conditions) and occurs as free hydrogen sulfide (H_2S) - characterized by its rotten egg odor. We are most likely to encounter biogenic H_2S in sour groundwaters, swamps and marshes, natural gas deposits, and sewage collection/treatment systems. Manmade sources of H_2S typically occur as a result of natural sulfur containing materials (e.g., coal, gas and oil) being refined into industrial products.

For a variety of reasons - aesthetics (odor control), health (toxicity), ecological (oxygen depletion in receiving waters), and economic (corrosion of equipment and infrastructure) - sulfide laden wastewaters must be handled carefully and remediated before they can be released to the environment. Typical discharge limits for sulfide are < 1 mg/L (Reference Library 2006).

Hydrogen Sulfide (H_2S) Headspace was tested for to monitor system performance, ensure health and safety precautions are taken as necessary, and to address any odor and corrosion concerns.

03.03.11 Monitoring Volatile Organic Compounds (VOCs)

According to the USEPA volatile organic compounds (VOCs) are organic chemical compounds that have high enough vapor pressures under normal conditions to vaporize and enter the atmosphere.

VOCs are sometimes accidentally released into the environment, where they can become soil and groundwater contaminants. VOCs include a variety of chemicals, some of which may have short- and long-term adverse health effects.

VOC Headspace was sampled and tested for to monitor overall system performance and to determine if any health and safety concerns due to VOC headspace is or is not present.

03.04 Chapter 3 Figures

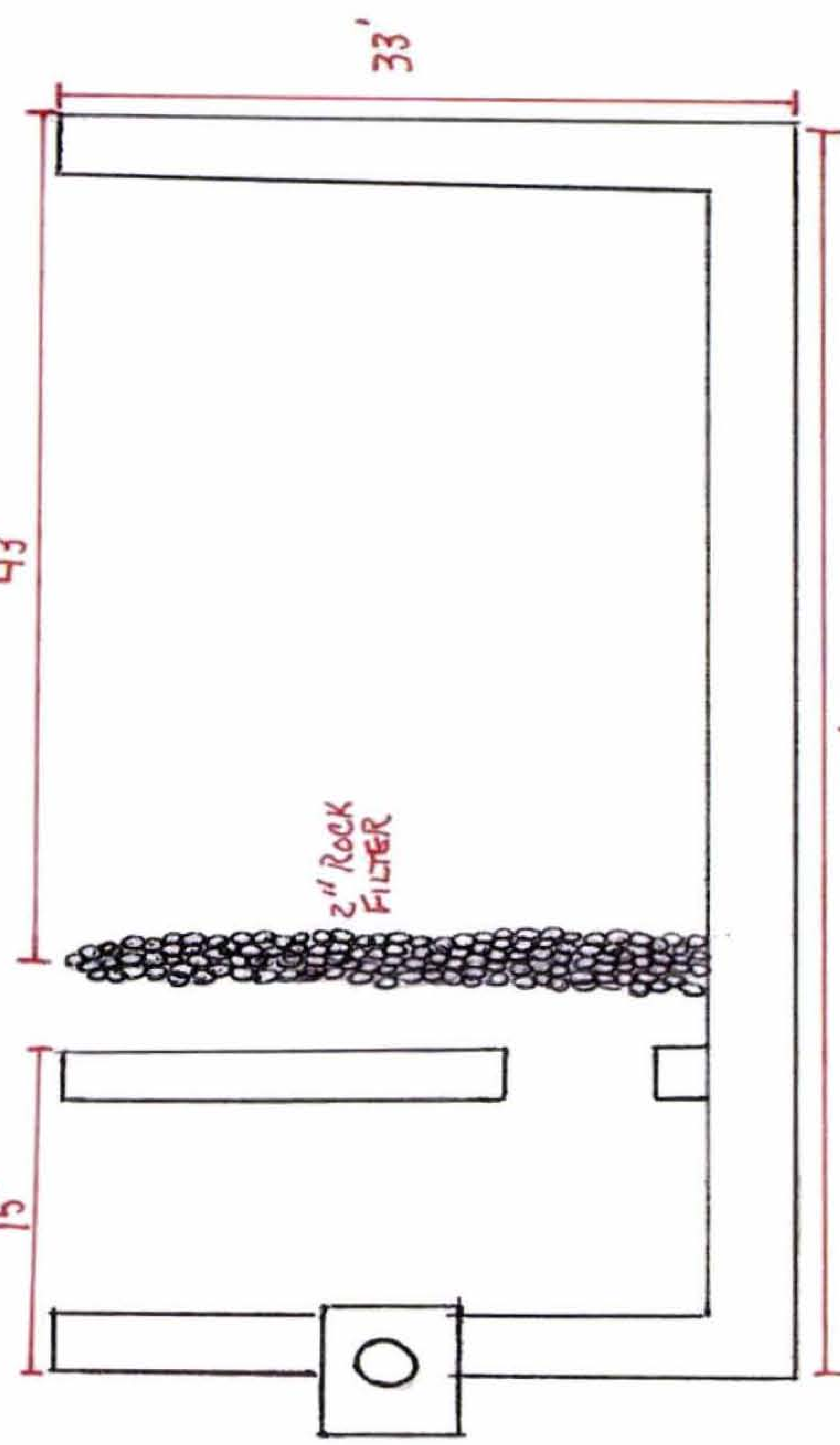


Figure 3.1: Adams Street Decant Basin Plan View (sketch by Jeff Brockett/ACHD 2006)

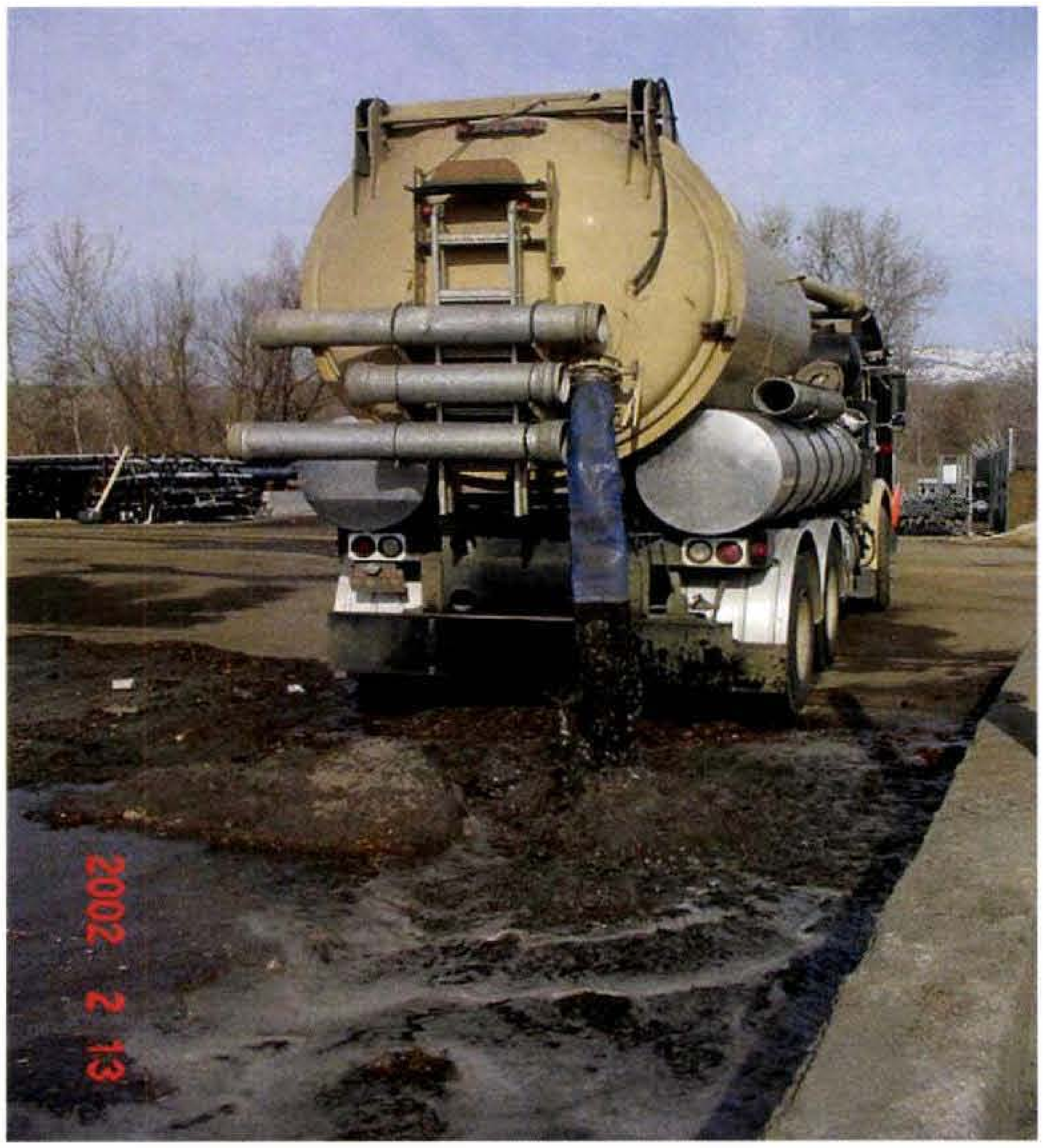


Figure 3.2 - Vactor Truck dumping at ACHD's Adams Street Facility

Figure 3.3 - Decant Basin at ACHD's Adams Street Facility (photo taken facing West)

Figure 3.4 – Decant Basin at ACHD's Adams Street Facility (photo taken facing South)

Figure 3.5 – Decant Basin at ACHD's Adams Street Facility (photo taken facing East)

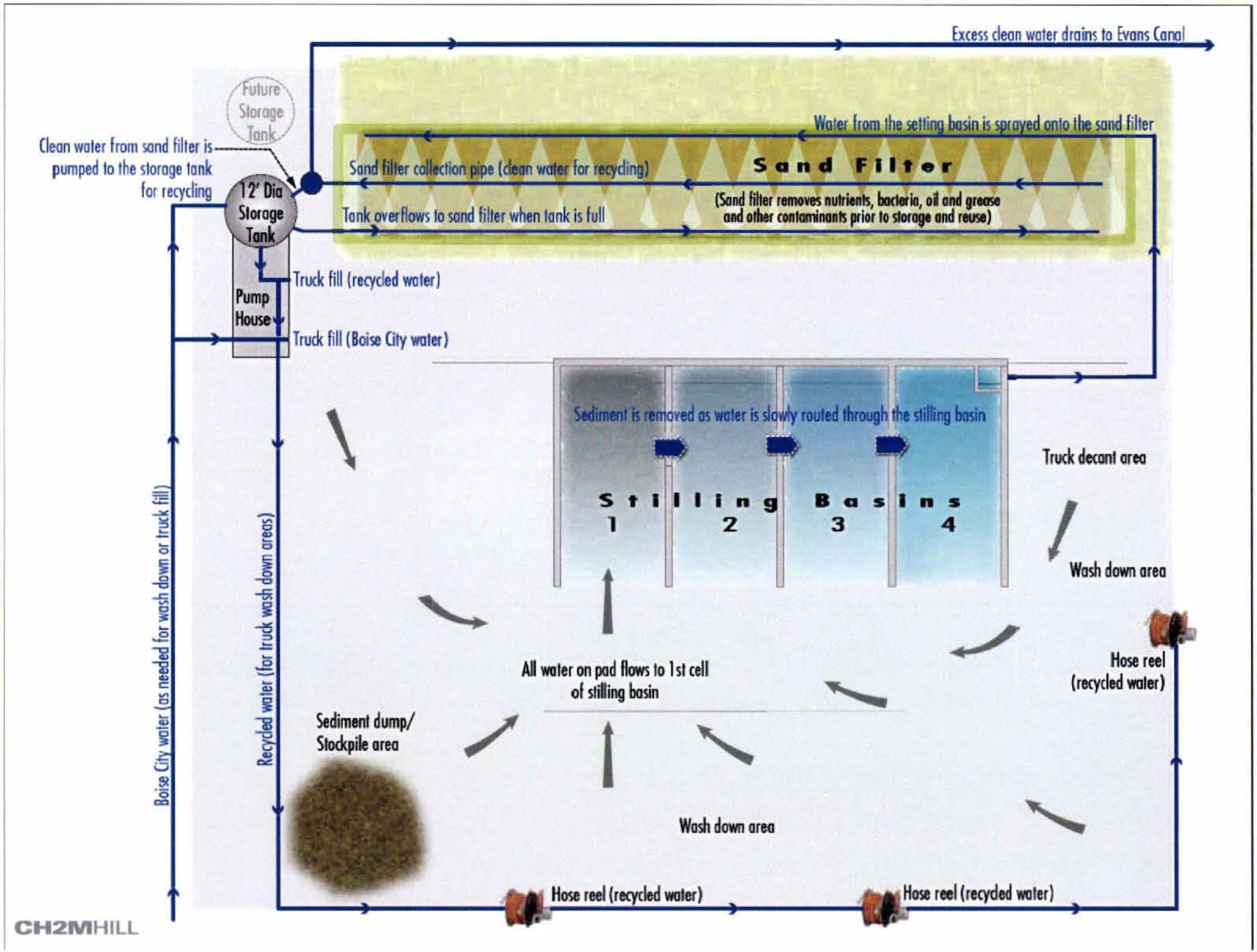


Figure 3.6: Cloverdale facility site plan. (CH2M HILL 2002)

CHAPTER 4
DESIGN AND CONSTRUCTION

The site design utilizes the entire 150 foot by 175 foot area provided by ACHD for the facility. The site plan is shown in both Sheet 1 of the design drawings (Appendix A), and in Figure 3.6 of the previous section.

The facility consists of a concrete pad for trucks to decant, wash, fill and dump, and stockpile solid debris; a sediment basin to route the decanted storm water through; a slow sand filter for treatment of the storm water after it has moved through the sediment basin; a storage tank for the treated water; and a storm drain system to discharge excess water to. The facility was designed without the option of utilizing the City sewer system; the excess water must be discharged via a storm drain system.

The flow schematic is illustrated on Sheet 6 of the design drawings (Appendix A).

The features are discussed individually in the following sections. Photographs of the site during and after construction are provided in Appendix B.

04.01 Truck Decant, Wash, and Fill / Debris Dumping and Stockpiling

The concrete pad covers nearly the entire footprint of the facility, with the exception of the gravel access road between the sand filter and the sediment basin.

The concrete pad provides ample space for all truck operations, including decant, wash and fill areas as well as areas for debris and sediment dumping and stockpiling. Hose reels are included in the design as shown in Figure 3.6 so that washing and filling can be performed at multiple locations. Suggested locations for decanting and debris stockpiling are shown in Figure 3.6 as well. The recommended layout as shown in Figure 3.6 provides approximately 2000 SF for stockpiling wet debris in the southwest corner of the facility, an additional 2000 SF for filling operations near the storage tank, and more than 5000 SF for wash down areas near the hose reels. However, the pad can be used in whatever manner operators feel is best – all water on the concrete pad will drain to the beginning of treatment operations regardless of where it is placed.

The grading of the concrete pad was designed to drain completely to the first basin. All wash water will drain to the first basin; all of the water drained from the stockpiled debris, all storm water from storm events, all water on the concrete pad will drain to the first sediment basin and enter the treatment process. The grading plan is shown on Sheet 4 of the design drawings (Appendix A).

04.02 Sediment Basin

The sediment basin was designed as a series of four basins to maximize available retention time and remove as much of the larger solid particles as possible before sand filter application. The design of the sediment basins is basic and straightforward. The basins were designed using the space available, while also making sure that the anticipated hydraulic loading would be accommodated, and that the basins could be easily cleaned and maintained. The basins ultimately were designed to fit the site, and are larger than required by any hydraulic calculations performed during the design.

The area chosen for the sedimentation basin is located roughly in the center of the facility and covers a rectangular area 80 feet in length and 40 feet in width. The basin is designed to operate in a series of four basins, each having the same overall square footage but varying depth. The floor of the basins slopes down at a 2% grade from the first to the fourth, such that the hydraulic capacity of the last basin is significantly greater than that of the first. The floor of the fourth basin is approximately 14 inches lower than that of the first basin.

The basins are constructed of Portland cement concrete. The design includes separating the basins by 18-inch concrete walls with slide gates staggered across so that the flow of water between basins can be controlled. These walls also provide walkways for personnel to access the basins if necessary.

To accommodate trucks and equipment entering and exiting the basins the design includes ramps into each basin. The ramps are sloped at an approximately 20% incline into each basin before leveling off. The ramps extend an average of 20 feet into each basin followed by a 24 foot length of level basin floor.

Speed bumps are located at the entrance to basins 2, 3, and 4. The speed bumps allow rubber tired equipment to enter and exit the basins while simultaneously serving as drainage guides as well as barriers for the water on the upper end of the basins.

The estimated required hydraulic load was determined based on the following information provided by ACHD:

- All three of the County's vacuum trucks, two of the five vacuum sweepers and none of the mechanical sweepers would be assigned to decant, wash, and fill at the Cloverdale site.
- The trucks operate between 250 and 300 days per year.

- Each truck is expected to decant at the site at least once per day and not more than five times per day.
- The volume of water and solids carried by a full vacuum truck is 3,000 gallons. The volume carried by a full vacuum sweeper is 1,500 gallons – though vacuum sweepers sometimes decant when only 2/3 full.
- Of the volume carried by the trucks to the decant facility the percent water in the vacuum truck loads varies from 50 to 95%. In the vacuum sweepers water content varies from 10 to 95%.

Combining these factors gives an average daily loading of approximately 11,500 gallons, as shown in Table 3.1.

Table 4.1 provides end-area calculations for each of the four basins, giving a total basin volume of more than 70,000 gallons. This volume will allow for an average detention period of approximately 6 days, which is very good. Literature suggests a detention period of anywhere from 12 to 48 hours.

Table 4.1 (Tim Mosko/CH2MHILL 2002)

			Basin Volume Calculations	
Basin 1: End-Area Volume Calculations				
Storage Section			Area 1 = (1/2(a+b)h) in ft^2	Area 2 = Area 1in ft^2
a	b	h	62.9	62.9
3.2	3.6	18.6	Volume (1/2 (A_1+A_2) 24') in ft^3	
			1509.4	
Ramp Section				
Area 1	Area 2	Volume (1/2 (A_1+A_2)16')		
0	62.9	503.1		
Basin 2: End-Area Volume Calculations				
Storage Section			Area 1 = (1/2(a+b)h) in ft^2	Area 2 = Area 1in ft^2
a	b	h	70.3	70.3
3.6	4.0	18.6	Volume (1/2 (A_1+A_2) 24') in ft^3	
			1687.8	
Ramp Section				
Area 1	Area 2	Volume (1/2 (A_1+A_2)16')		
0	70.3	562.6		
Basin 3: End-Area Volume Calculations				
Storage Section			Area 1 = (1/2(a+b)h) in ft^2	Area 2 = Area 1in ft^2
a	b	h	77.8	77.8
4	4.4	18.6	Volume (1/2 (A_1+A_2) 24') in ft^3	
			1866.2	
Ramp Section				
Area 1	Area 2	Volume (1/2 (A_1+A_2)16')		
0	77.8	622.1		
Basin 4: End-Area Volume Calculations				
Storage Section			Area 1 = (1/2(a+b)h) in ft^2	Area 2 = Area 1in ft^2
a	b	h	85.2	85.2
4.4	4.8	18.6	Volume (1/2 (A_1+A_2) 24') in ft^3	
			2044.5	
Ramp Section				
Area 1	Area 2	Volume (1/2 (A_1+A_2)16')		
0	85.2	681.5		
Total Basin Volume:		9477	ft^3	
		70895	gal.	

The basin details are illustrated in Figures 4.2 through 4.5, taken from the design drawings. The full set of design drawings is provided as Appendix A.

Figure 4.2 shows the plan view of the basins; all dimensions given are in feet. Figures 4.3, 4.4, and 4.5 show section views of the basins which illustrate the ramp design as well as the containment and flow control features.

Water is pumped from the fourth basin via a 60 gallon per minute (gpm) float actuated sump pump to the distribution line on the sand filter. The piping schematic is illustrated on Sheet 6 of the design drawings (Appendix A).

04.03 Slow Sand Filter

The sand filter constructed at the Cloverdale site was designed using general slow sand filter design guidelines while maximizing the space available. Additionally, a design was developed using design guidelines presented in Urbonas's article, *Design of a Sand Filter for Stormwater Quality Enhancement*. Urbonas's article is included as Appendix C. The design process and results are presented here.

The design process has four steps:

1. Estimate the stormwater run-off volume and suspended solid load for the area.
2. Calculate the rate of accumulation of solids on the filter's surface.
3. Relate the accumulation of solids to the available hydraulic flow through rate.
4. Use the available hydraulic flow through rate to determine final sizing for the sand filter and capture volume.

Step 1: Stormwater run-off volume and suspended solid load

Using the equation recommended in the article, an estimate for storm-water run-off from the facility site itself can be obtained.

I_a - Imperviousness of catchment area = 70.5% (0.43 acres of 0.61 acre facility)

i_a - Fraction of catchment's total area covered by impervious surfaces, $I_a/100$

C - Catchment's run-off coefficient

$C = 0.858\ i_a^3 - 0.78\ i_a^2 + 0.774\ i_a + 0.04$ (Urbonas Equation 2)

$C = 0.858\ (0.705)^3 - 0.78\ (0.705)^2 + 0.774\ (0.705) + 0.04$

$C = 0.50$

P_6 - Average storm depth = 0.30 inches (from Urbonas Figure 1)

n - Average number of storms per year ≥ 0.1 inch in depth = 33 (from Urbonas Figure 3)

P_A - Average annual total stormwater run-off from the catchment in inches

$P_A = n * P_6 * C$ (Urbonas Equation 3)

$P_A = 33 * 0.30$ inches $* 0.50$

$P_A = 4.94$ inches

A_c - Tributary area = 0.61 acres (Facility area)

E_s - Expected estimated maximum concentration (EMC) of TSS = 908 mg/L (from *ACHD 2001 Sediment/Decant Water Sampling Report*)

L_a - Average annual TSS load in stormwater, in pounds (lb)

$L_a = 0.2265 * A_c * P_A * E_s$ (Urbonas Equation 4)

$L_a = 0.2265 * 0.61$ acres $* 4.94$ inches $* 908$ mg/L

$L_a = 619$ lb

<u>Step 2</u>: *Solid accumulation*

R_T - Total system's average percent removal rate of TSS = 95% (assumed removal rate for detention basin / filter combination system)

R_D - Assumed percent removal rate for upstream detention basin = 60% [from Urbonas Table 1, based on 48 hour detention time (T_d)]

E_{sfr} - The reduction in the EMC of TSS by the filter, mg/L

$E_{sfr} = E_s * [(R_T - R_D) / 100]$ (Urbonas Equation 7)

$E_{sfr} = 908$ mg/L $*[(95 - 60) / 100]$

$E_{sfr} = 317.8$ mg/L

b - The fraction of all average annual run-off volumes that is treated by the facility = 90%

L_{afr} - Average annual TSS load removed by the filter, lb

$L_{afr} = b * (E_{sfr} / E_s) * L_a$ (Urbonas Equation 12)

$L_{afr} = 0.90 * (317.8 / 908) * 621$ lb

$L_{afr} = 195.0$ lb

Step 3: Establish solid loading / hydraulic loading relationships

m – Annual maintenance schedule = 0.5 (assume once every other year)

L_m – Average TSS load removed by each square foot of the filter during each maintenance cycle, lb/sq ft

A_{fm} – Surface area of the filter sized on the basis of TSS for load removed, in square feet

$A_{fm} = L_{afr} / (L_m * m)$ (Urbonas Equation 14) and

$L_m = L_{afr} / (A_{fm} * m)$ (Urbonas Equation 13)

Two equations, two unknowns. In order to relate the flow through rate, Urbonas introduces the following:

q – Design flow through rate through sand filter's surface, in inches per hour (in/hr)

$q = 0.75\, L_m^{-1.165}$ (Urbonas Equation 11 from Figure 5)

Three equations, three unknowns.

Next, a separate requirement is added in.

Two equations for the filter's surface area must be satisfied; one based on TSS load removal (A_{fm}) and the other based on hydraulic sizing (A_{fh}).

P_o – Maximized water quality capture volume = 0.23 inches (from Urbonas Figure 2 and Equation 1)

T_d – Time for volume P_o to totally drain out at the design flow through rate q = 48 hours

A_{fh} – Surface area of the filter based on hydraulic sizing, in square feet

$A_{fh} = (P_o * A_c * 43560) / (q * T_d)$

Step 4: Final sand filter sizing and capture volume

Finally, tying all of the equations established in step 3 together, a spreadsheet solution is obtained. The goal is to find,

$A_{fm} \approx A_{fh}$

By changing only the value of L_m, a solution of 270 square feet is obtained for the filter surface area.

Note: The design at this point only takes into account the run-off produced at the facility itself by storm events. Stormwater volumes must be adjusted to reflect actual applied hydraulic loading.

The volume obtained in the previous section for sizing the sediment basin is 11,500 gallons per day, which converts to 0.42 acre-inches.

Repeating the steps in this section using a drainage area of 0.42 acres, 100% impervious, a storm depth of 1 inch, and 275 events per year (average number of days of truck operation); keep the maintenance requirements at once every other year, an additional 1800 square feet surface area is required.

This brings the total square footage to approximately 2100 square feet. The actual square footage of the sand filter is 2800 square feet.

Final Design

The design features are fully illustrated in the design drawings, Appendix A.

- The square footage of the sand filter was amply sized at 2800 square feet; a 20-foot by 140-foot rectangle. The loading rate is therefore determined based on this square footage and the 60 gpm float actuated sump pump which transfers the water from the basin to the sand filter. The design loading rate is 60 gpm per 2800 square feet or 0.021 gpm/ft^2. This is at the low end of typical values for slow sand filter loading rates which rates range from 0.015 gpm/ft^2 to 0.16 gpm/ft^2.

- The media chosen was ASTM C-33 mix concrete sand which, as noted by Urbonas, has proven to provide "a good balance between hydraulic flow through rates and filtering efficiencies (1999)." The specifications and gradation curve are provided as Appendixes B1 and B2.
- The media depth was designated at 3 feet. This was chosen somewhat arbitrarily; typically media depth for slow sand filters is between 3 and 4 feet.
- The sand filter was designed with a 6-inch layer of topsoil covering the sand layer. The topsoil design was intended to improve microorganism removal rates, similar to using a peat-sand mixture media.
- The distribution and collection piping is shown in the cross section, Figure 4.6 and on Sheet 3 of the design drawings.
- A plastic liner was placed at the bottom of the filter to prevent infiltration of the treated water into the soil.
- The sand layer was isolated from surrounding layers (topsoil and drainrock) by a geotextile filter fabric. The geotextile (Permeatex 4045 non-woven) specifications are included as Appendix B3.

04.04 Storage Tank and Piping

A 17,000 gallon capacity tank was selected for holding the treated water. The piping configuration is shown in Appendix A, Sheet 5.

Treated water is pumped from the sump at the end of the sand filter into the tank. From the tank, the water is available for use at the facility for washing or filling the trucks. For the situation where not enough water is available in the tank for operational needs, City water supply line is available to the tank to supplement the water supply.

In addition to the fill lines to the tank, there is a drain line from the tank that discharges to a sand filter distribution pipe. As necessary, the water within the tank can be retreated to further remove pollutant loads by recirculation through the sand filter.

The tank and piping design is best shown by Appendix A, Sheets 5 and 6.

04.05 Storm Drain System

In the case that the storage tank is full and the system is still sending treated water through the sand filter, overflow water needs a place to go. Since the facility was designed without the option of utilizing the City sewer system, the excess water must be discharged via a storm drain system.

There is a drainage canal just beyond the east end of the Cloverdale property. This drainage canal is called Evans Drain and is a tributary of the Boise River. Evans Drain is owned and operated by the Nampa and Meridian Irrigation District (NMID). Evans Drain was selected as the best option for the storm drain discharge point.

Additionally, there is a set of Union Pacific Railroad tracks between the Cloverdale property and Evans Drain.

The 6-inch storm drain runs from the sump at the end of the sand filter north to back of the sand filter and heads east along the north property boundary, past the east property boundary, under a set of Union Pacific Railroad tracks and discharges into Evans Drain.

A license agreement between ACHD and NMID was established permitting the discharge to Evans Drain, and ACHD purchased an easement from UPRR in order to encroach upon their right of way.

The NMID license agreement requires that ACHD not discharge more that 0.16 cfs along with monitoring requirements proposed by ACHD. These monitoring requirements were already required by ACHD as part of their National Pollutant Discharge Elimination System (NPDES) permit. The monitoring requirements and practices were described in Chapter 3, *Scope of Study*. NPDES requirements for the ACHD at this time are to monitor pollutants during throughout the calendar year and to report on the findings in order to accurately characterize the quality and quantity of pollutants discharged. There are currently no numerical limits set on any particular pollutants.

Union Pacific Railroad granted an easement, but would not allow trenching across the right-of-way. The piping was installed by "pushing" it underneath the tracks. Photos are included in Appendix B.

The storm drain profile is shown in Appendix A, Sheet 7.

04.06 Chapter 4 Figures
(the following figures taken from CH2MHILL project 148140.Q1.01 design drawings)

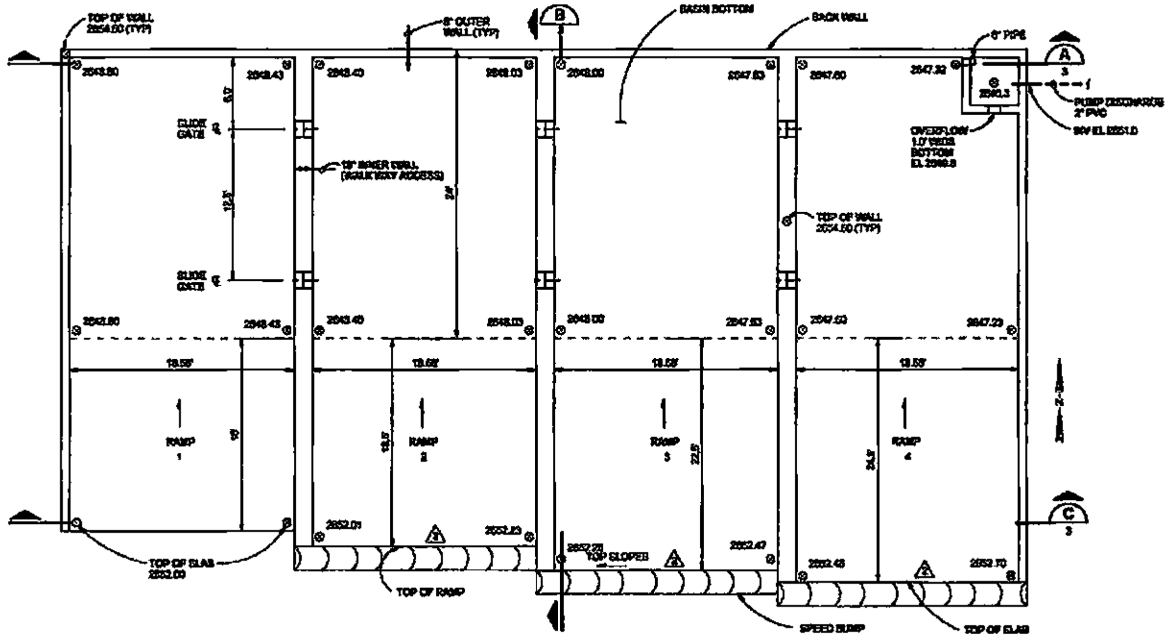


Figure 4.1: Sediment basin, plan view

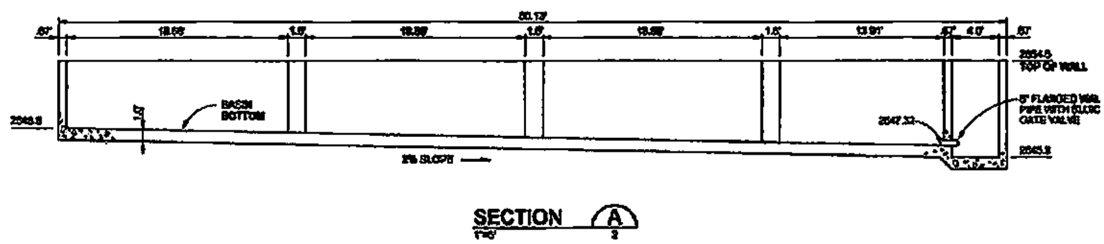


Figure 4.2: Sediment basin, section A

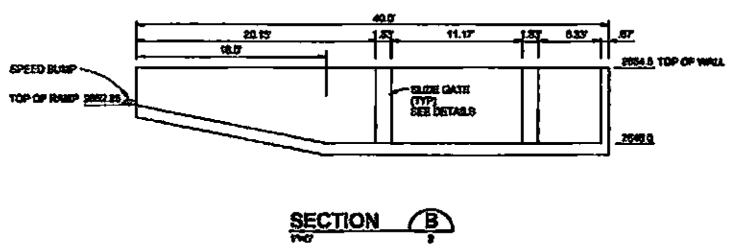


Figure 4.3: Sediment basin, section B

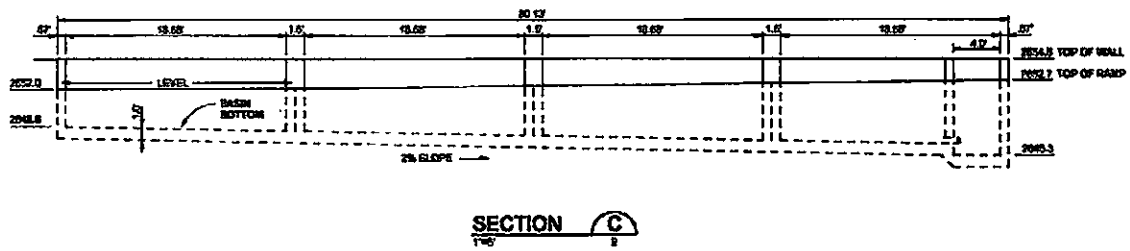


Figure 4.4: Sediment basin, section C

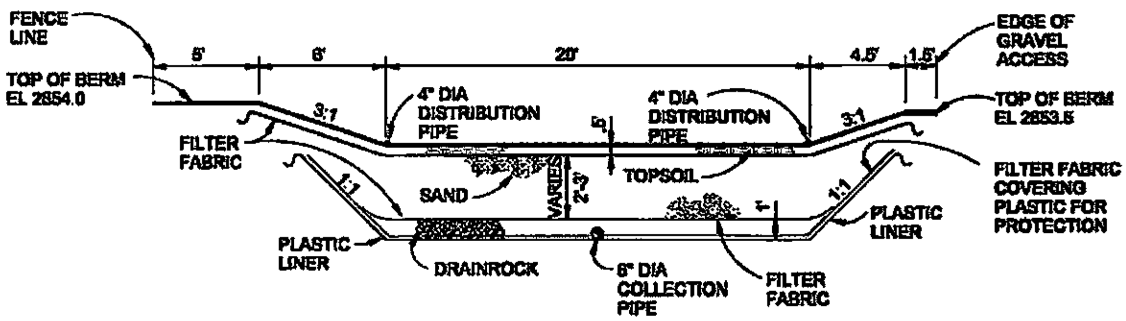


Figure 4.5: Sand filter section

CHAPTER 5
RESULTS AND DISCUSSION

Facility construction was completed in early 2003 and operations began in May 2003.

The sand filter had clogging problems almost immediately after the facility began operations. To remedy this, the top layer (6-inches of topsoil) was removed and replaced with sand. This was performed in July 2003.

Other post-construction modifications include only minor structural changes to the basin speed bumps and relocation of hose reels.

Monthly water quality monitoring was performed as described in Chapter 3 of this paper, beginning May 2003 and concluding in November 2004. Samples were taken at the basin outlet (Figure 5.1), the filter outlet (Figures 5.2 and 5.3), the tank spigot (Figure 5.4), and at the outfall to Evans Drain (Figure 5.5). Water quality monitoring beyond November 2004 is not included in this study.

The primary focus of this paper is to evaluate the effectiveness of the sand filter, which is achieved by comparing water quality data from the basin outlet (basin effluent is equal to sand filter influent) to that from the sand filter outlet. In addition, some information is presented which provides only data on water quality at the tank where it is stored for reuse.

05.01 Water Quality Results

The results are presented below by category as discussed in Chapter 3.

05.01.01 *General Monitoring*

pH and Water Temperature were measured at the basin outlet and sand filter outlet.

Figures 5.6 and 5.7 show pH and temperature values before and after sand filter application of the storm water. No significant change is shown to occur in either during this process.

The data shows an average pH around 7.7 at the basin effluent/sand filter influent, and 7.6 at the sand filter effluent. These values are as expected and indicate that neither extreme acidic or basic conditions are present in the system. Likewise, temperatures are moderate and do not present any cause for alarm.

05.01.02 *Solids Monitoring*

Total Suspended Solids (TSS) removals, as shown in Table 5.1, are very good at the facility. Roughly 95% of the TSS concentration is removed by the sand filter.

Table 5.1

Percent Removals of TSS in Sand Filter

Parameter	Sand Filter Influent (Mean)	Sand Filter Effluent (Mean)	Percent Removal
TSS mg/L	75.5	4.9	93%
	Sand Filter Influent (Median)	**Sand Filter Effluent (Median)**	**Percent Removal**
	61.45	2.45	96%
	Sand Filter Influent (GeoMean)	**Sand Filter Effluent (GeoMean)**	**Percent Removal**
	60.2	2.7	95%

Figure 5.8 shows sand filter influent and effluent TSS concentrations over time. The figure highlights an important feature of the sand filter – not only can we expect high removal rates (on average 95%) but we can expect a relatively constant effluent TSS

concentration regardless of influent concentration. This phenomenon is discussed in greater detail in Chapter 2 of this paper and explained fully by Michael Barrett in his article *Performance, Cost, and Maintenance Requirements of Austin Sand Filters* (Barrett 2003).

A quick review of Chapter 2 (*Literature Review*) tables shows that the sand filter removal efficiencies found at the Cloverdale yard are in line with what is expected given the size of the filter and relatively low application rate of storm water. Recall from Chapter 4 (*Design and Construction*) that the sand filter's loading rate is 0.021 gpm/ft^2. This is at the low end of typical values for slow sand filter loading rates which range from 0.015 gpm/ft^2 to 0.16 gpm/ft^2. The literature review (see Chapter 2 Tables 2.1, 2.2, 2.3, 2.4, and 2.5) revealed that sand filters typically can be expected to remove between 60 and 85 percent of TSS concentrations. The sand filter at Cloverdale is more efficient due to the large surface area and low application rate. Effluent values between 2 and 40 mg/L are expected per the literature review, actual values averaging 4.9 mg/L compares very well.

Total Dissolved Solids (TDS), and Conductivity concentrations tend to show an increase as the water is processed through the facility.

Sand filter application of the basin effluent is somewhat insignificant for TDS concentrations and conductivity measurements as can be seen in both Table 5.2 and Figures 5.9 and 5.10. As expected, the sand filter does not affect TDS or Conductivity.

Table 5.2

Percent Removal of TDS and Conductivity Measurements in Sand Filter			
Parameter	Sand Filter Influent (Mean)	Sand Filter Effluent (Mean)	Percent Removal
TDS mg/L	678.8	729.1	-7%
Conductivity µs/cm	899.8	680.8	24%
	Sand Filter Influent (Median)	Sand Filter Effluent (Median)	Percent Removal
TDS mg/L	496	453	9%
Conductivity µs/cm	620	575	7%
	Sand Filter Influent (GeoMean)	Sand Filter Effluent (GeoMean)	Percent Removal
TDS mg/L	591.4	555.8	6%
Conductivity µs/cm	547.9	621.7	-13%

05.01.03 Phosphorous Monitoring

Total Phosphorous and Dissolved Reactive Phosphorous (DRP) concentrations are shown in Figures 5.11 through 5.14 and average removal percentages are shown in Table 5.3.

Table 5.3

Percent Removal of Phosphorous in Sand Filter			
Parameter	Sand Filter Influent (Mean)	Sand Filter Effluent (Mean)	Percent Removal
DRP mg/L	0.082	0.160	-95%
Total P mg/L	0.605	0.268	56%
	Sand Filter Influent (Median)	Sand Filter Effluent (Median)	Percent Removal
DRP mg/L	0.014	0.152	-957%
Total P mg/L	0.558	0.264	53%
	Sand Filter Influent (GeoMean)	Sand Filter Effluent (GeoMean)	Percent Removal
DRP mg/L	0.022	0.151	-580%
Total P mg/L	0.531	0.233	56%

For total phosphorous, the EPA recommends a concentration of 0.1 mg/L for flowing waters. Sand filter effluent concentrations are consistently higher than 0.1

mg/L (mean value 0.3 mg/L). However, this is cannot be deemed a problem without knowledge of the receiving water's total phosphorous concentration, and that data is not currently available.

As expected per the literature review – removals of this nutrient are moderate. Literature review suggested removals between 30 and 75%. With actual TP removals averaging 56%, the sand filter is performing well, and as expected.

05.01.04 Nitrogen Monitoring

Total Kjeldahl Nitrogen (TKN) and Nitrate + Nitrite (NO_3 + NO_2) concentrations are shown in Figures 5.15 and 5.16, and average removal percentages are shown in Table 5.4.

Table 5.4

Percent Removal of Nitrogen in Sand Filter

Parameter	Sand Filter Influent (Mean)	Sand Filter Effluent (Mean)	Percent Removal
Nitrate + Nitrite mg/L	0.41	2.34	-470%
TKN mg/L	6.08	2.23	63%
	Sand Filter Influent (Median)	**Sand Filter Effluent (Median)**	**Percent Removal**
Nitrate + Nitrite mg/L	0.125	1.810	-1348%
TKN mg/L	5.65	1.60	72%
	Sand Filter Influent (GeoMean)	**Sand Filter Effluent (GeoMean)**	**Percent Removal**
Nitrate + Nitrite mg/L	0.215	1.393	-547%
TKN mg/L	5.444	1.717	68%

To measure total nitrogen, we measure TKN (organic nitrogen plus ammonia nitrogen) and nitrate plus nitrite. Aerobic bacteria within the filter will convert ammonia into nitrate and nitrite through nitrification. This explains the decrease in TKN through the filter and the associated increase in $NO_3 + NO_2$.

Comparing these efficiencies to those found through the literature review, it is apparent that this phenomenon was to be expected. Table 2.5 of the *Literature Review* chapter of this paper clearly shows a decrease in ammonia nitrogen (NH_4-N) accompanied by an increase in Nitrate nitrogen (NO_3-N).

Examination of Tables 2.2, 2.3, and 2.4 of the *Literature Review* chapter lead to the conclusion that a removal of total nitrogen can be expected. Examination of Table 5.4 shows an average increase in mg/L of $NO_3 + NO_2$ (due to nitrification) is between 1 and 2 mg/L and the decrease in TKN mg/L of is about 4 mg/L – resulting in a net decrease of nitrogen of about 2 or 3 mg/L. TKN removal is as expected – literature suggests 60 to 75% and Table 5.4 shows averages of 63 to 72%.

05.01.05 Total Organic Carbon (TOC)

Total organic carbon (TOC) concentrations, and sand filter removal rates are shown in Figures 5.17 through 5.19 and Table 5.5.

Table 5.5

Percent Removal of Total Organic Carbon in Sand Filter

Parameter	Sand Filter Influent (Mean)	Sand Filter Effluent (Mean)	Percent Removal
TOC mg/L	45.4	26.0	43%
	Sand Filter Influent (Median)	**Sand Filter Effluent (Median)**	**Percent Removal**
	31.6	14.0	56%
	Sand Filter Influent (GeoMean)	**Sand Filter Effluent (GeoMean)**	**Percent Removal**
	34.97	17.41	50%

Figure 5.17 shows sand filter removals plotted on a logarithmic scale. By taking out the two extreme high influent values (May 2004, 210,000 mg/L and September 2004, 75,000 mg/L) the data can be seen as in Figure 5.18 plotted on a standard arithmetic scale.

Table 5.5 shows that percent removals are in line with what would be expected per Table 2.2 of the *Literature Review* chapter of this paper, which reports removals averaging 48%.

05.01.06 Monitoring Available Oxygen / Oxygen Demand

Figure 5.20 shows Dissolved Oxygen (DO) concentration fluctuation as the water is processed throughout the facility.

DO concentrations before and after sand filter application are shown in Figure 5.21 compared to the desired level of 5.0 mg/L. As shown, DO is levels are raised as the water is processed through the sand filter. This occurs when the water is sprinkled onto the sand filter. The filter itself is not replenishing the oxygen; rather the water is absorbing oxygen it as it is sprinkled onto the filter's surface.

The average percent increase in DO is shown in Table 5.6.

Table 5.6

Sand Filter Removals of DO, BOD and COD			
Parameter	Sand Filter Influent (Mean)	Sand Filter Effluent (Mean)	Percent Removal
DO mg/L	2.87	6.76	-136%
BOD mg/L	20.31	4.81	76%
COD mg/L	190.1	91.2	52%
	Sand Filter Influent (Median)	Sand Filter Effluent (Median)	Percent Removal
DO mg/L	2.33	6.78	-191%
BOD mg/L	17	2	88%
COD mg/L	148.5	58.5	61%
	Sand Filter Influent (GeoMean)	Sand Filter Effluent (GeoMean)	Percent Removal
DO mg/L	1.90	6.57	-246%
BOD mg/L	15.95	3.41	79%
COD mg/L	162.3	67.6	58%

Chemical Oxygen Demand (COD_5) and Biochemical Oxygen Demand (BOD_5) measurements are illustrated in Figures 5.22 and 5.23. The figures show a fairly consistent removal rate for both.

Table 2.2 of the *Literature Review* chapter suggests that a 70% BOD removal rate can be expected; no separate data was found for expected COD removal rates. Average removal rates achieved by the sand filter at the Cloverdale site, as shown in Table 5.6, for BOD are above the expected value (76% mean).

05.01.07 Monitoring Oil and Grease

Observable Floating Oil and Grease was seen only 6 of the 17 reported sample dates, as illustrated in Figure 5.24.

Oil and Grease concentrations were not detected to be of much concern, as on most occasions, the measurements were below the detectable limit of the tests that were performed. The data is provided in Table 5.7.

Table 5.7

Oil & Grease Sample Data		
	Filter In	Filter Out
	mg/L	mg/L
Jul-03	<0.8	5.0
Aug-03	<11.6	<5.0
Sep-03	<11.6	<11.4
Oct-03	<11.1	<5.0
Nov-03	<5.0	<5.0
Mar-04	<5	<5
Apr-04	<5	<5
May-04	<5	<13.10
Jun-04	<5	<11.20
Jul-04	<5	<5
Aug-04	<5	<5
Sep-04	5.1	<5
Oct-04	<5	<5
Nov-04	<5	<5

< indicates concentration below detectable limit

05.01.08 Monitoring Total Metals

Total metals: Copper, Lead, and Zinc (Cu, Pb, Zn) removals, as shown in Table 5.8, are very good at the facility, especially for Zinc. An average of approximately 88% of the Zn concentration is removed by the sand filter.

Table 5.8

Sand Filter Removals of Total Metals

Parameter	Sand Filter Influent (Mean)	Sand Filter Effluent (Mean)	Percent Removal
Copper, µg/L	11.98	5.76	52%
Lead, µg/L	9.18	3.63	60%
Zinc, µg/L	101.9	13.3	87%
	Sand Filter Influent (Median)	**Sand Filter Effluent (Median)**	**Percent Removal**
Copper, µg/L	8.90	5.40	39%
Lead, µg/L	6.55	3.40	48%
Zinc, µg/L	86.75	10.25	88%
	Sand Filter Influent (GeoMean)	**Sand Filter Effluent (GeoMean)**	**Percent Removal**
Copper, µg/L	9.76	5.46	44%
Lead, µg/L	7.42	3.61	51%
Zinc, µg/L	86.32	10.46	88%

Figures 5.25, 5.26, and 5.27 show sand filter influent and effluent metal concentrations over time. The figures again highlight an important feature of the sand filter – not only can we expect high removal rates for total metals but we can expect relatively constant effluent concentrations regardless of influent concentrations. This phenomenon is discussed in greater detail in Chapter 2 of this paper and explained fully by Michael Barrett in his article *Performance, Cost, and Maintenance Requirements of Austin Sand Filters* (2003).

Examination of Chapter 2 (*Literature Review*) tables shows that the sand filter removal efficiencies found at the Cloverdale site are in line with what is expected given the size of the filter and relatively low application rate of storm water. Recall from Chapter 4 (*Design and Construction*) that the sand filter's loading rate is 0.021 gpm/ft^2.

This is at the low end of typical values for slow sand filter loading rates which range from 0.015 gpm/ft^2 to 0.16 gpm/ft^2. The literature review (see Chapter 2 Tables 2.1, 2.2, and 2.4) revealed that sand filters typically can be expected to remove between 45 and 90 percent of Total Metal concentrations. These results are in line with those for TSS removals, as expected since total metals make up part of the TSS concentration.

05.01.09 Monitoring Pathogens with Coliforms

Total coliforms were measured only at the basin outlet (sand filter influent) and at the storage tank. If needed the storage tank could be supplemented with water from the City water supply, therefore these measurements do not accurately reflect sand filter efficiencies – as City water contains chlorine which would kill pathogens and alter coliform counts accordingly.

Figure 5.28 shows total coliform removals, on a log scale, for the sand filter using the tank data as effluent data. Removal rates appear to be fairly consistent – the assumption can be made that on or near the dates where the low points for the effluent data are shown City water was added to the tank.

Similarly, fecal coliforms were measured only at the basin outlet (sand filter influent) and at the storage tank.

Figure 5.29 shows fecal coliform removals for the sand filter also using tank data as effluent data. Removal rates shown correspond to those illustrated for total coliforms in Figure 5.28.

Information obtained in the literature review on coliform removals by a sand filter was specific to fecal coliforms. As shown in Tables 2.2, 2.4, and2.5 of the *Literature*

Review section of this paper, fecal coliform removals of 65 to 70 percent can be expected – with *Literature Review* Table 2.5 showing extremely high removal rates (96 to 98 percent) for rapid and intermittent sand filters.

Table 5.9 shows removal efficiencies for total and fecal coliforms through the sand filter at the Cloverdale site. The removal efficiencies are very high – however, the data is somewhat unreliable due to the effluent measurements being taken at the tank rather than the sand filter outlet.

Table 5.9

Total and Fecal Coliform Removals in Sand Filter

Parameter	Sand Filter Influent (Mean)	Storage Tank Effluent (Mean)	Percent Removal
Total Coliforms, CFU/100mL	1250625	110299	91%
Fecal Coliforms, CFU/100mL	5621	454	92%
	Sand Filter Influent (Median)	**Storage Tank Effluent (Median)**	**Percent Removal**
Total Coliforms, CFU/100mL	75000	625	99%
Fecal Coliforms, CFU/100mL	4150	10	99.8%
	Sand Filter Influent (GeoMean)	**Storage Tank Effluent (GeoMean)**	**Percent Removal**
Total Coliforms, CFU/100mL	50765	532	99%
Fecal Coliforms, CFU/100mL	2387	41	98%

E-coli measurements were taken at the actual sand filter outlet, so the best coliform data available is on e-coli.

Comparison of Figure 5.30 with Figures 5.28 and 5.29 shows that the coliform decrease in the spring of 2004 can be attributed to sand filter removals, rather than to chlorine present in the storage tank. The assumption can be made here that it took the sand filter approximately 1 year before the *Schmutzdecke* had sufficiently developed at the surface, and that is the reason for the delay in effective pathogen removal rates.

Table 5.10 provides average removal rates of e-coli by the sand filter at the Cloverdale facility.

Table 5.10

E-Coli Removals in Sand Filter

Parameter	Sand Filter Influent (Mean)	Sand Filter Effluent (Mean)	Percent Removal
E-Coli, CFU/100mL	4999	622	88%
	Sand Filter Influent (Median)	**Sand Filter Effluent (Median)**	**Percent Removal**
	700	17	98%
	Sand Filter Influent (GeoMean)	**Sand Filter Effluent (GeoMean)**	**Percent Removal**
	590	30	95%

Idaho Administrative Code Section 58.01.02 Water Quality Standards provides standards for acceptable e coli levels in waters with recreational use designations. Section 251.01 states the following:

> Waters designated for primary or secondary contact recreation are to contain E coli bacteria in concentrations exceeding a geometric mean of 126 e coli organisms per 100 mL based on a minimum of 5 samples taken every three to seven days...
>
> For waters designated as secondary contact recreation, a single sample maximum of 576 e coli organisms per 100 mL; or
>
> For water designated as primary contact recreation, a single sample maximum for 406 e coli organisms per 100 mL; or

For areas within waters designated as primary contact recreation that are additionally specified as public swimming beaches, a single sample maximum of 235 e coli organisms per 100 mL. Single sample counts above this value should be used in considering beach closures.

Based on this information, ANY e coli present in water can present some safety concerns. The water used at the Cloverdale facility does not, however, fall under the category of waters for recreational use. The water is not likely to be ingested, and all taps into the treated water supply are marked "Non- Potable. Do Not Drink!"

Using these numbers as guidelines, and Table 5.10, we can see that levels of e coli in the water are, mean values, low enough to not cause concern. Even the average value of 622 CFU/100mL is only slightly higher than the recommended maximum for waters designated as secondary contact recreation (576 organisms/100mL).

05.01.10 Monitoring Hydrogen Sulfide (H_2S) and Volatile Organic Compounds (VOCs)

Hydrogen Sulfide (H_2S) Headspace was measured at the basin outlet and at the sand filter outlet. No H_2S headspace measurement reached a detectable level above normal background levels during any sampling event.

VOC Headspace was also measured at the basin outlet and at the sand filter outlet. Again, no VOC headspace measurement reached a detectable level above normal background levels during any sampling event.

05.02 Summary

The sand filter at the ACHD Cloverdale facility performs well. Pollutant removals are as expected and in some cases better than expected. NPDES compliance is met by monitoring effluent in this manner and reporting on the findings annually. There are currently no water reuse regulations applicable to this site. A summary of the findings is presented below:

- Total Suspended Solids – The sand filter shows removals averaging 95%, with an average effluent concentration of 4.9 mg/L. Sand filters are expected to perform well at filtering solids, and this filter does not disappoint.
- Phosphorous and Nitrogen – Nutrient removal trough the sand filter is as expected per the literature review. For total phosphorus we have removals averaging 56%, which is moderate but right on track with what was suggested in the literature (30 – 75%). And for total Kjeldahl nitrogen we see about the same, removals averaging about 63% where literature suggests between 60 and 75% can be expected.
- Total Organic Carbon – Similarly, TOC removals are as expected per the literature review. Literature suggests removals averaging 48%, and results from this study show removals averaging about 43%.
- Dissolved Oxygen – DO levels for sand filter effluent are very good. The application method (sprinkling) provides good aeration for the treated water before entering the sand filter. Basin effluent levels and sand filter effluent

levels show that the amount of DO more than doubles between the two stages of treatment.

- Biochemical Oxygen Demand, Chemical Oxygen Demand – removal rates for BOD and COD are in line with what was expected per literature reviews. Literature suggests 70% removal of BOD and Table 5.6 shows an average 76% removal. No data was found specific to COD in the literature review, however, removal rates at the Cloverdale facility are moderate – averaging 52%.
- Oil and Grease – The presence of oil and grease was not found to be a concern.
- Metals – Literature suggests that between 45 and 90% of total metal concentrations can be expected to be removed by sand filtration. This is as expected for solids removals, since total metals are included in the total suspended solids of the water being treated. At the Cloverdale facility, as shown in Table 5.8, Copper removals average 52%, Lead 60%, and Zinc 87%. This is very good.
- Coliforms – The sand filter provides excellent removals of coliform bacteria. E. Coli being the coliform of primary concern, it can be seen from Table 5.10 that removal rates are very high – averaging 88%. Effluent concentrations are near or below levels recommended for waters designated for recreational use (which this water is NOT), thereby alleviating some health and safety concerns about the facility.

- Hydrogen Sulfide and Volatile Organic Compounds – The presence of H_2S and VOCs was found not to be an issue.

Overall, the sand filter is performing as expected, and in some cases (as with TSS) even better than expected. Pollutant removals are good; NPDES requirements are satisfied; and effluent levels are acceptable for reuse of the treated water.

05.03 Chapter 5, Figures

Figure 5.1:. Basin Outlet Sampling Site, photo taken facing NW (photo by Monica Lowe/ACHD 2003)

Figure 5.2: Sand filter with storage tank in background, photo taken facing North (photo by Monica Lowe/ACHD 2003)

Figure 5.3: Filter sampling point, photo taken looking down with cover removed (photo by Monica Lowe/ACHD 2003)

Figure 5.4: Tank sampling point, photo taken facing North (photo by Monica Lowe/ACHD 2003)

Figure 5.5: Evan's Drain Outfall, photo taken facing East (photo by Monica Lowe/ACHD 2003)

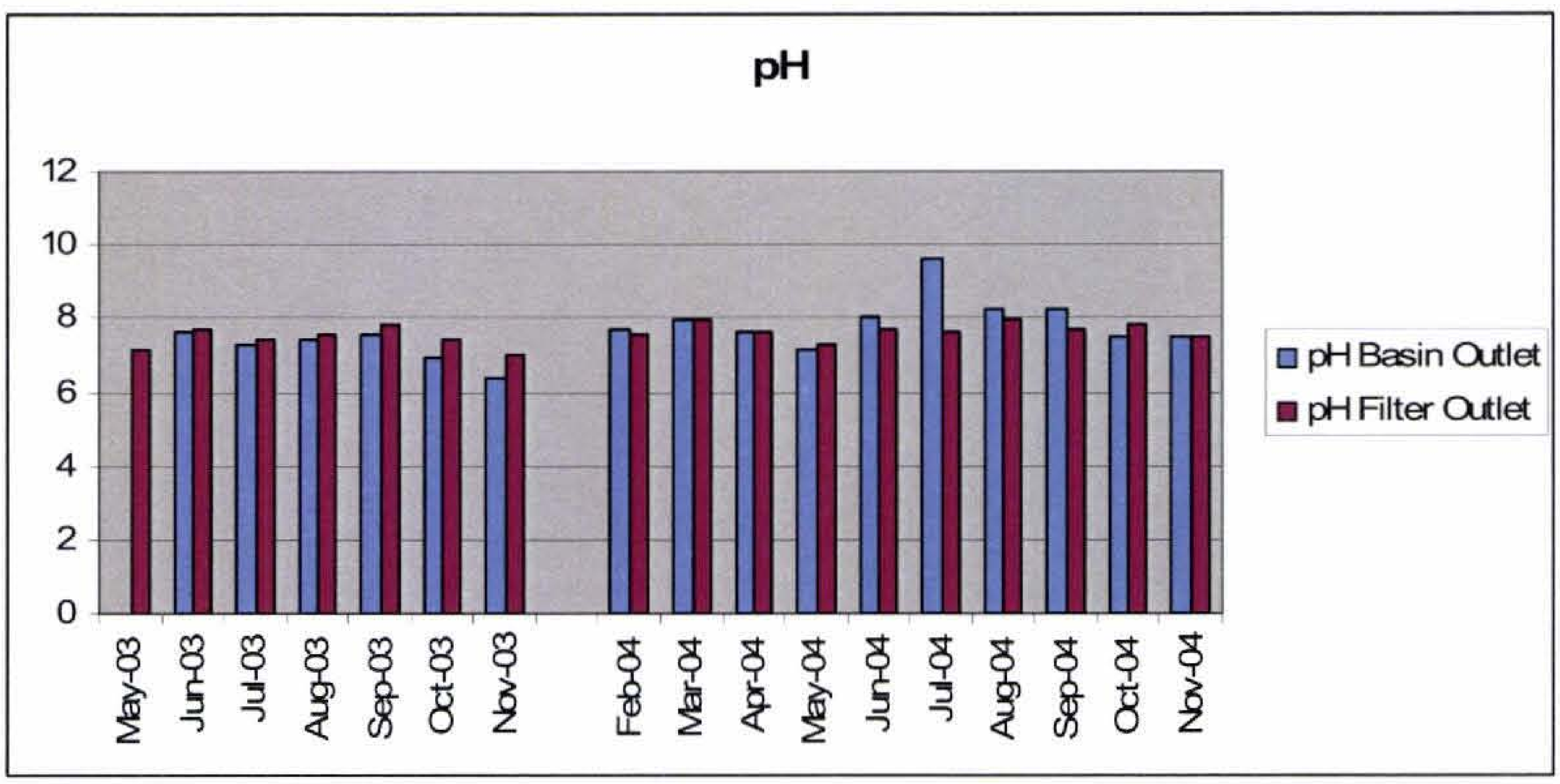


Figure 5.6: pH variation by month at basin outlet and at filter outlet

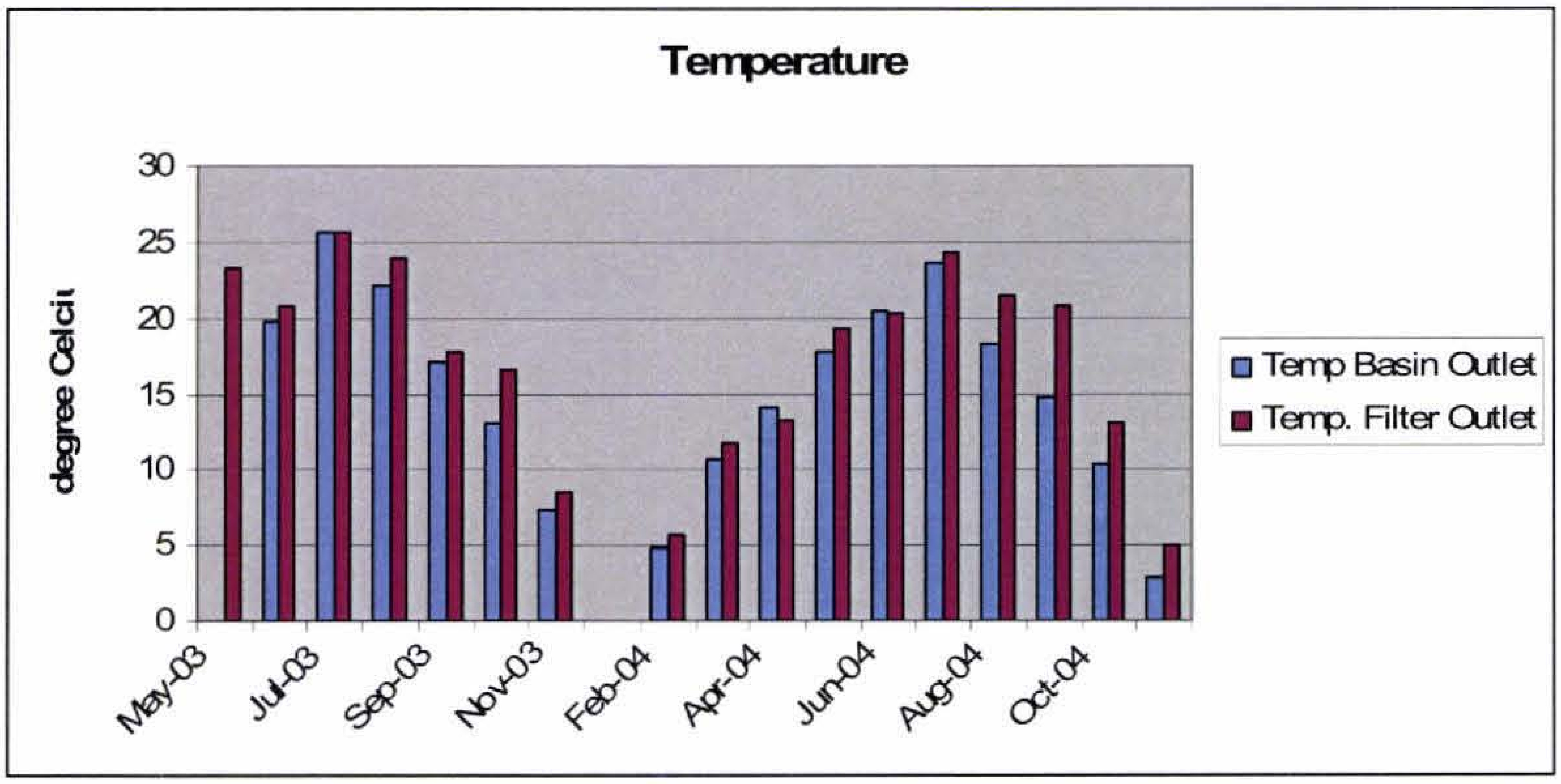


Figure 5.7: Temperature variation by month at basin outlet and at filter outlet

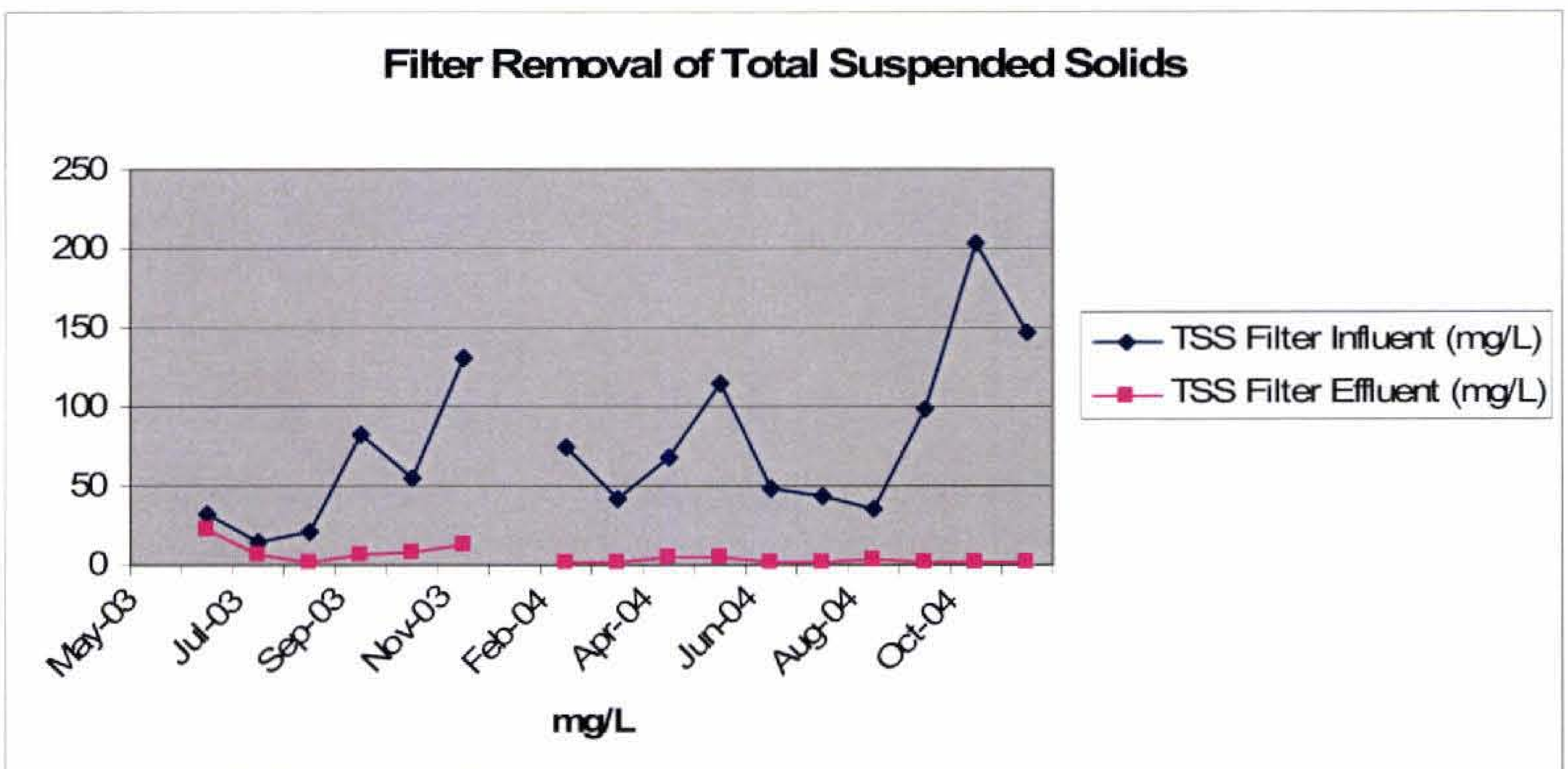


Figure 5.8: TSS removal by sand filter at Cloverdale Site 2003 - 2004

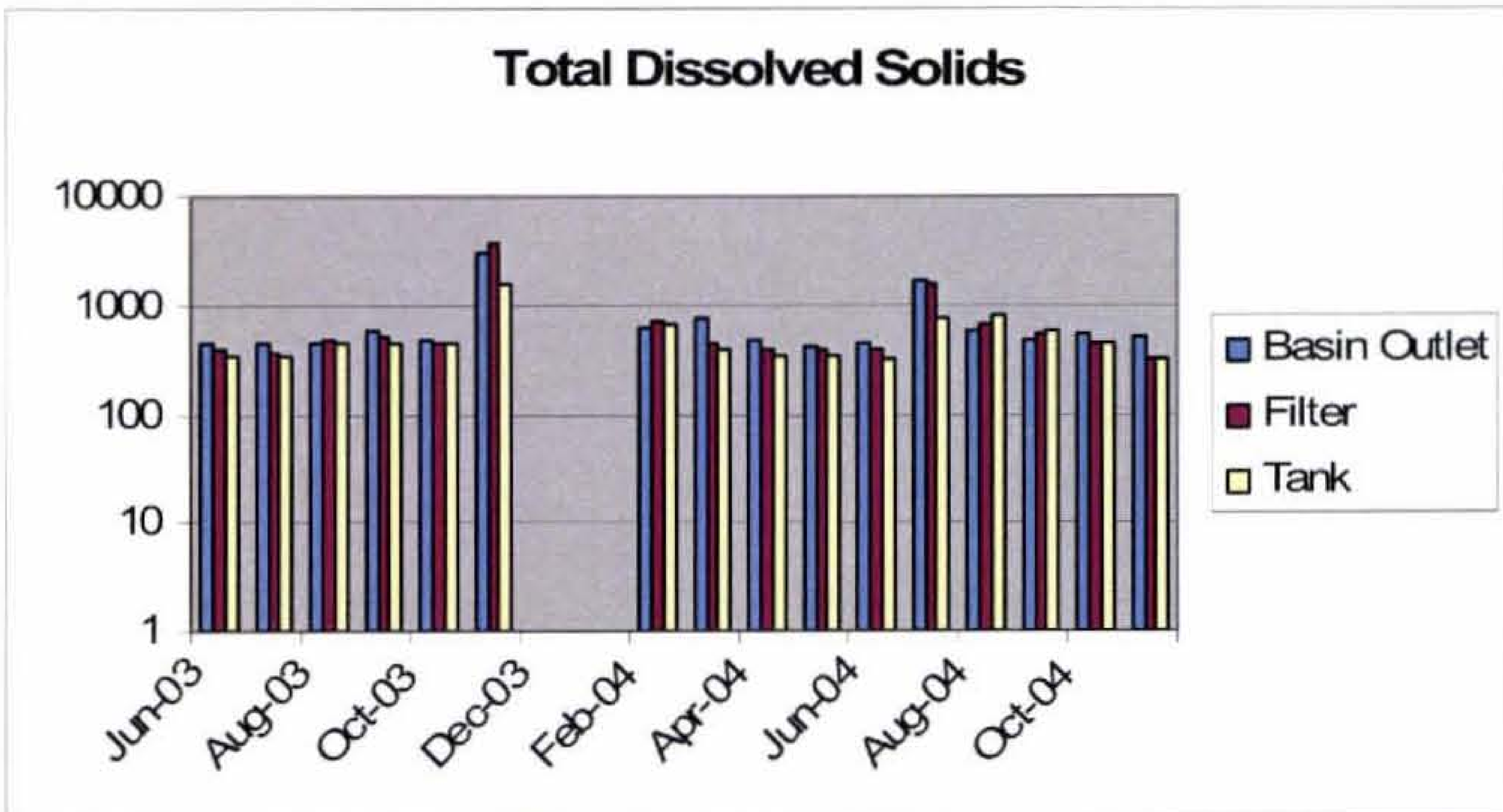


Figure 5.9: log-scale chart of TDS concentrations across the Cloverdale facility

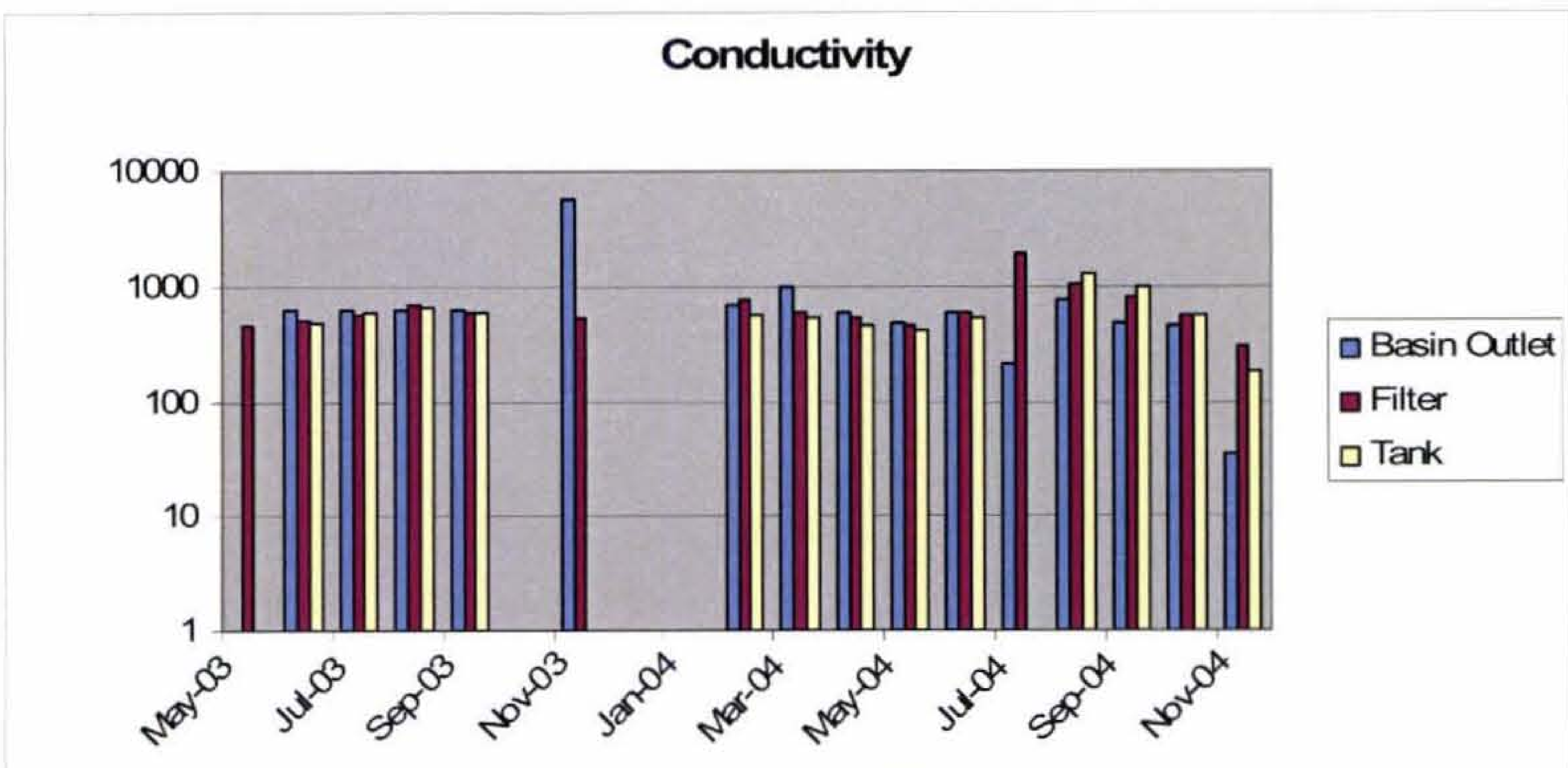


Figure 5.10: log-scale chart of Conductivity measurements across the Cloverdale facility

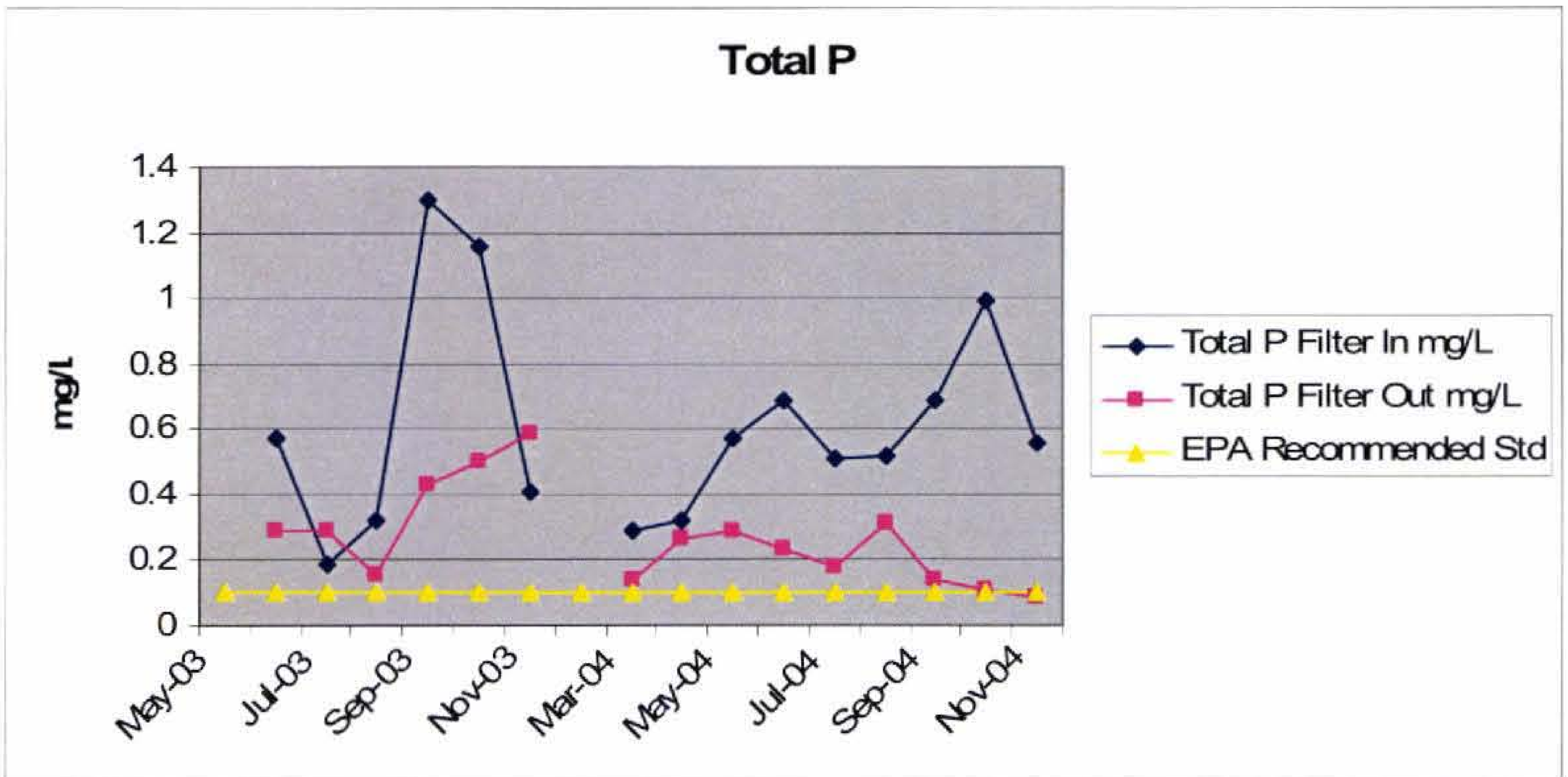


Figure 5.11: Total phosphorous removal by sand filter at Cloverdale site 2003-2004.

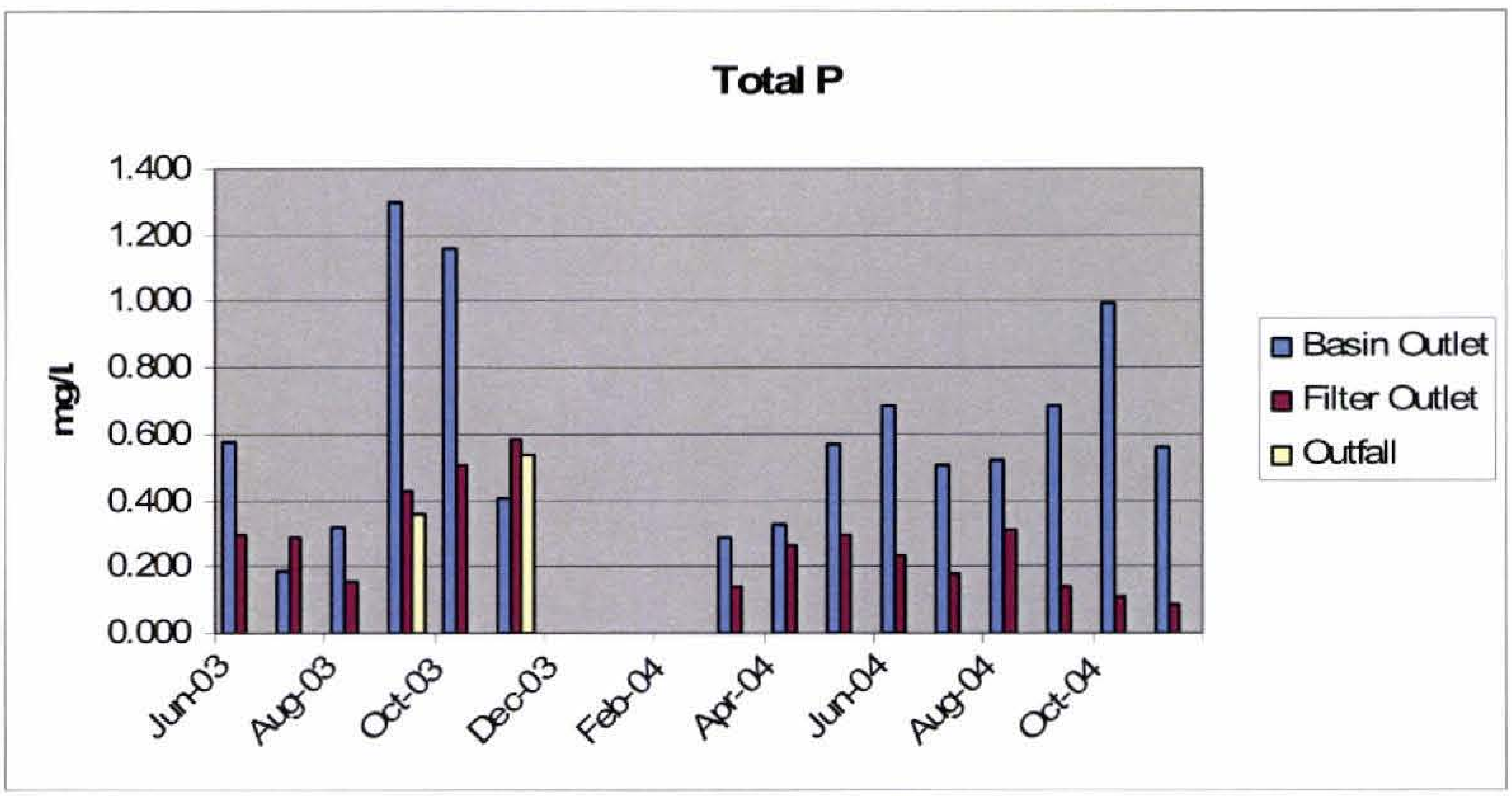


Figure 5.12: Total phosphorous concentrations across Cloverdale facility.

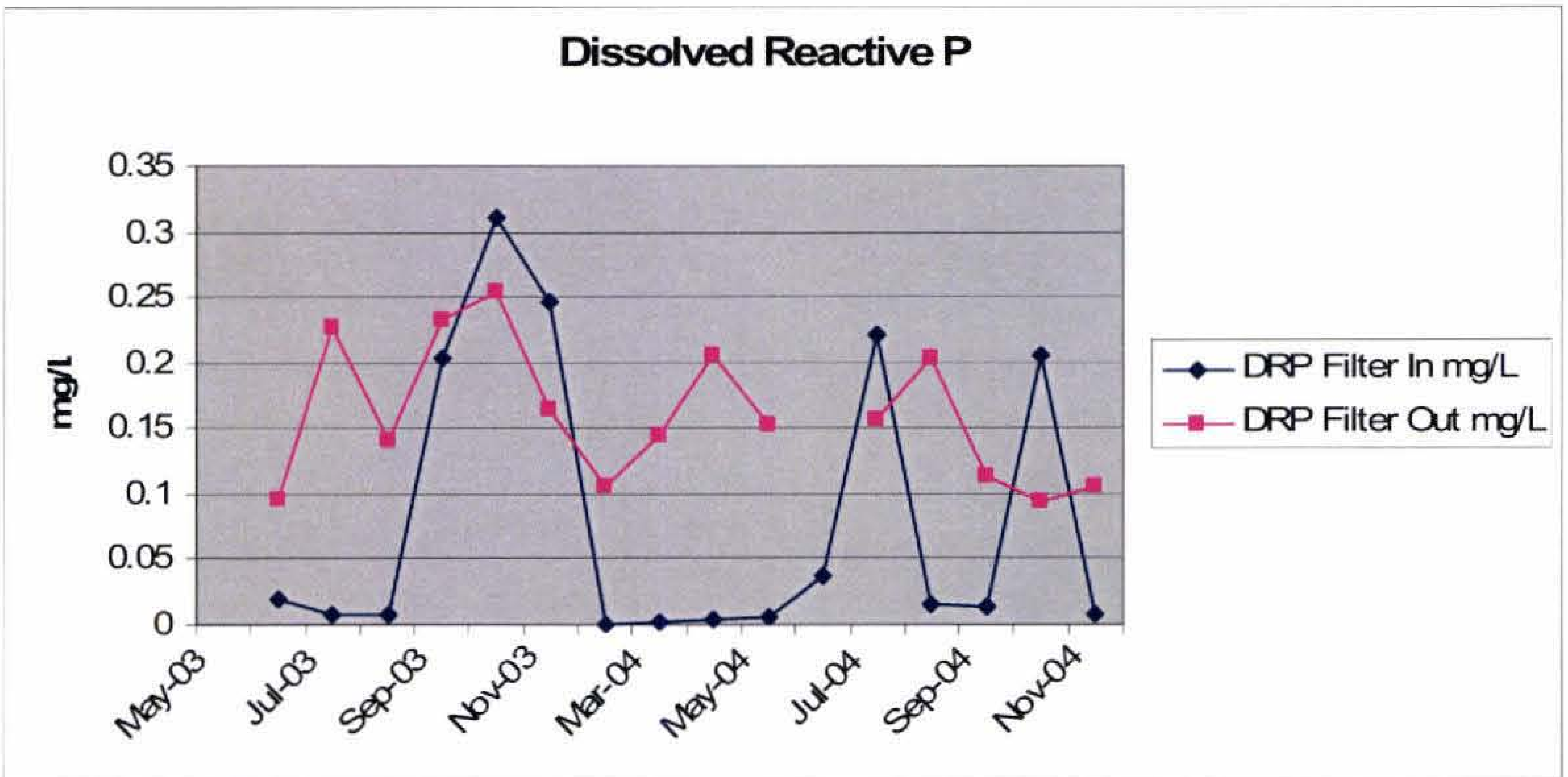


Figure5.13: DRP concentrations before and after sand filter application at Cloverdale facility.

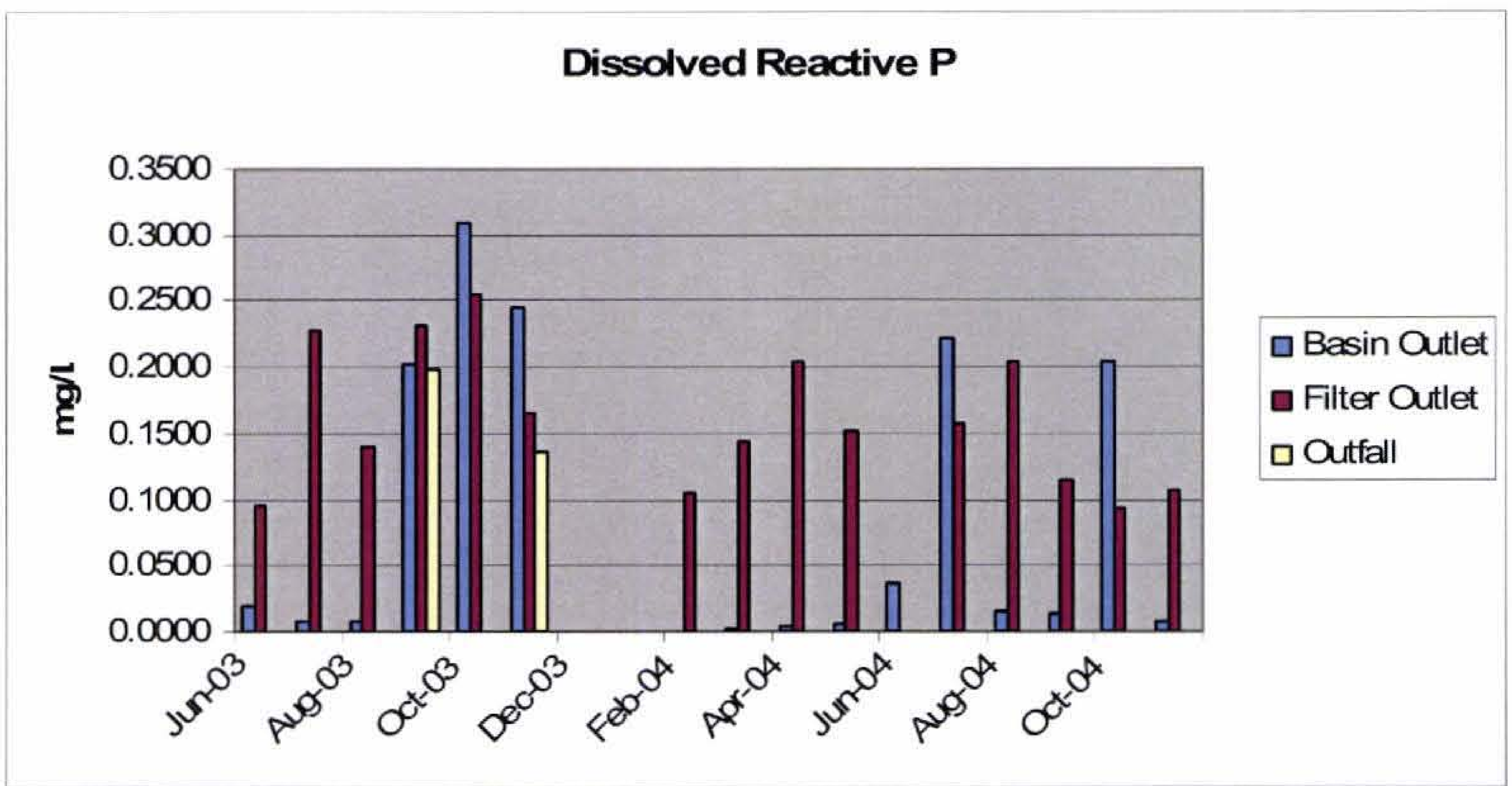


Figure 5.14: DRP concentrations across Cloverdale facility.

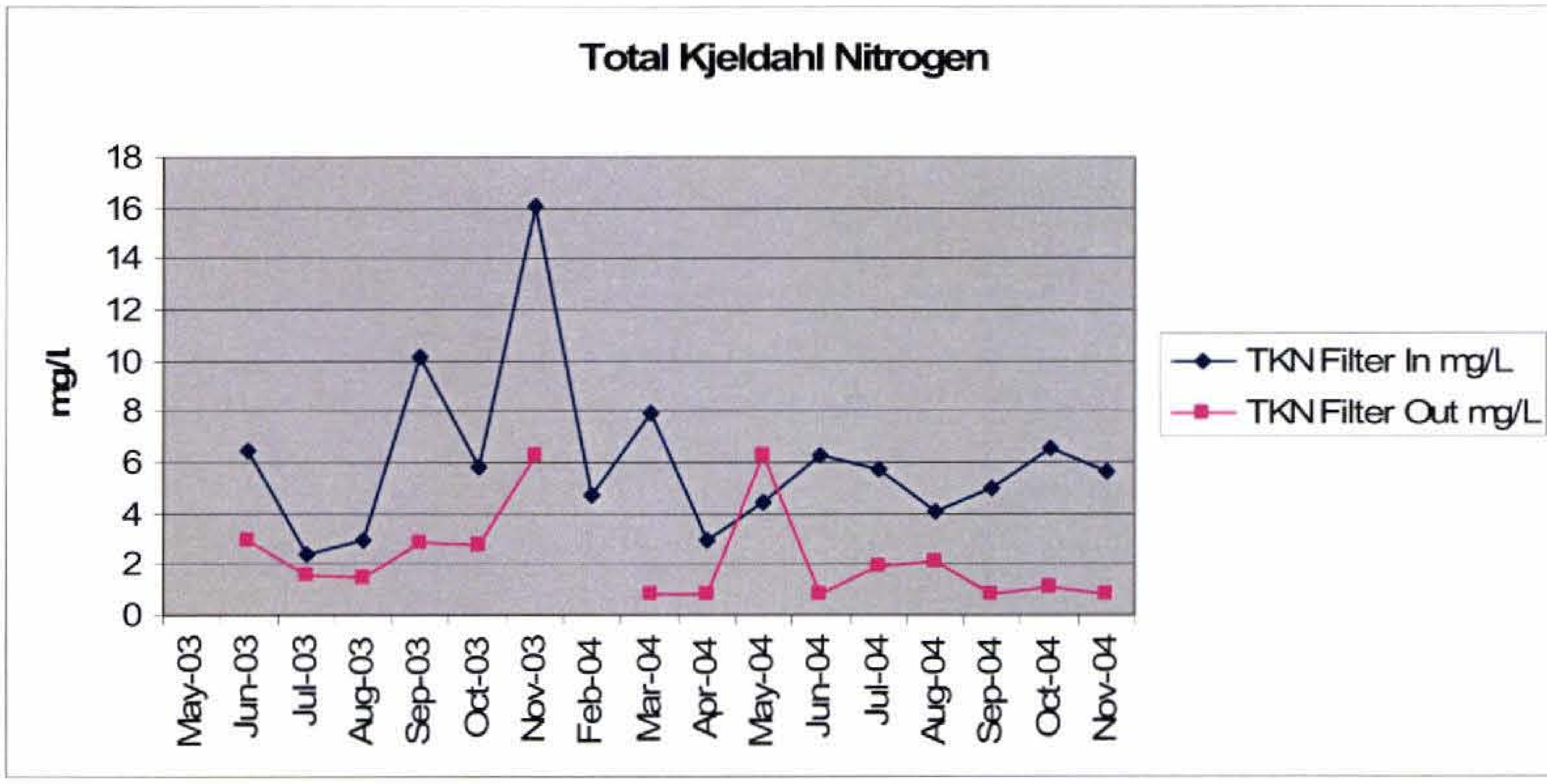


Figure 5.15: TKN removal through sand filter at Cloverdale site 2003 - 2004.

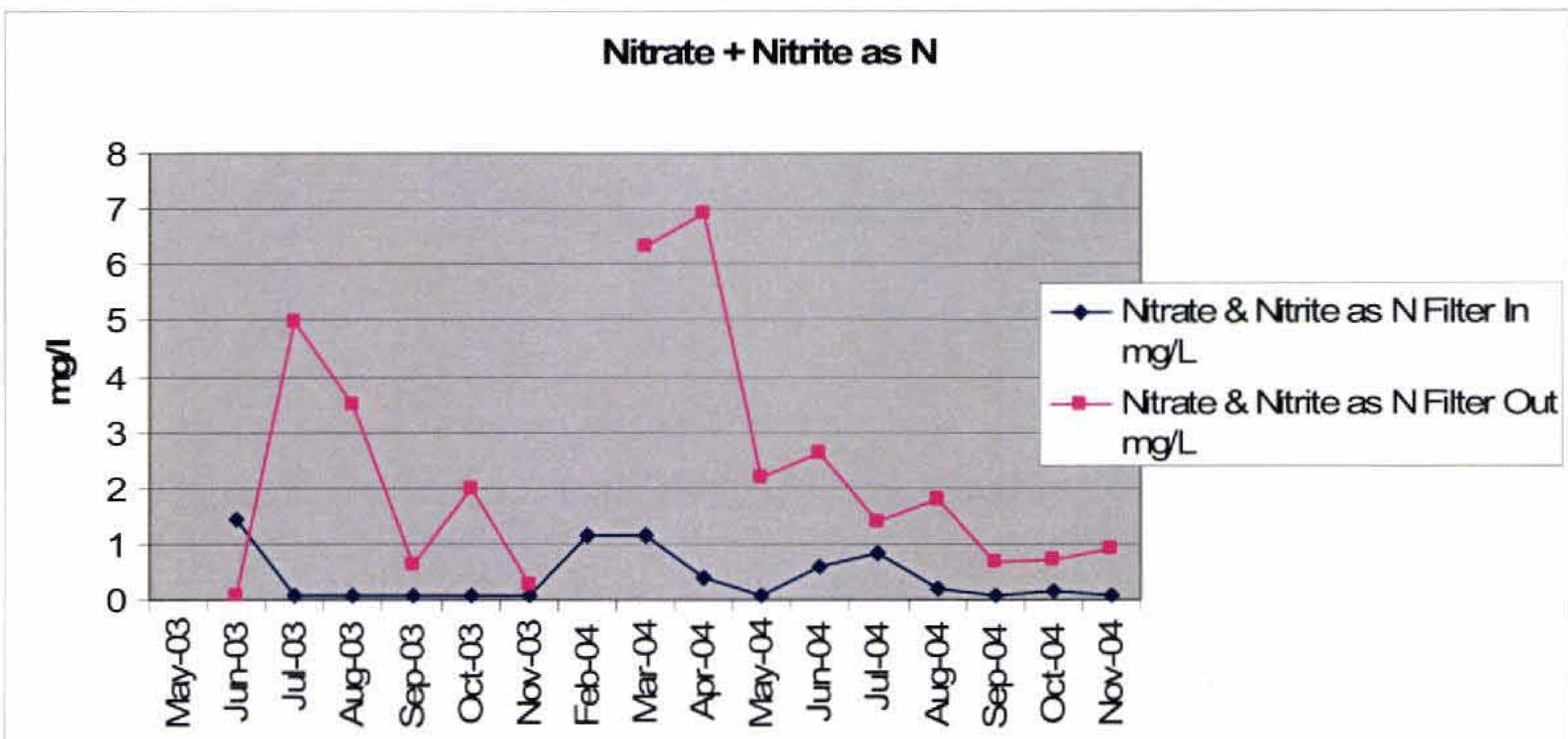


Figure 5.16: Nitrate + Nitrite removal (actually increase) through sand filter at Cloverdale site 2003 - 2004.

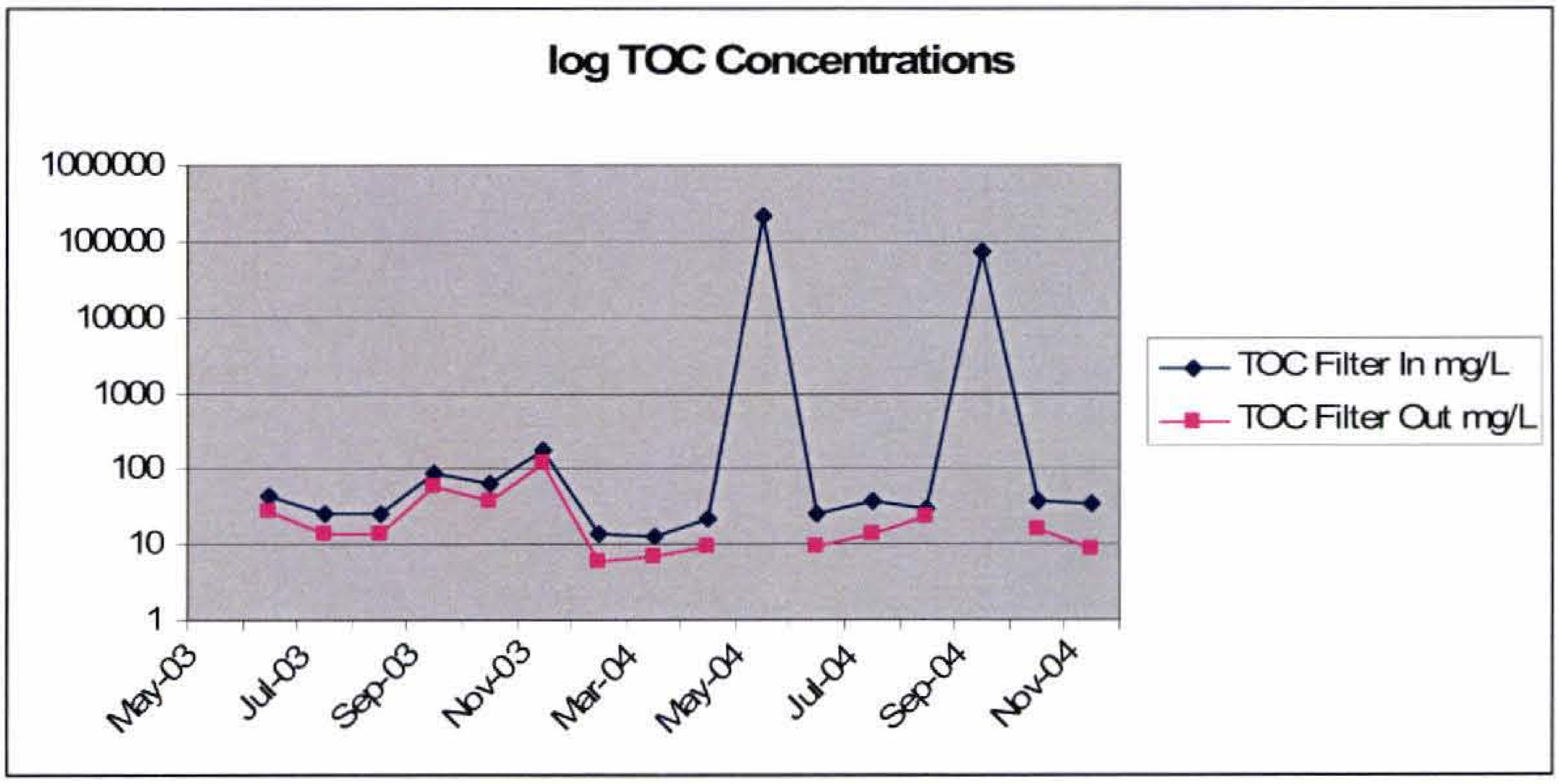


Figure 5.17: TOC removal through sand filter at Cloverdale site 2003 - 2004 (log-scale).

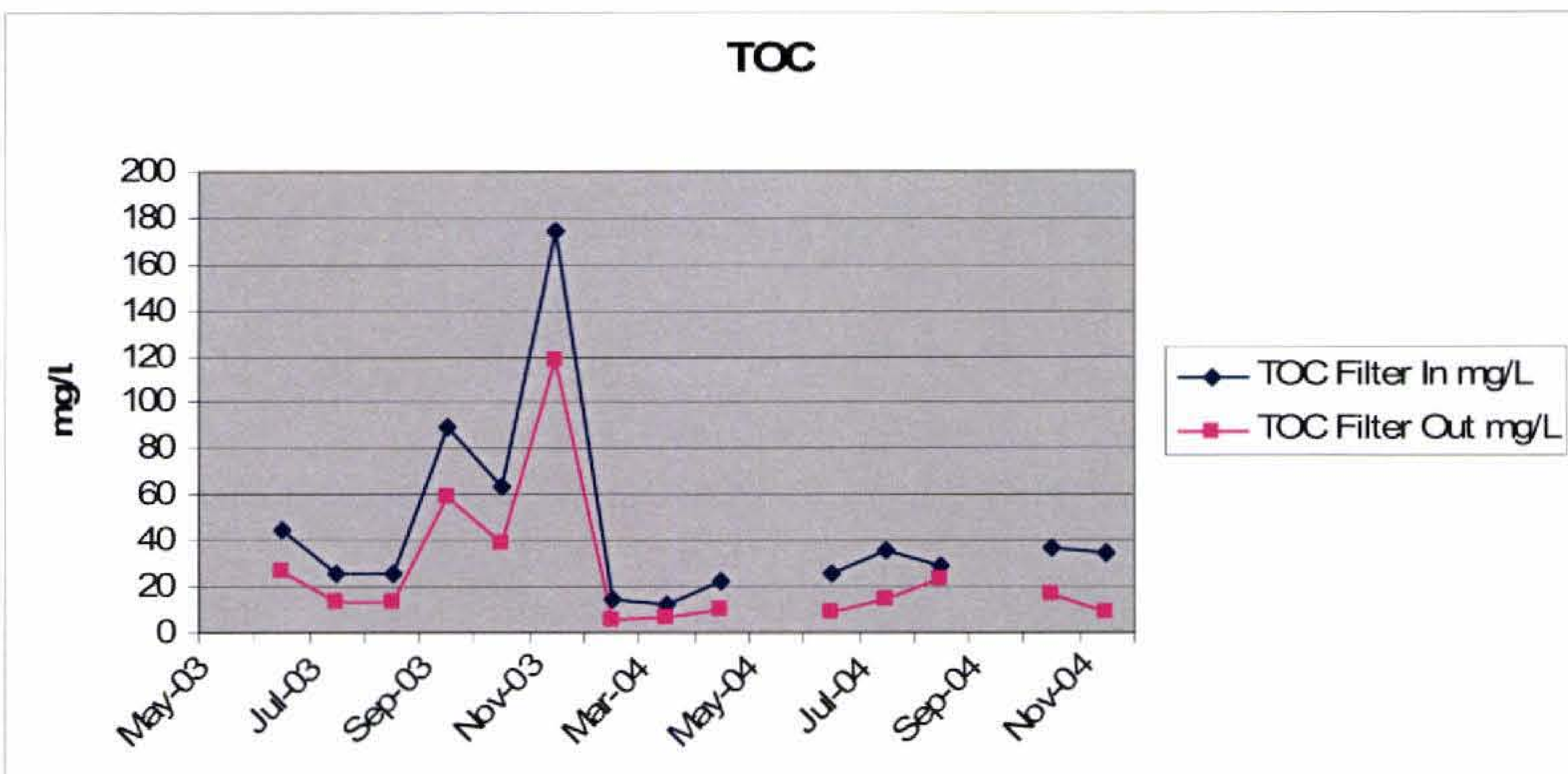


Figure 5.18: TOC removal through sand filter at Cloverdale site 2003 - 2004, not including extreme values.

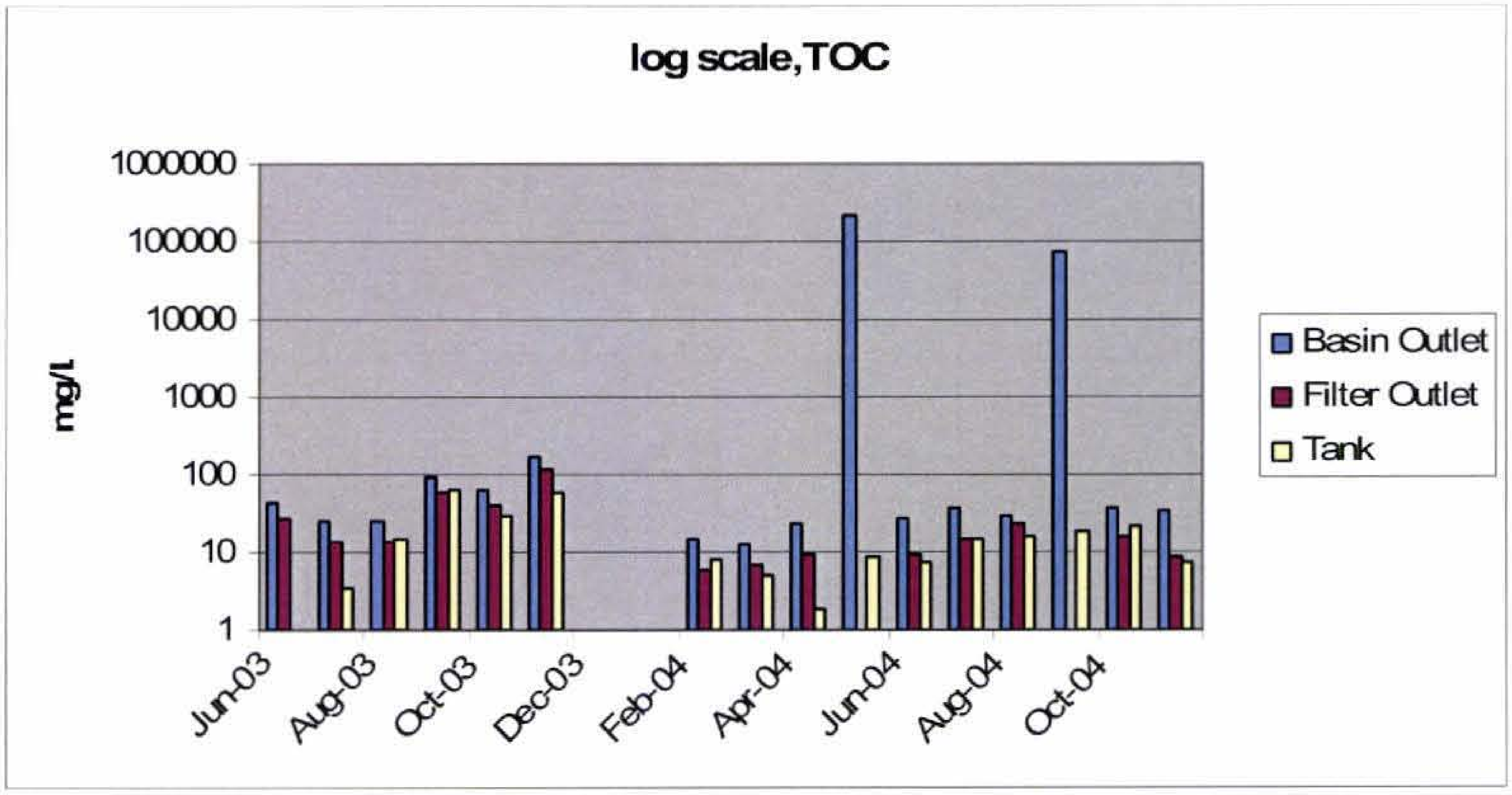


Figure 5.19: TOC concentrations across Cloverdale facility (log-scale).

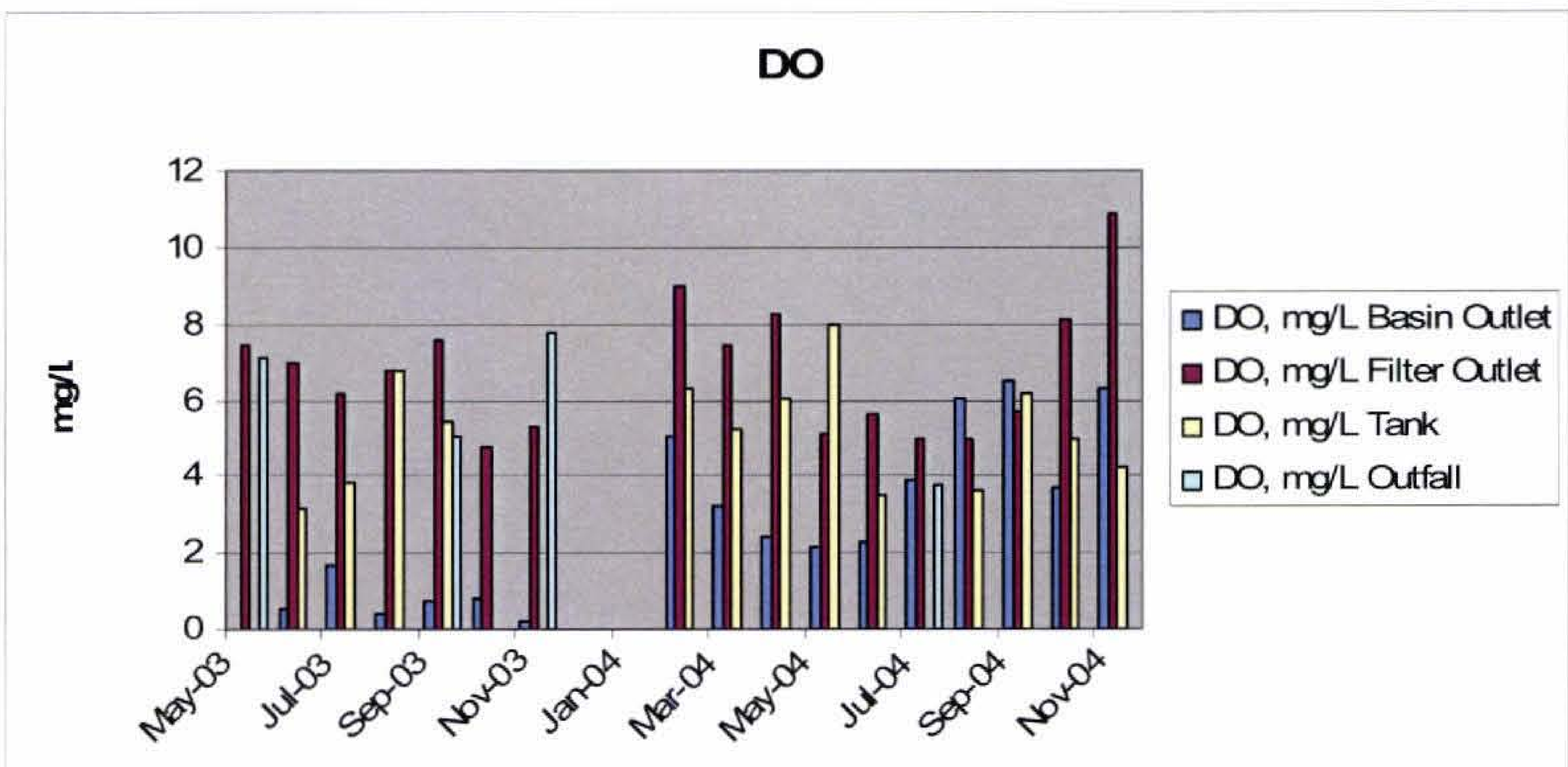


Figure 5.20: Dissolved oxygen fluctuations across Cloverdale facility.

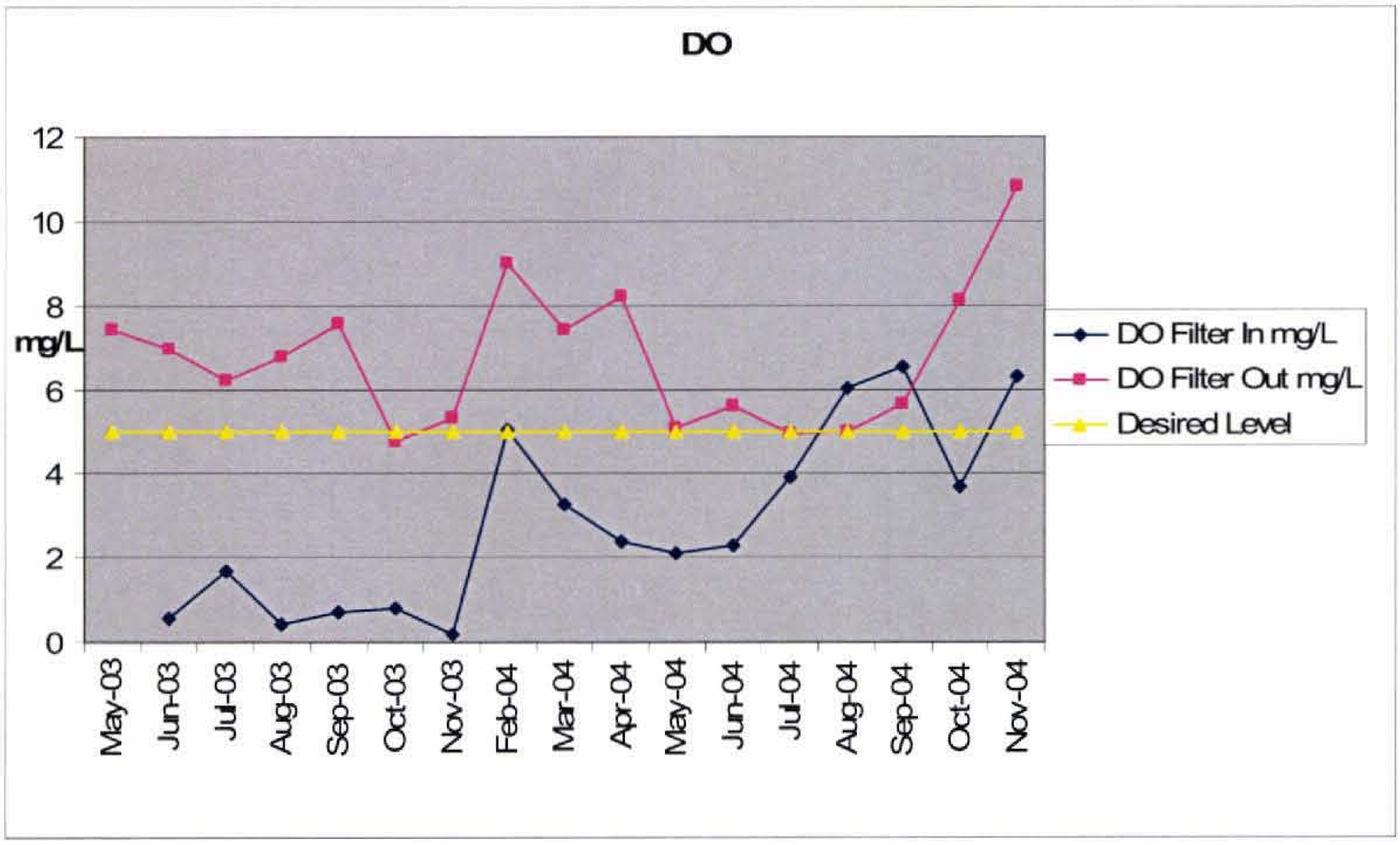


Figure 5.21: Dissolved oxygen concentrations before and after sand filter application shown compared to the 5.0 mg/L level desired for support of aquatic life.

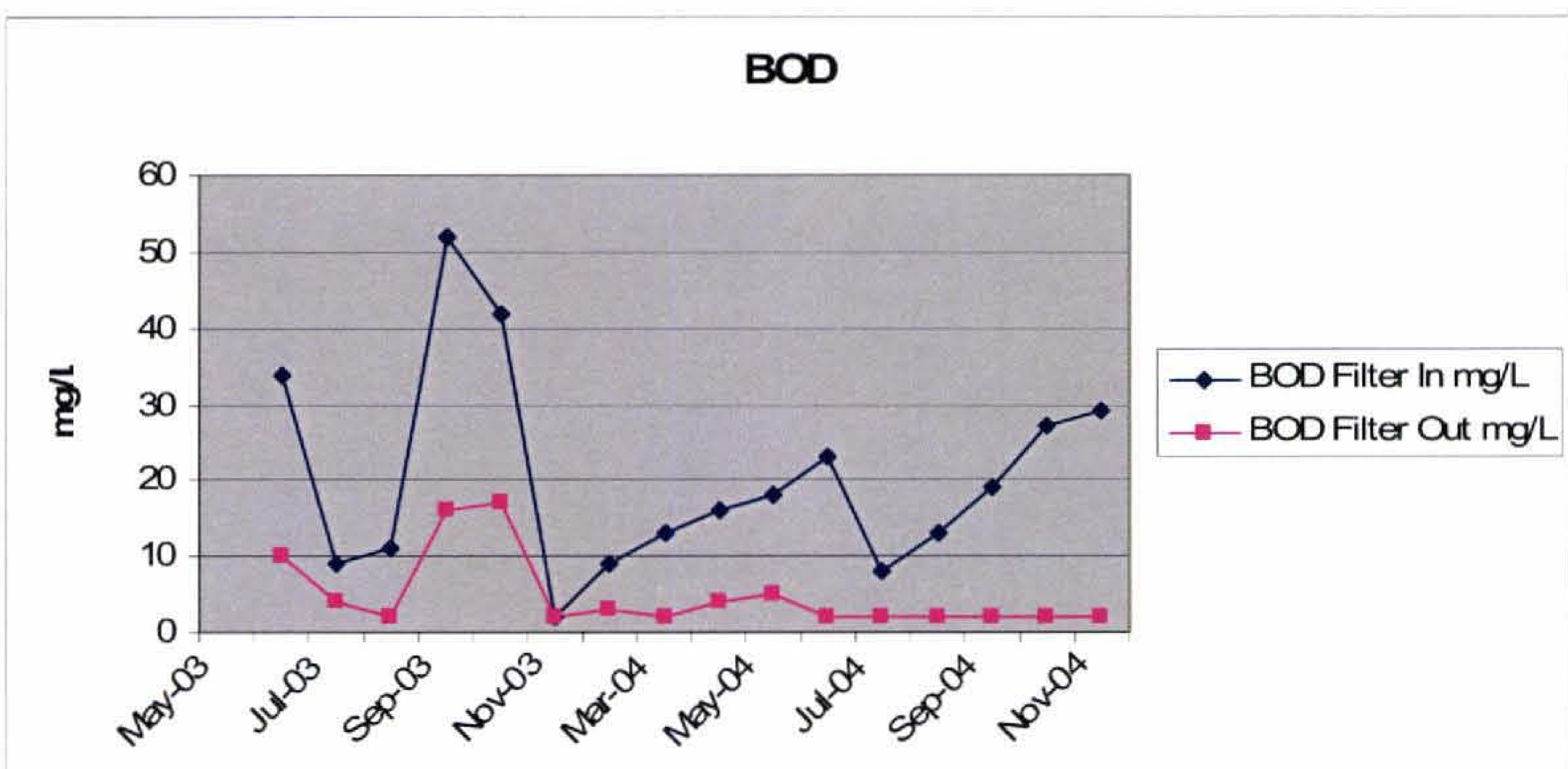


Figure 5.22: Biochemical oxygen demand before and after sand filter application.

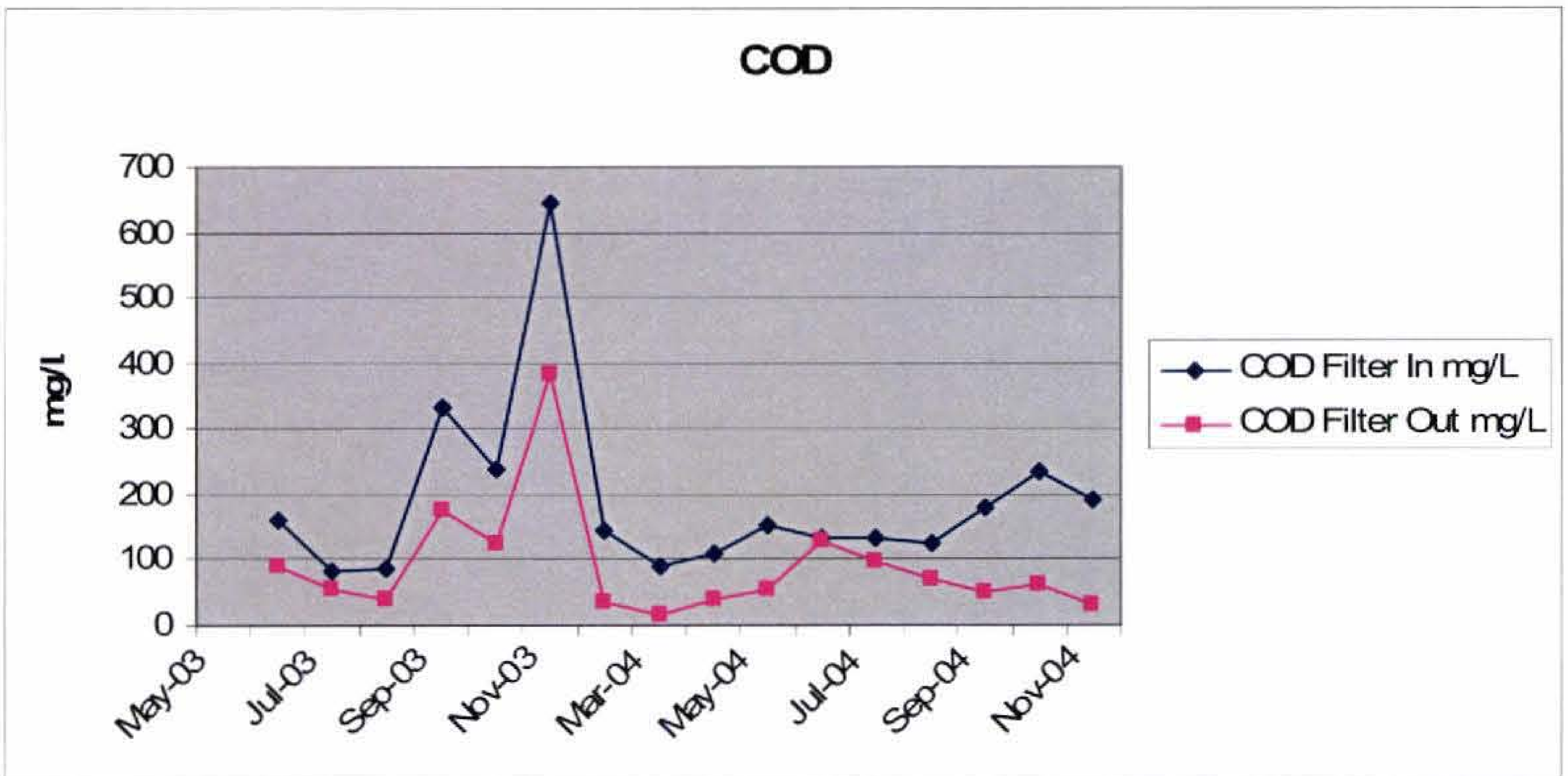


Figure 5.23: Chemical oxygen demand before and after sand filter application.

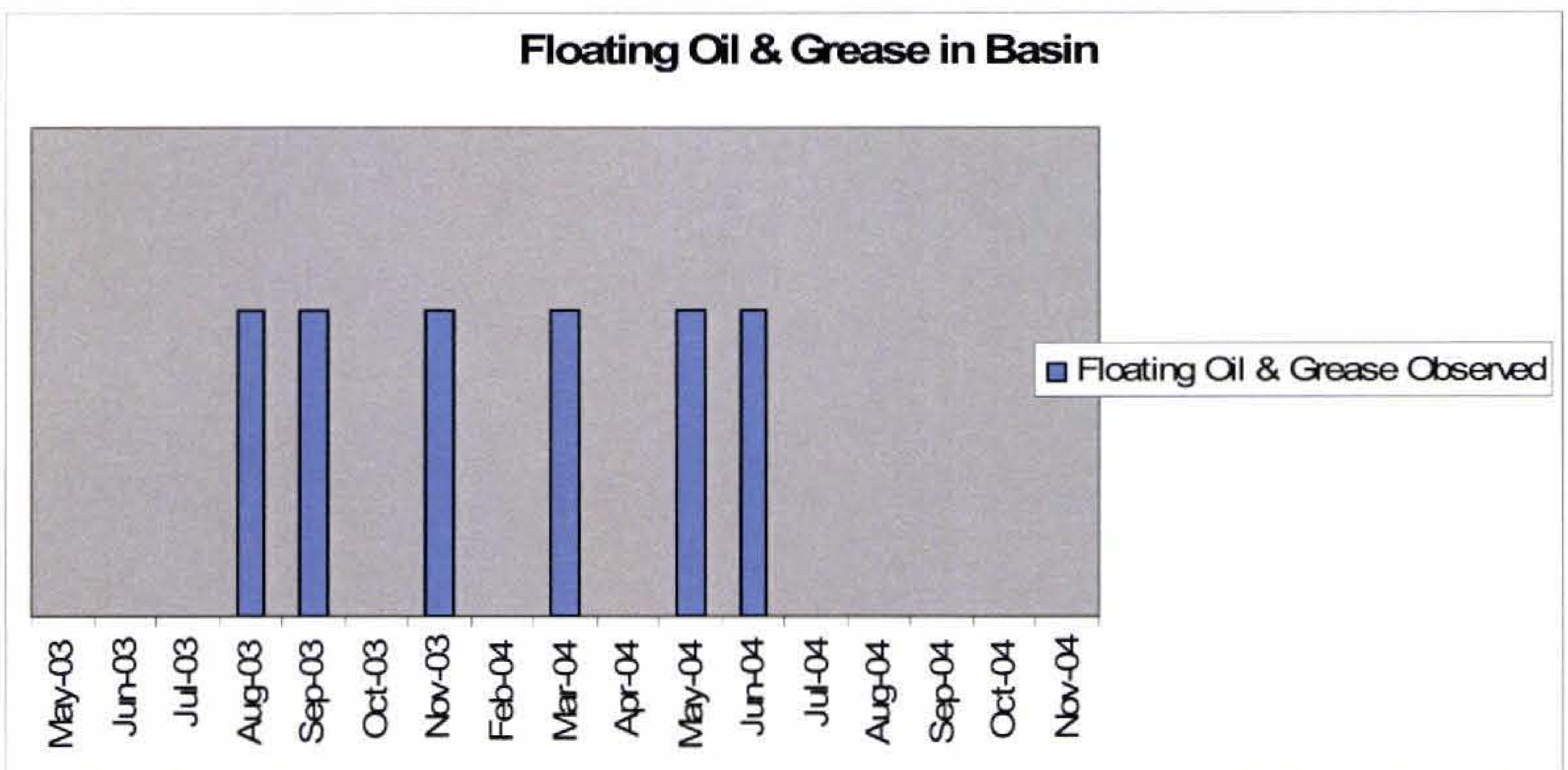


Figure 5.24: Sample dates when visual sheen from oil and grease was present in basin.

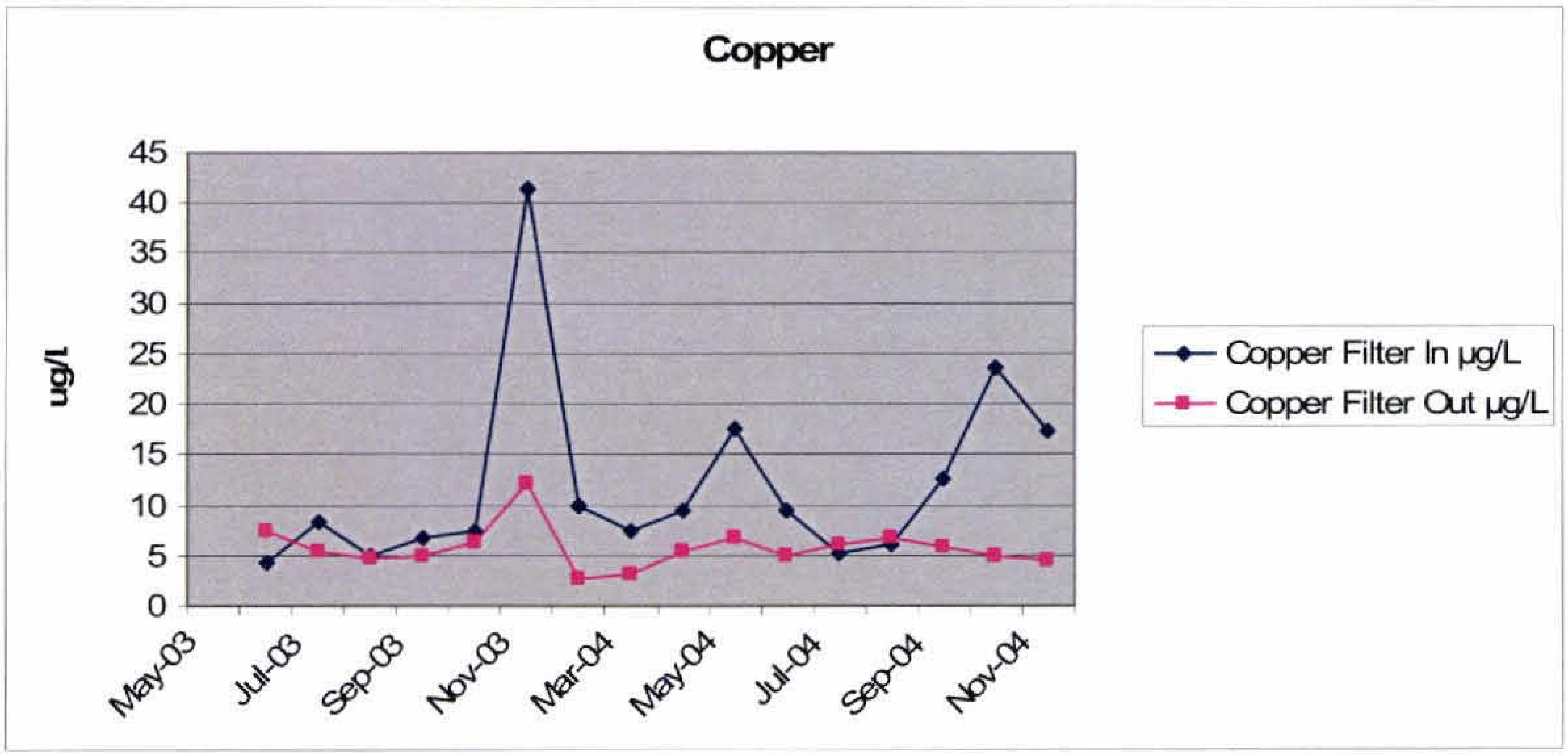


Figure 5.25: Total copper concentrations before and after sand filter application of storm water.

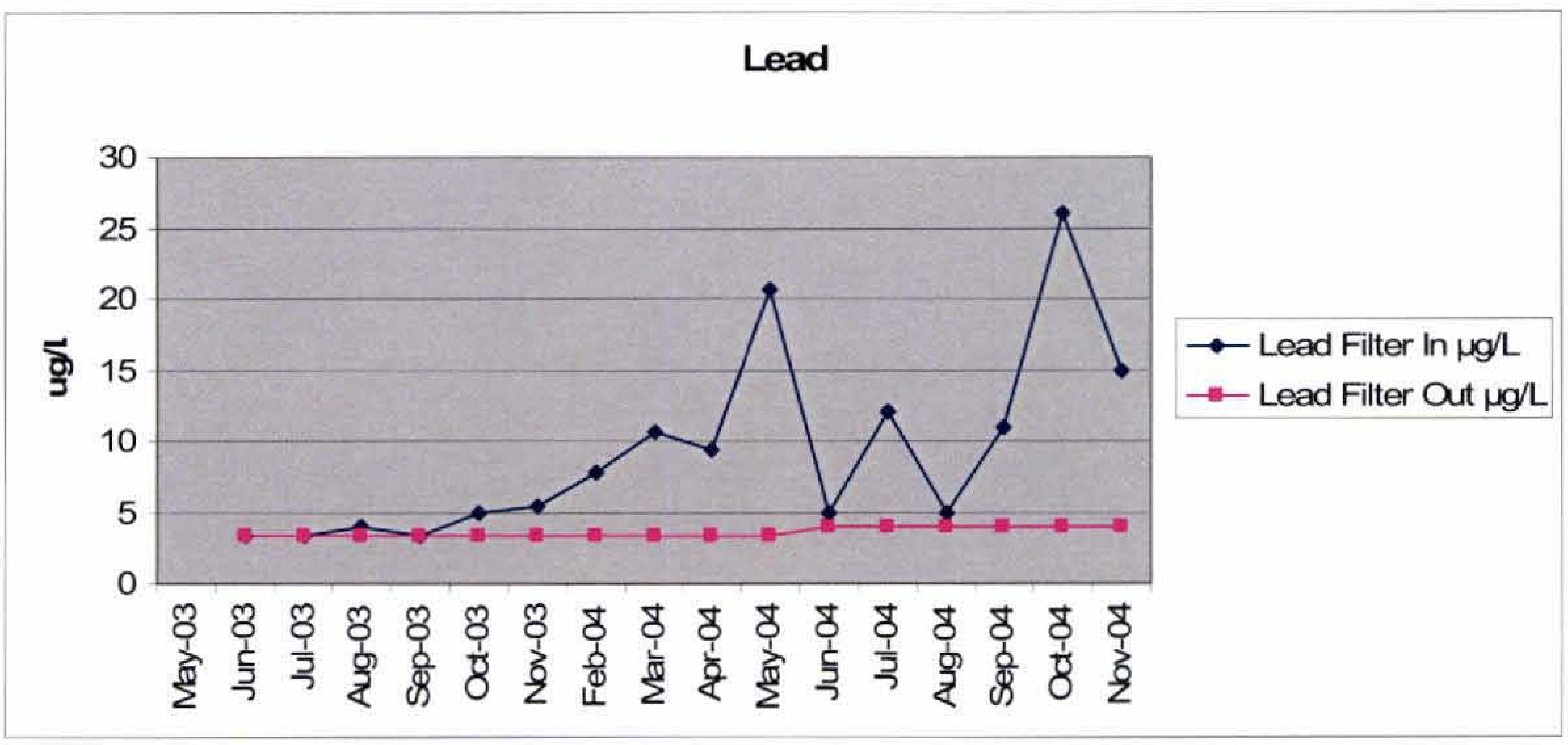


Figure 5.26: Total lead concentrations before and after sand filter application of storm water.

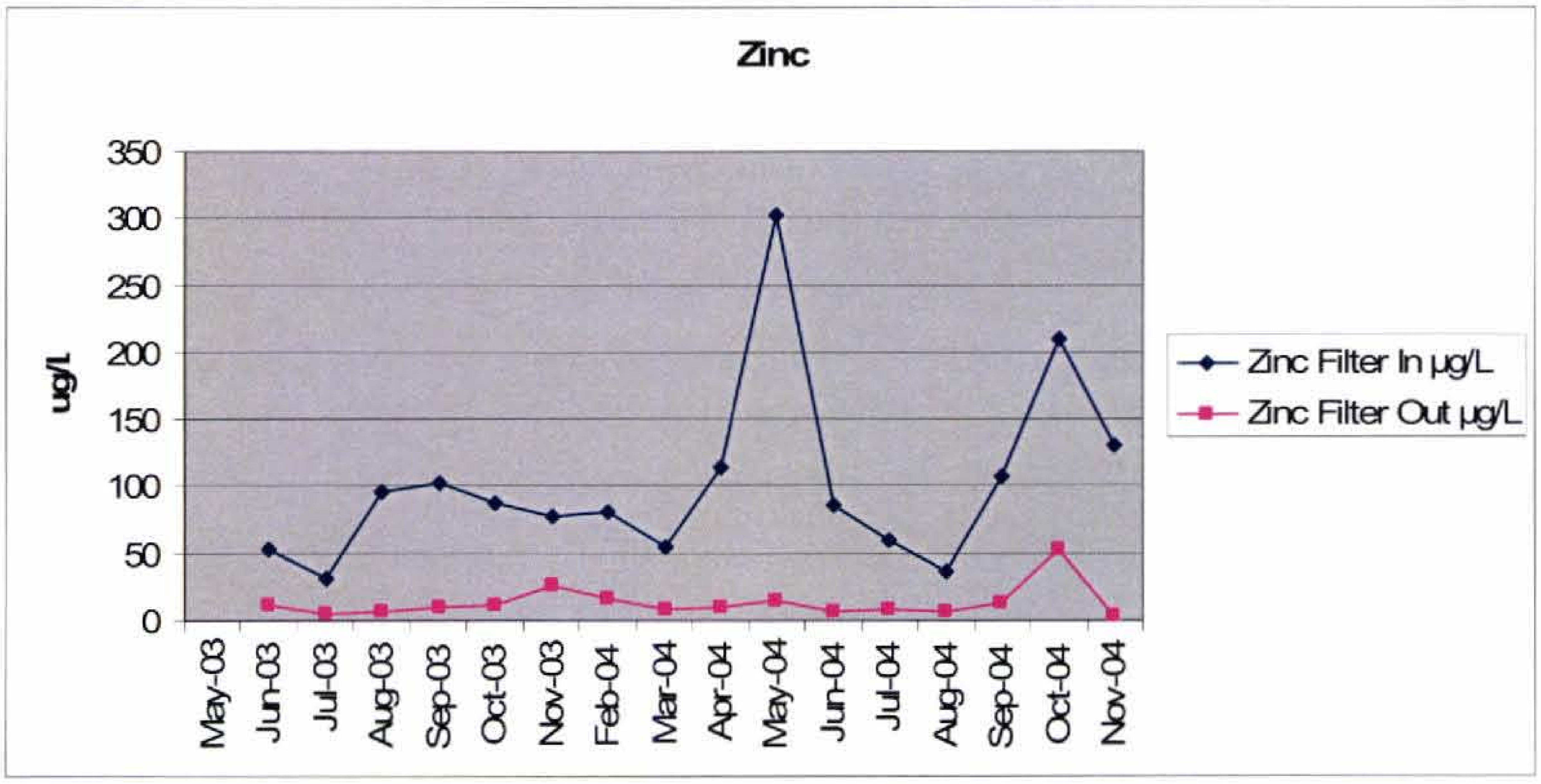


Figure 5.27: Total zinc concentrations before and after sand filter application of storm water.

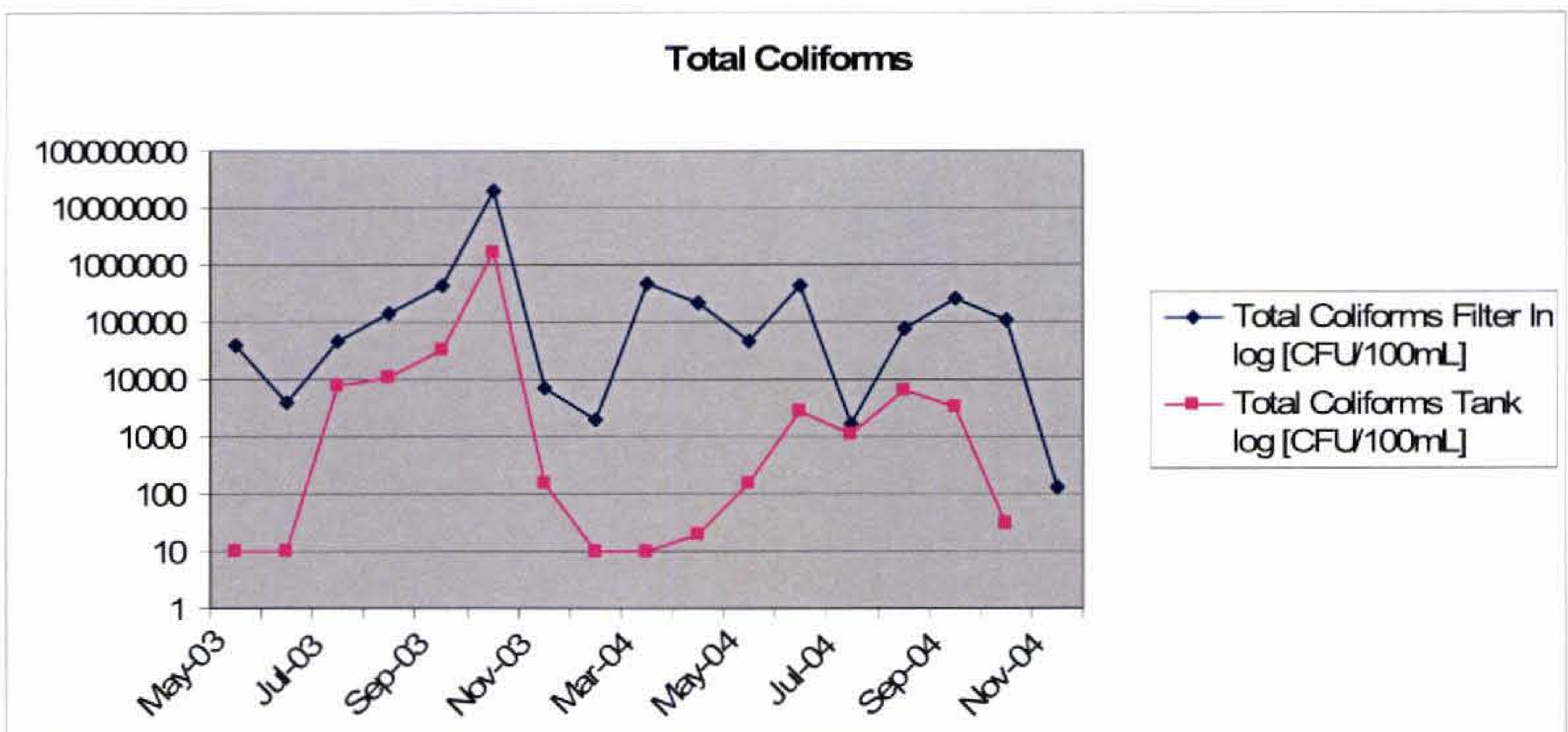


Figure 5.28: Total coliform measurements for sand filter influent and storage tank effluent (log scale).

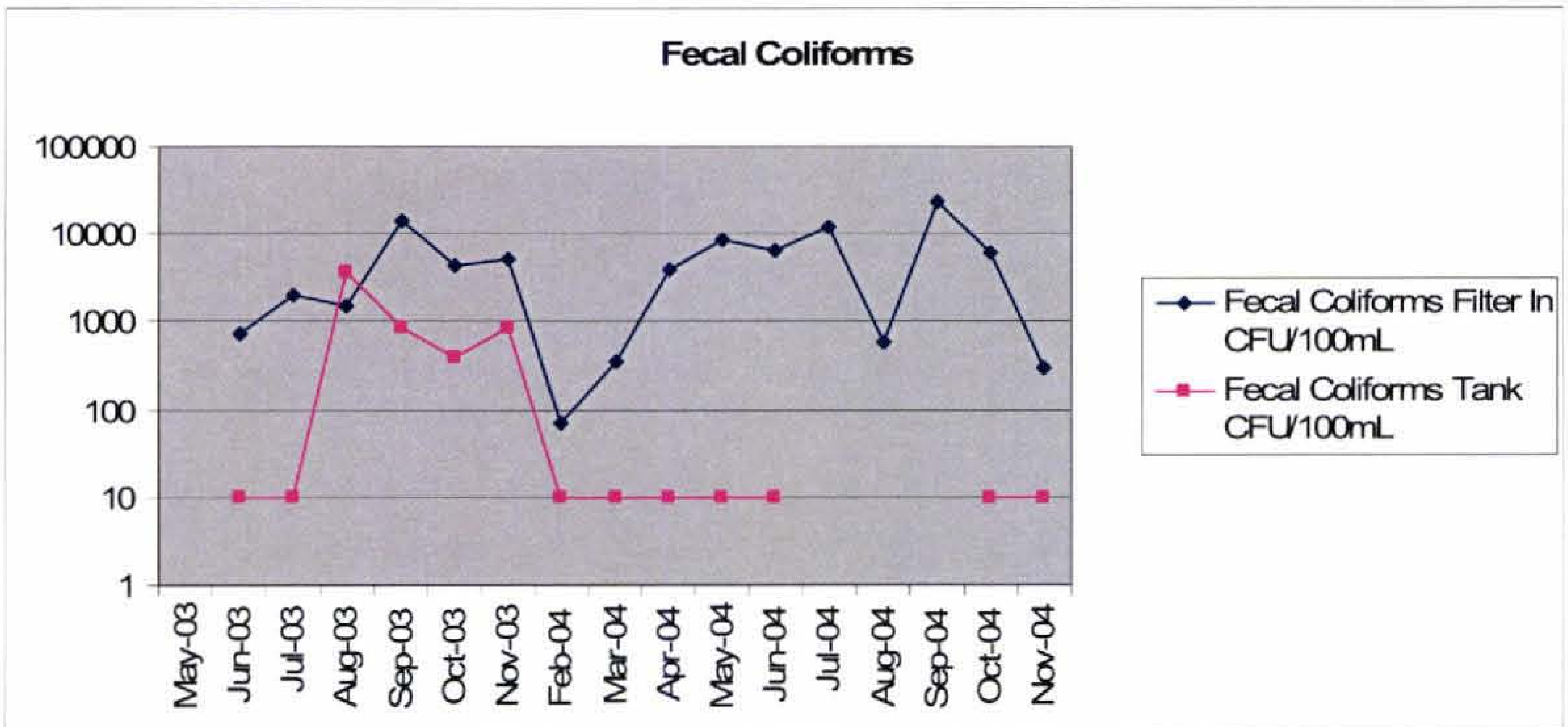


Figure 5.29: Fecal coliform measurements for sand filter influent and storage tank effluent (log scale).

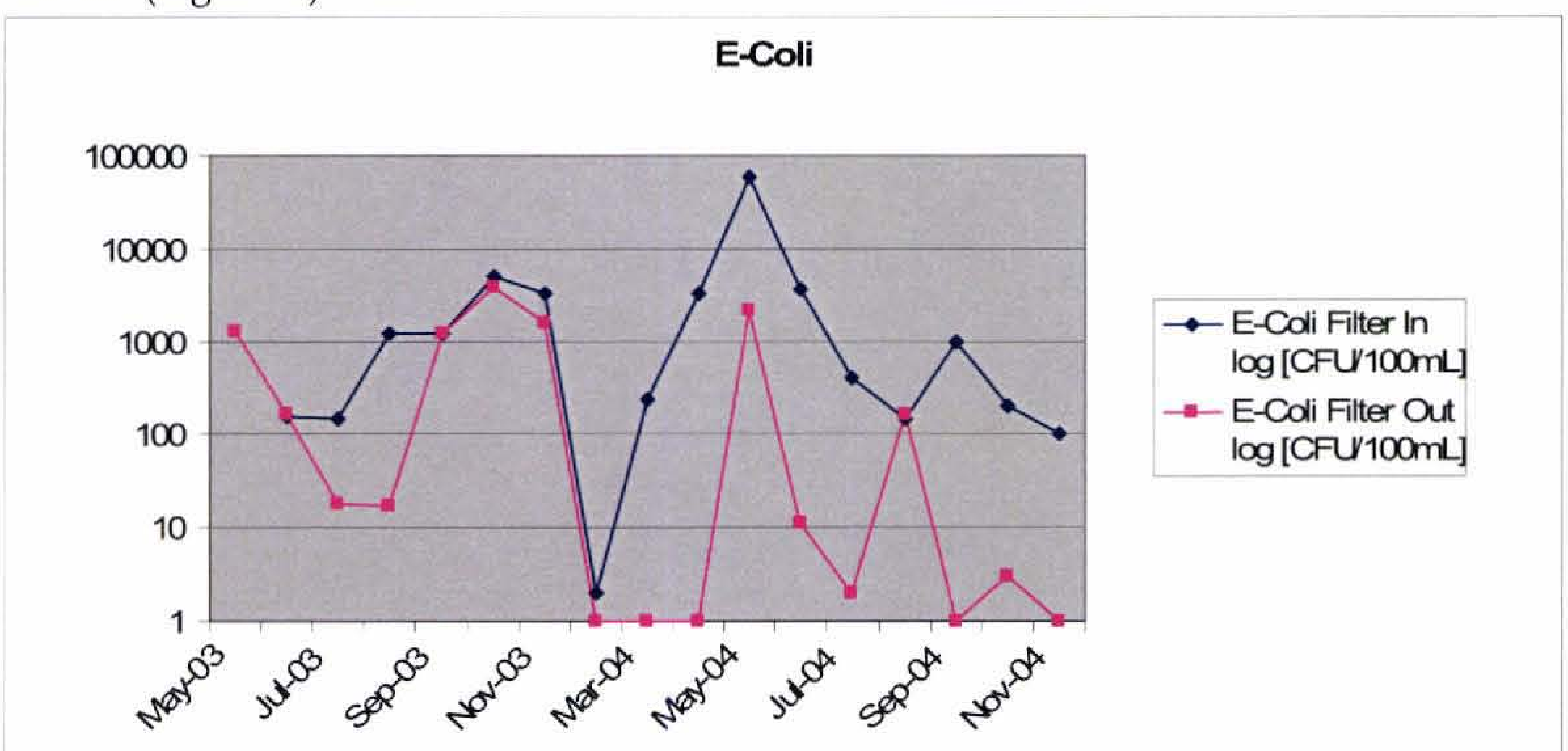


Figure 5.30: E-coli concentrations before and after sand filter application of stormwater (log-scale).

CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

The project was a success. The goals set out by ACHD were all sufficiently met by the design.

1. to provide adequate capacity for anticipated loads
2. provide sufficient room for trucks to decant, wash and fill
3. to create a facility that can be easily maintained, and
4. to treat the decanted water for reuse

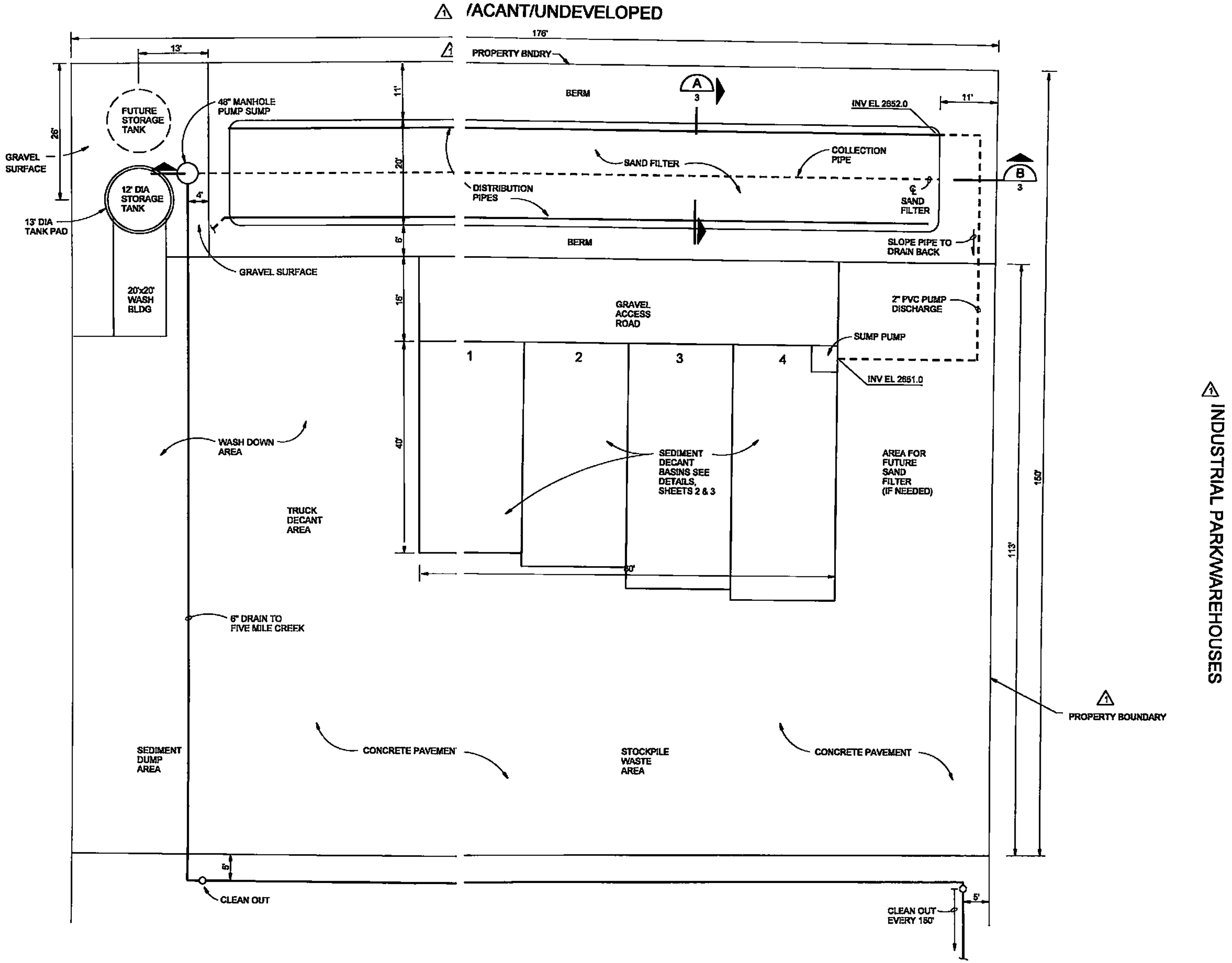
The primary focus of this thesis is goal number 4; the treatment of the decanted water – and even more specifically, the efficiency of the sand filter used for treatment. Overall, the effluent water quality is shown to be sufficiently treated for reuse in road maintenance operations. Solids concentrations are relatively low alleviating concerns over using the water in maintenance equipment; e. coli levels are low; in general all pollutants monitored are acceptably removed in the system. NPDES compliance is achieved through the monitoring program, and no other regulations currently apply to water at this site.

Of the lessons learned through the course of this study; the use of topsoil as the top layer of the sand filter proved not to be of any added value, and actually inhibited performance of the sand filter. Including topsoil would not be recommended for future designs of this type of sand filter. Secondly, a large part of the success of the design is contributable to ample input collected from facility users. The site layout was designed with them in mind, and is flexible – operations (decant, wash, fill, stockpile locations) can be moved as users see fit. And lastly, the large surface area of the sand filter provides a relatively low loading rate, and thus high pollutant removal efficiencies and low maintenance requirements is achieved. The first scheduled cleaning (replacement of top sand layer) set for summer 2006, 3 ½ years after beginning of operations at the Cloverdale site.

APPENDIX A
DESIGN DRAWINGS (CH2MHILL)

NOTES:

1. ALL FLOW FROM CONCRETE PAD IS DIRECTED TO DECANT BASIN #1.
2. SPEED BUMPS ARE PLACED TO CONTROL FLOW.



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DSGN LIZ ADAMS					
DR H.J. SUCHY					
CHK LIZ ADAMS	1	5/02	ADD COMMENTS		
APVD RANDY PETERSON	NO.	DATE	REVISION	BY	APVD

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"
IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

CH2MHILL

BOISE, IDAHO

SITE PLAN

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DATE MAY 2002
PROJ 148140

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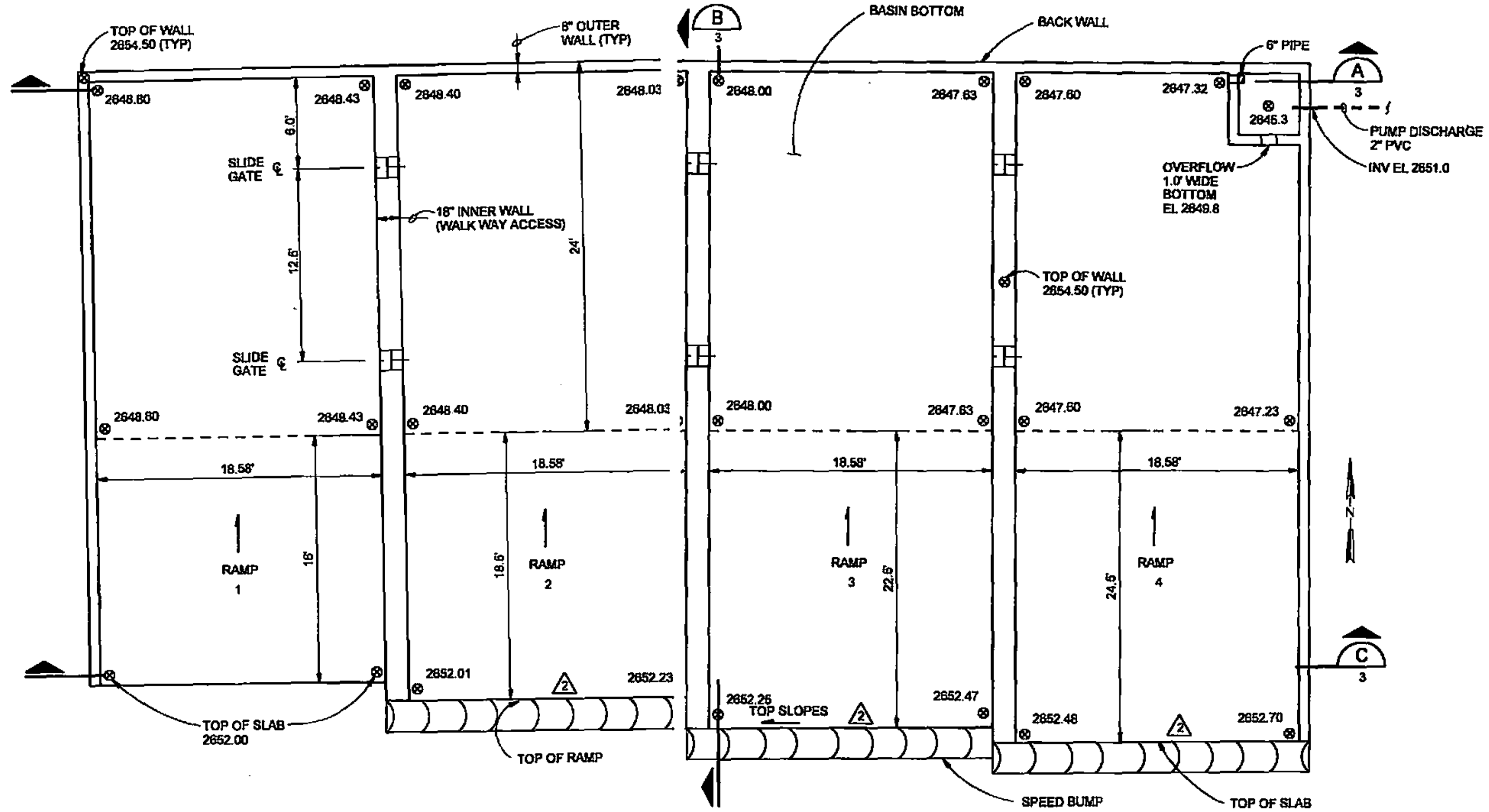

SEDIMENT DE ANT BASIN PLAN

1"=10'

DSGN	LIZ ADAMS			
DR	H.J. SUCHY			
CHK	LIZ ADAMS	2	10/02	REVISED GRADING
APVD	RANDY PETERSON	NO.	DATE	REVISION

BY | APVD

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BASIN PLAN

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PROJ 148140

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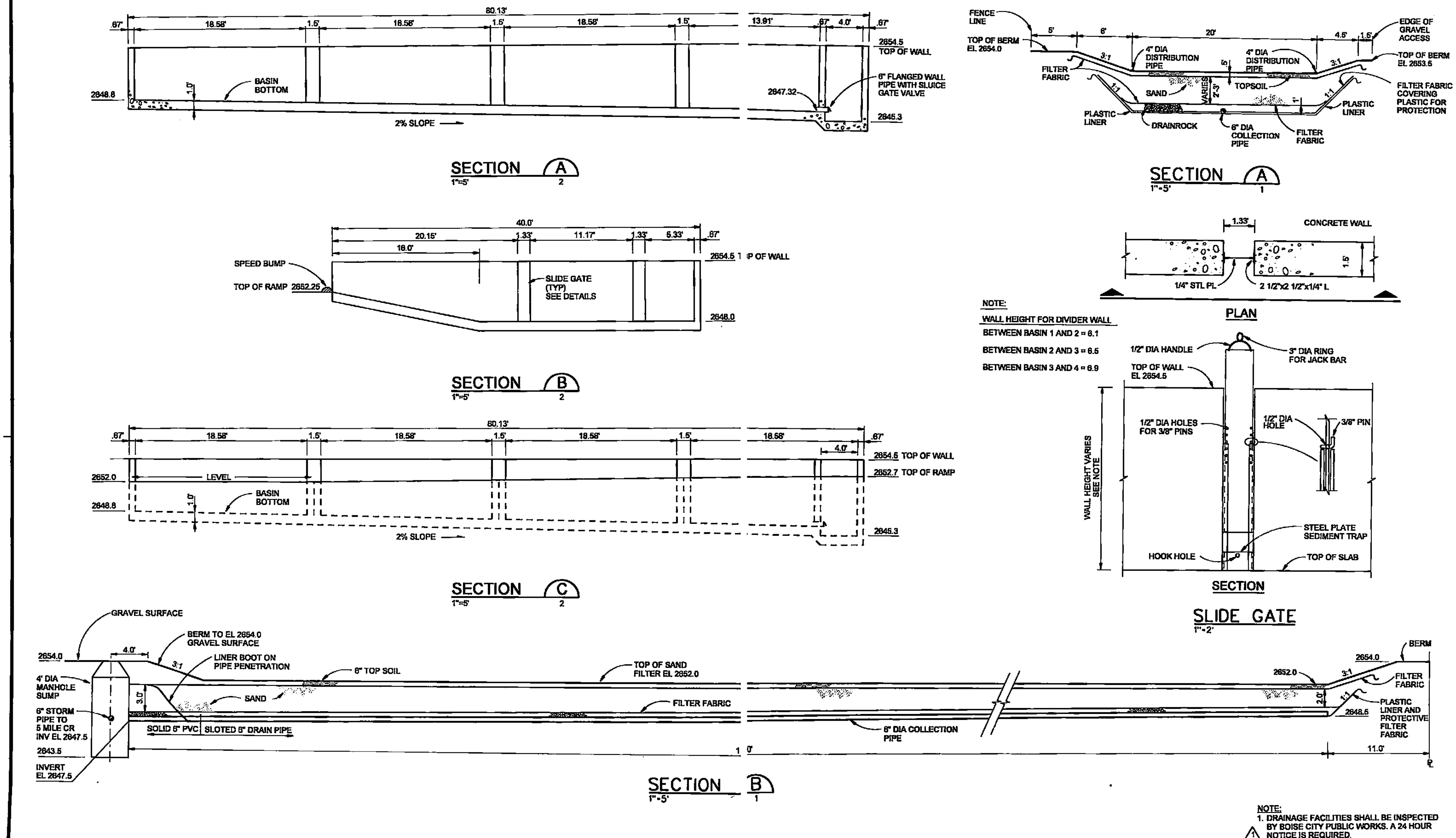

DSGN	LIZ ADAMS			
DR	H.J. SUCHY			
CHK	LIZ ADAMS	1	5/02	ADD COMMENTS
APVD	RANDY PETERSON	NO.	DATE	REVISION

BY | APVD

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BOISE, IDAHO

SECTIONS

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DWG. NO.
DATE MAY 2002
PROJ 148140

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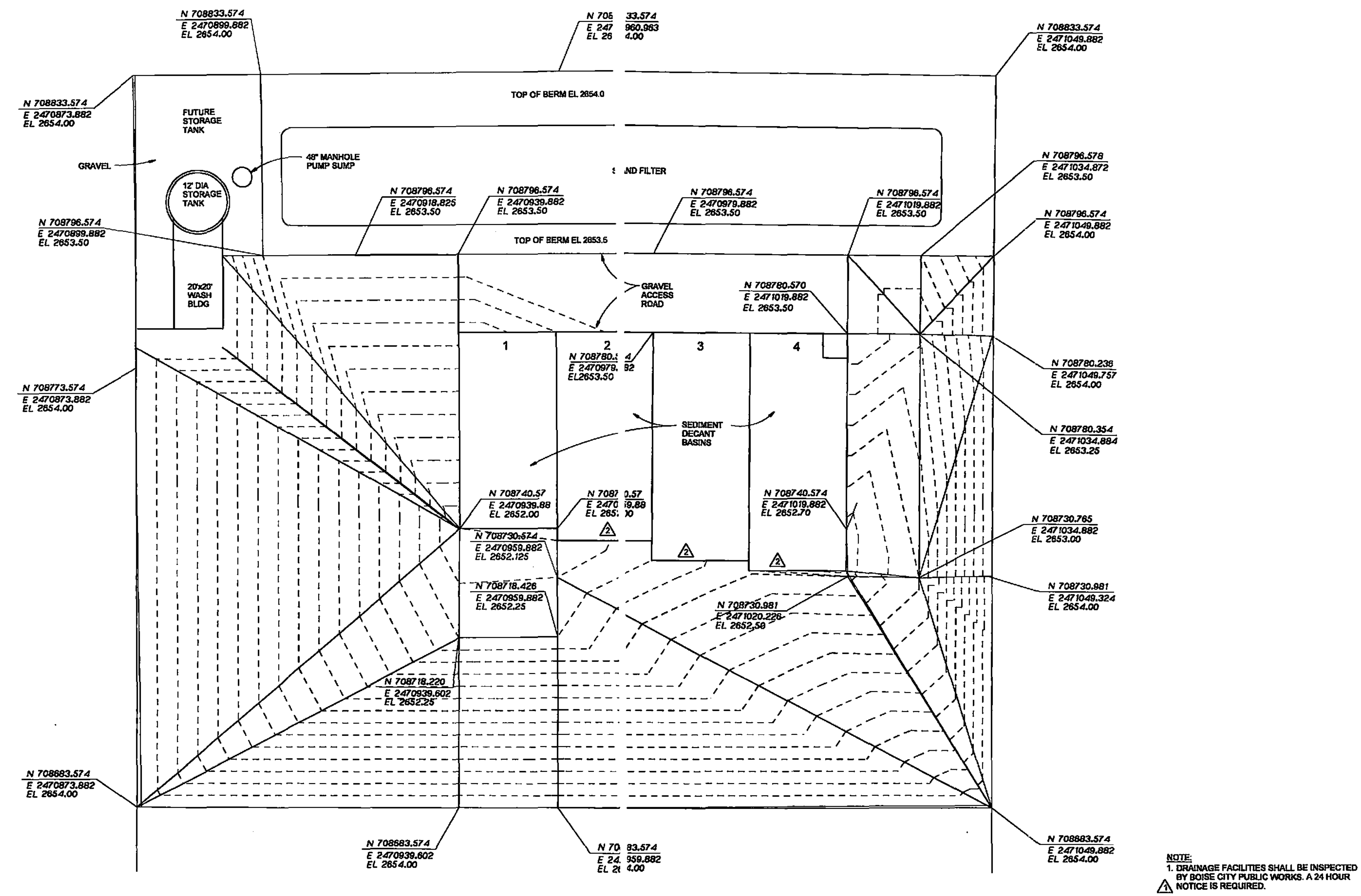

NOTE:
1. DRAINAGE FACILITIES SHALL BE INSPECTED BY BOISE CITY PUBLIC WORKS. A 24 HOUR NOTICE IS REQUIRED.

DSGN	LIZ ADAMS					
DR	H.J. SUCHY	2	10/02	REVISED GRADING		
CHK	LIZ ADAMS	1	5/02	ADD COMMENTS		
APVD	RANDY PETERSON	NO.	DATE	REVISION	BY	APVD

5/02 ADD COMMENTS

VERIFY SCALE
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0 1"
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BOISE, IDAHO

GRADING PLAN

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DATE MAY 2002
PROJ 148140

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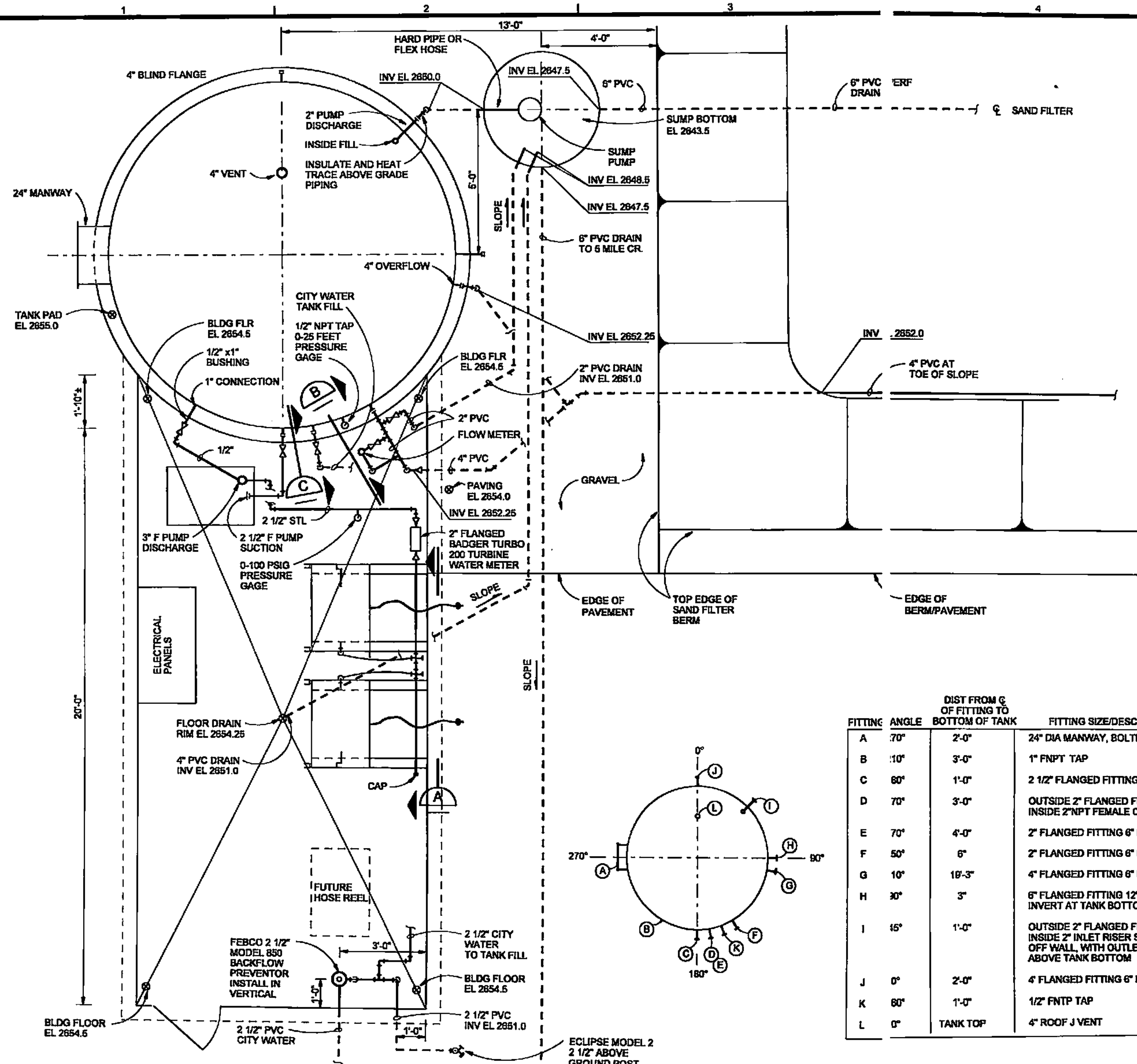
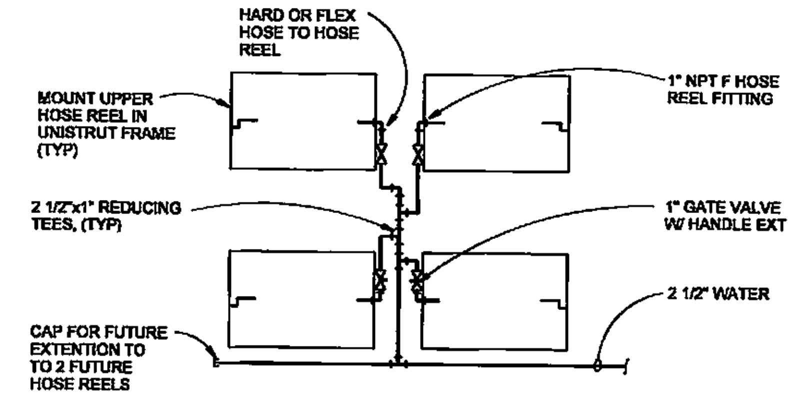


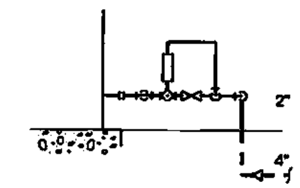
MECHANICAL PLAN

1/2"=1'-0"



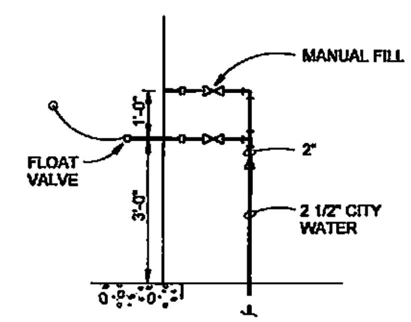
SECTION A

1/2"=1'-0"



SECTION B

1/2"=1'-0"



SECTION C

1/2"=1'-0"

FITTING	ANGLE	DIST FROM ℄ OF FITTING TO BOTTOM OF TANK	FITTING SIZE/DESCRIPTION	FITTING FUNCTION
A	[illegible]70°	2'-0"	24" DIA MANWAY, BOLTED W/GASKET	MANHOLE
B	[illegible]10°	3'-0"	1" FNPT TAP	PUMP DISCHARGE BLEED BACK TO TANK
C	60°	1'-0"	2 1/2" FLANGED FITTING, 6" LONG	PUMP SUCTION
D	70°	3'-0"	OUTSIDE 2" FLANGED FITTING 6" LONG INSIDE 2"NPT FEMALE COUPLING	CITY WATER FILL
E	70°	4'-0"	2" FLANGED FITTING 6" LONG	CITY WATER FILL
F	50°	6"	2" FLANGED FITTING 6" LONG	DRAIN TO SUMP & RECYCLE TO SAND FILTER
G	10°	19'-3"	4" FLANGED FITTING 6" LONG	OVERFLOW TO SAND FILTER
H	[illegible]0°	3"	6" FLANGED FITTING 12" LONG INVERT AT TANK BOTTOM	TANK WASH OUT DRAIN
I	[illegible]5°	1'-0"	OUTSIDE 2" FLANGED FITTING 6" LONG, INSIDE 2" INLET RISER STAND PIPE 6" OFF WALL, WITH OUTLET AT 18'-6" ABOVE TANK BOTTOM	SUMP DISCHARGE/TANK FILL
J	0°	2'-0"	4" FLANGED FITTING 6" LONG	FUTURE TANK CONNECTION
K	60°	1'-0"	1/2" FNTP TAP	PRESSURE GAGE
L	0°	TANK TOP	4" ROOF J VENT	VENT

TANK FITTING LAYOUT

NTS

DSGN LIZ ADAMS							
DR H.J. SUCHY							
CHK LIZ ADAMS							
APVD RANDY PETERSON	NO.	DATE	REVISION	BY	APVD		

VERIFY SCALE
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0 1"
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CH2MHILL

BOISE, IDAHO

PARTIAL PLAN AND SECTIONS

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DATE MAY 2002
PROJ 148140

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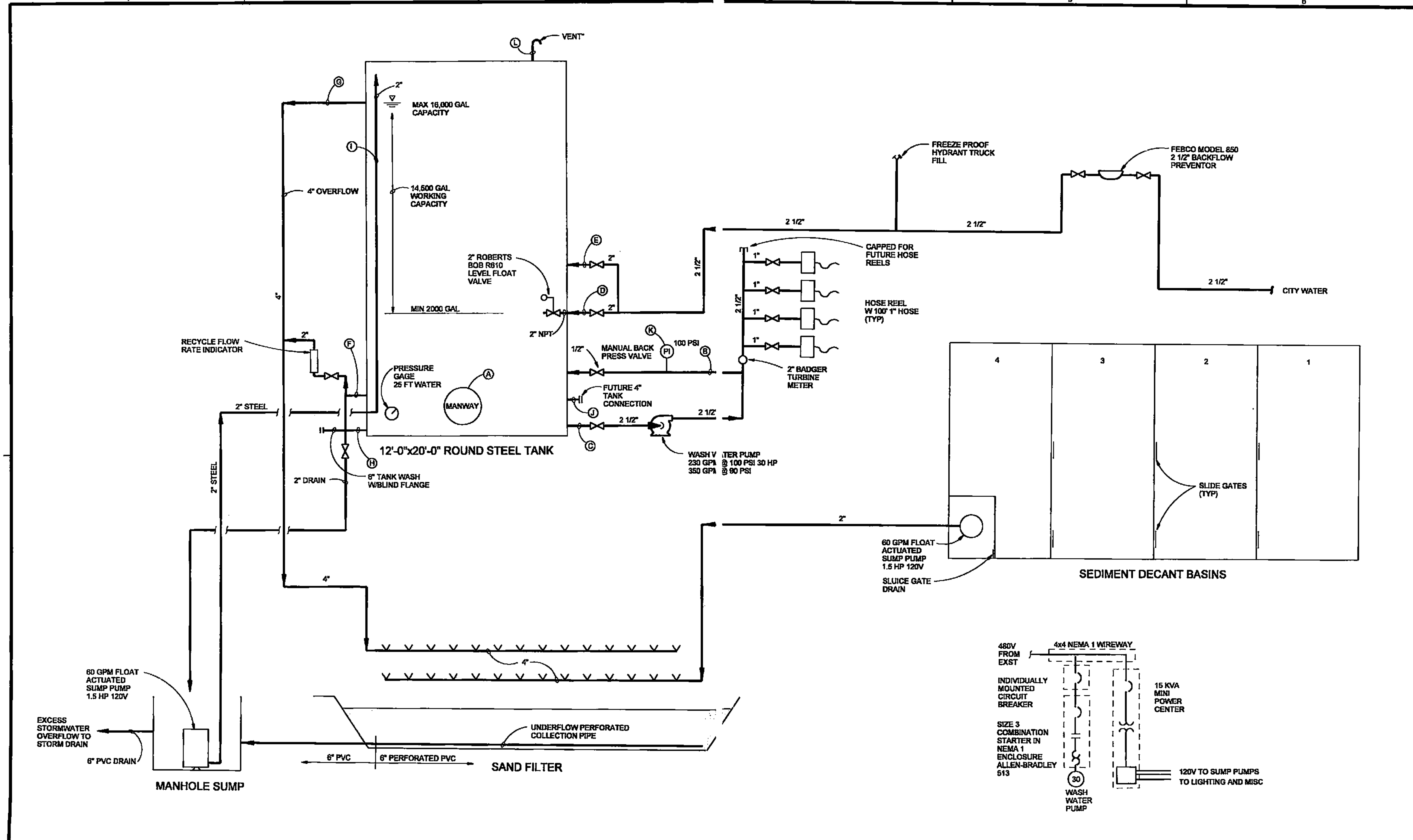

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CHK LIZ ADAMS						
APVD RANDY PETERSON						

CH2MHILL

BOISE, IDAHO

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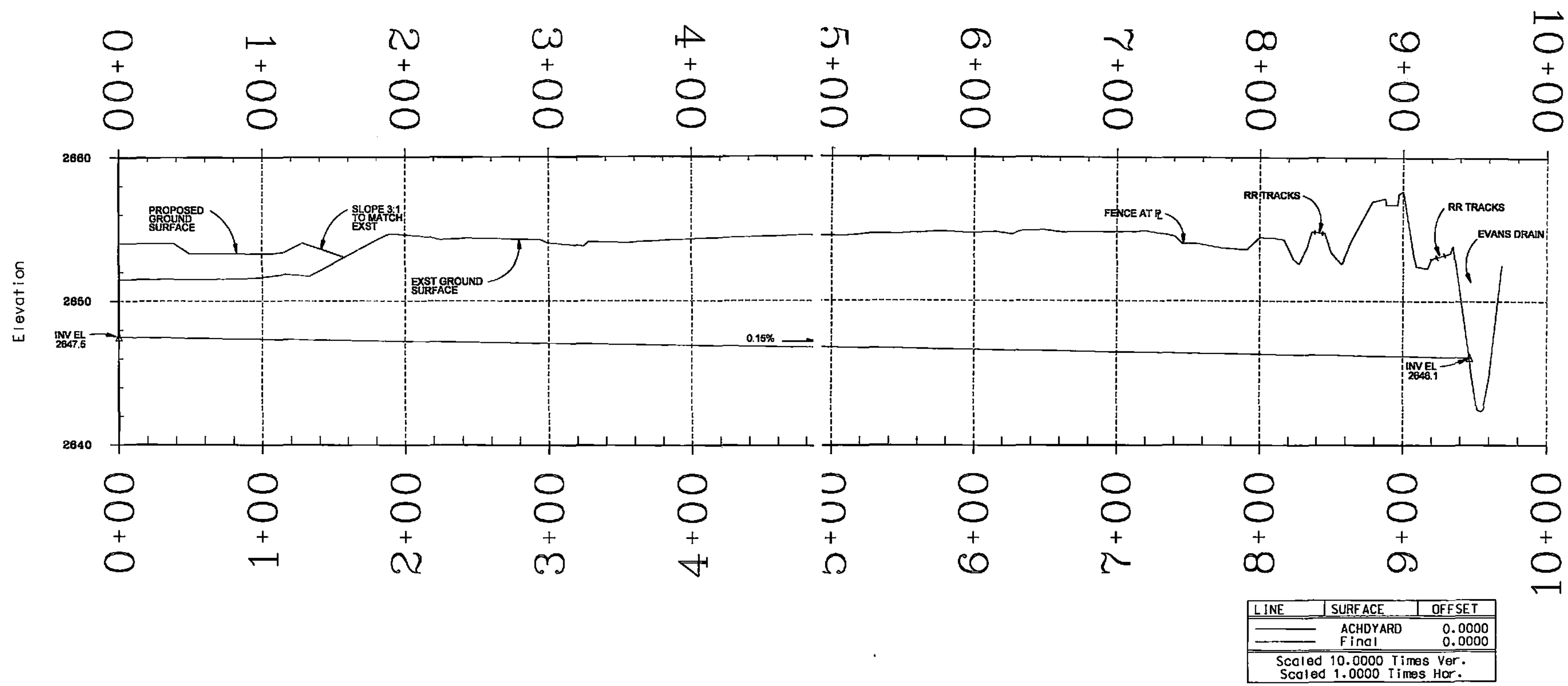

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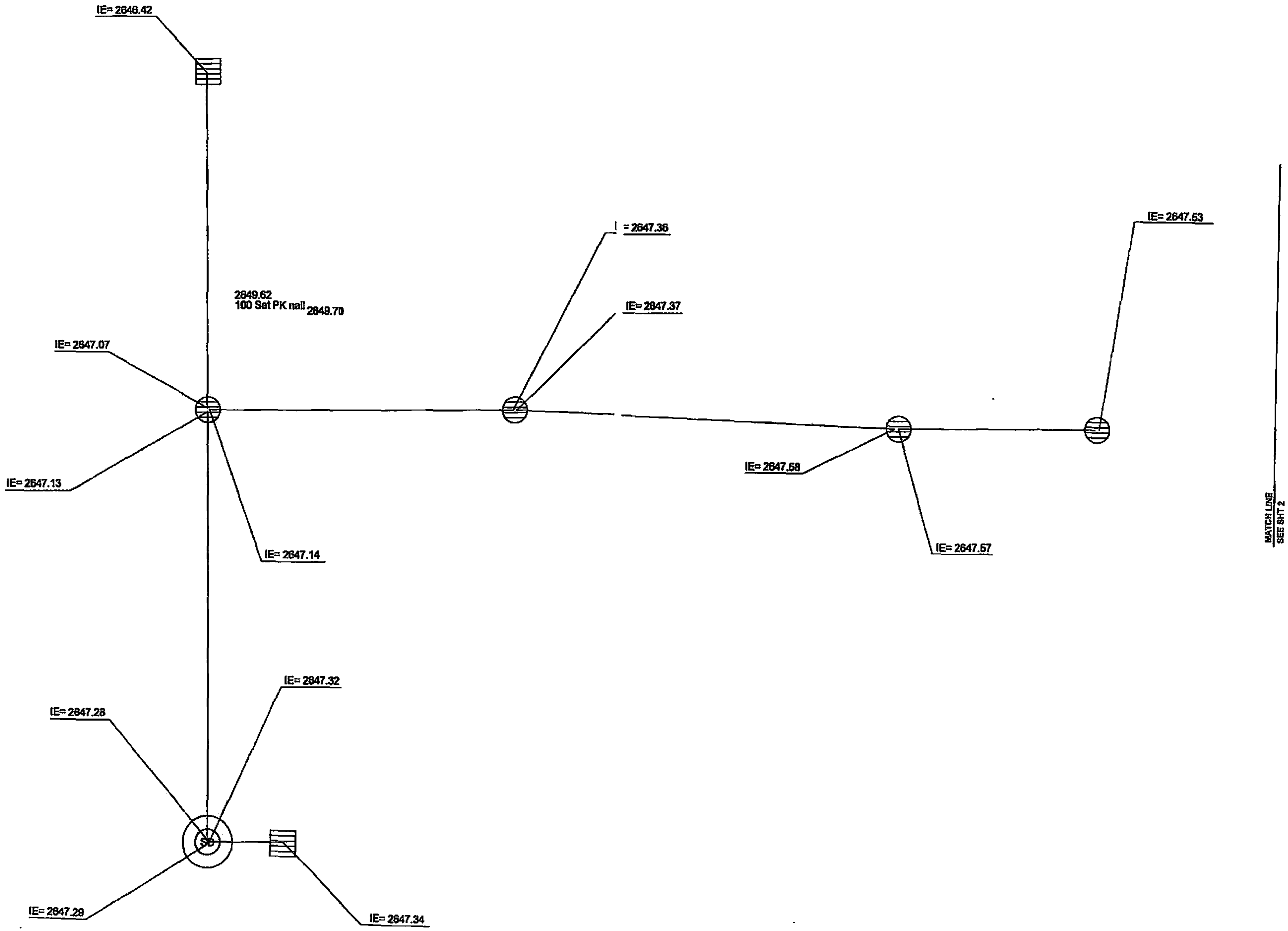

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PRE DEVELOPMENT
SITE ELEVATIONS
EXISTING STORM DRAINS

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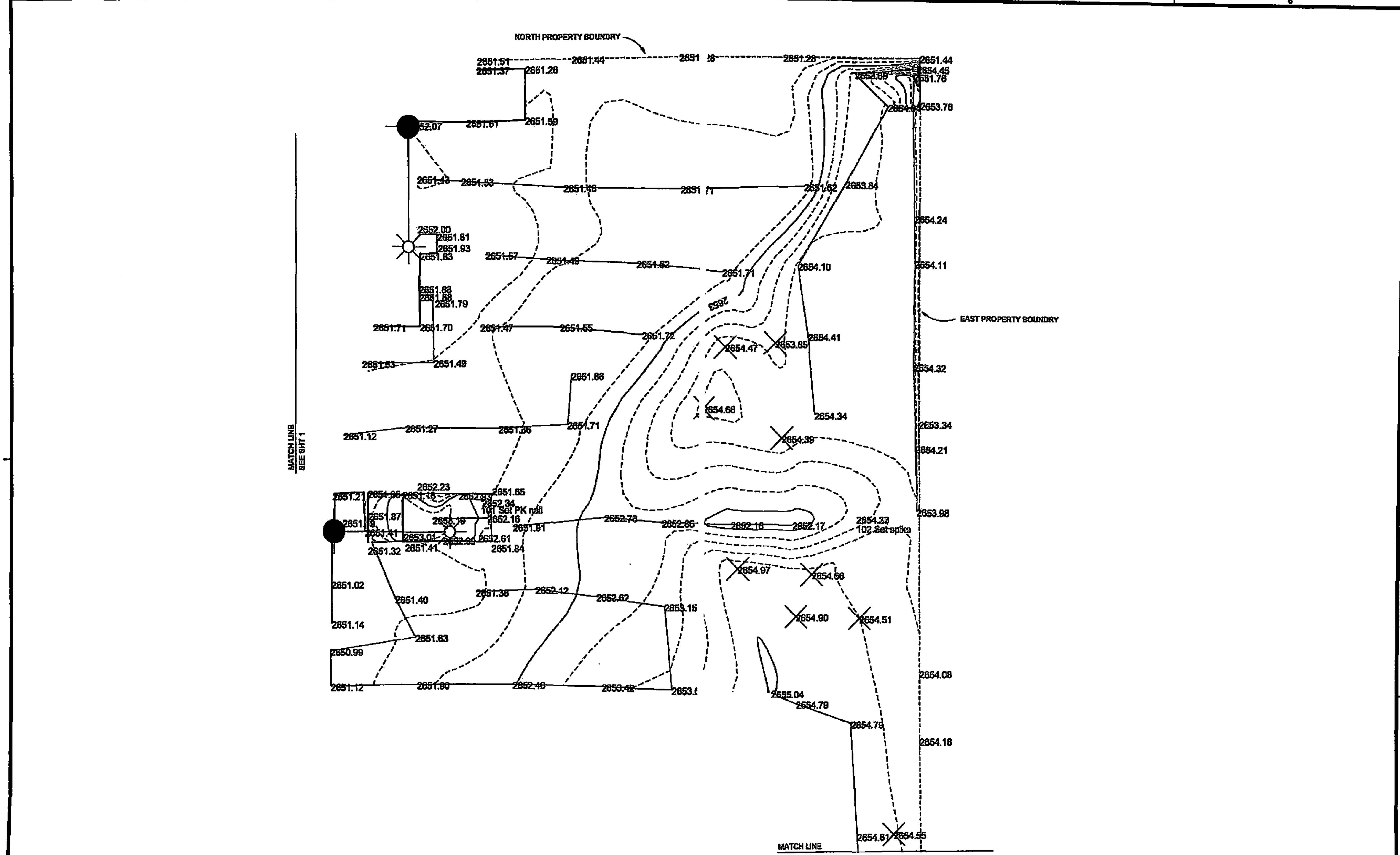

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PRE DEVELOPMENT
SITE ELEVATIONS
STILLING POND/DEWATERING POND

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DATE MAY 2002
PROJ 146140

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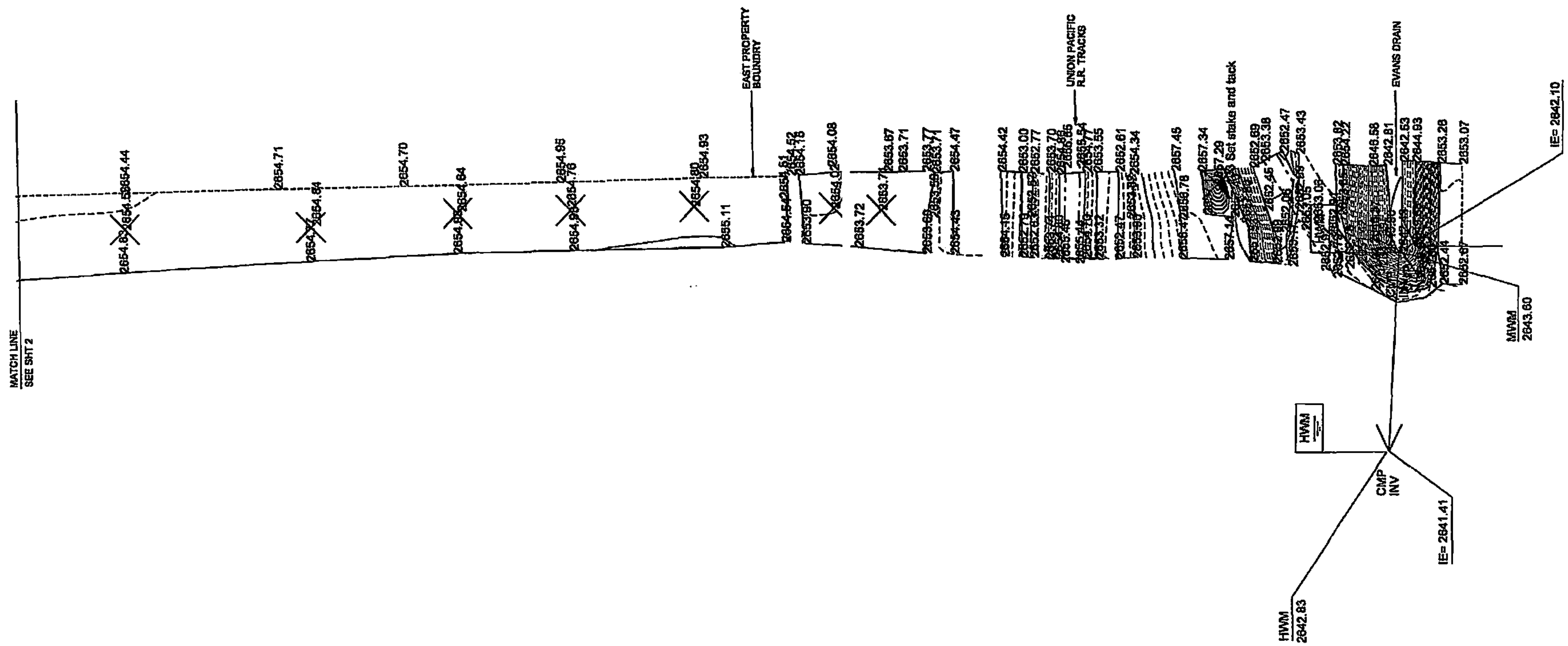

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PRE DEVELOPMENT
SITE ELEVATIONS
EAST PROPERTY BOUNDRY

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DWG. NO.
DATE MAY 2002
PROJ 148140

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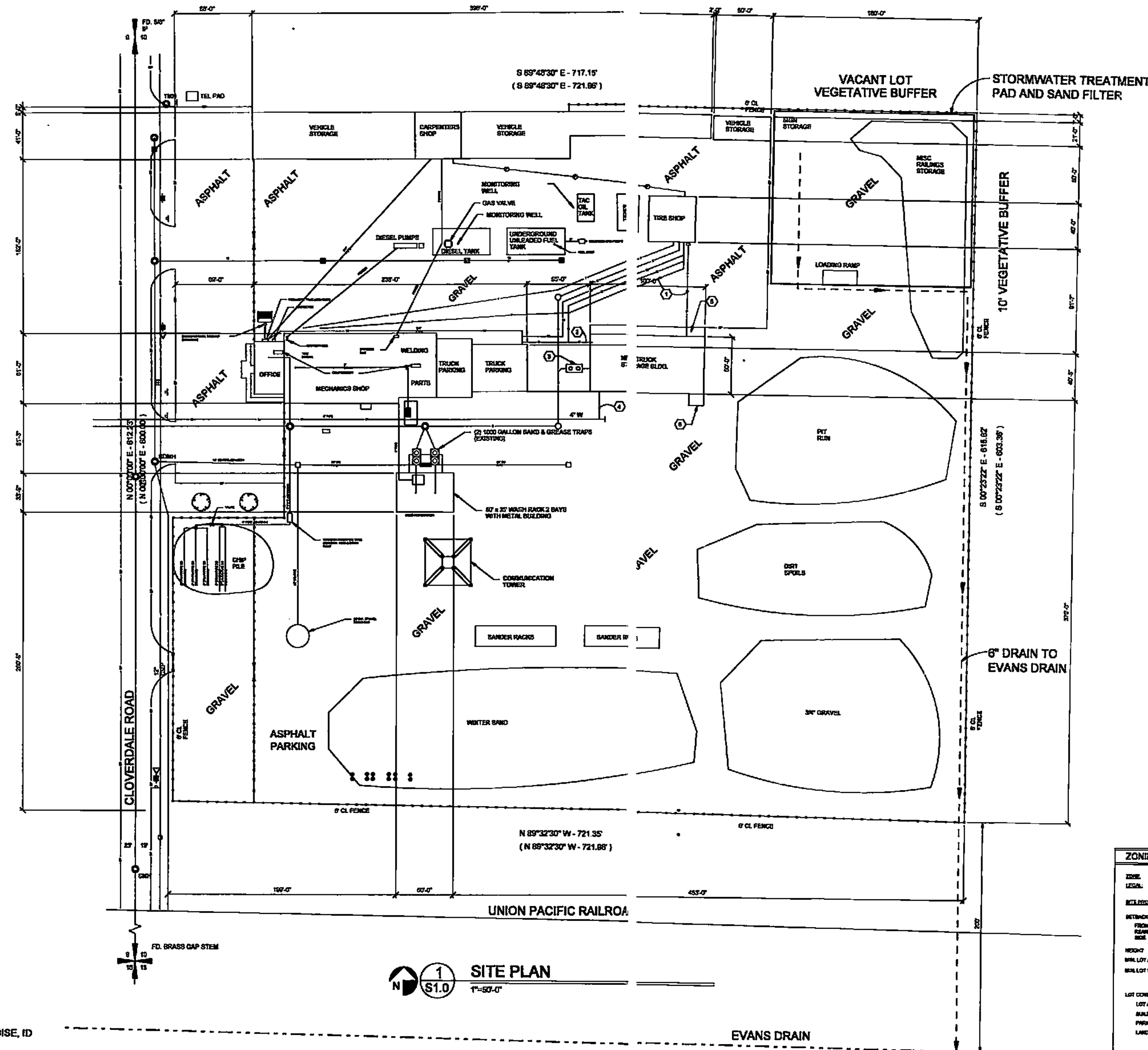
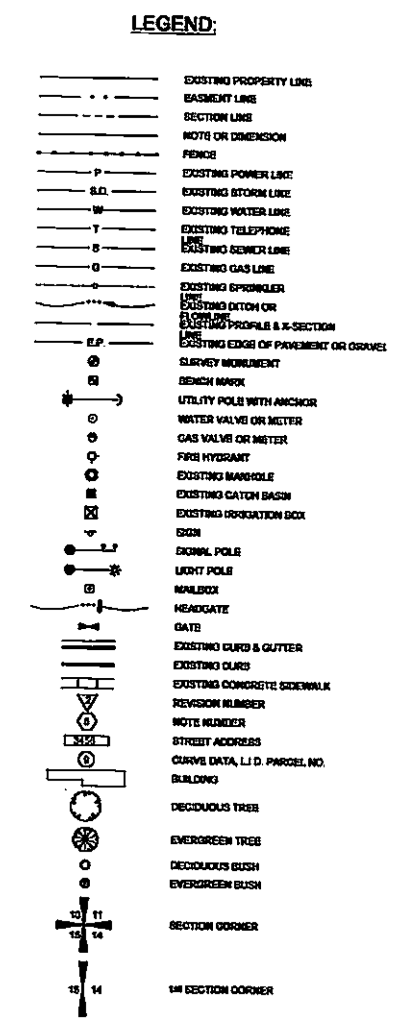


SITE PLAN

1 S1.0

1"=50'-0"



SHOP NOTES:

1. (2) 2" SCH. 40 PVC CONDUIT, U.G. FROM SERVICE ENTRY @ OFFICE TO TIRE SHOP. ROUTE (1) 2" CONDUIT & CONDUCTORS TO NEW TRUCK BLDG.
2. 1-1/4" GAS PIPING.
3. 4" MINIMUM DRAIN LINE. SLOPE MIN. 1/8" / FT. PROVIDE MAN HOLE W/ TRAFFIC LID AT CHANGES IN DIRECTION. PROVIDE GRADE CLEAN OUT @ BLDG. CONNECT TO CLOSEST AVAILABLE STORM DRAIN LINE THROUGH 1000 GAL. OIL/SAND SEPARATOR.
4. 4" WATER LINE - CONNECT TO EXISTING 4" MAIN. VERIFY SIZE. PROVIDE BACKFLOW PREVENTER @ BLDG.
5. CONCRETE APRON & LANDING, SEE FOUNDATION / SLAB PLAN

ZONING RECAP

ZONE: M-1D

SOURCE: ACHD TRUCK STORAGE, CLOVERDALE, BOISE, IDAHO, PATRICK McKEEGAN ARCHITECTS, BOISE, ID (208)424-8608, NOVEMBER 6, 2001.

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BOISE, IDAHO

ACHD TRUCK STORAGE SITE PLAN

SHEET	11
DWG	
DATE	MAY 2002
PROJ	148140

FILENAME: achd site plan.dgn PLOT DATE: 28-MAY-2002

APPENDIX B
CONSTRUCTION PHOTOGRAPHS

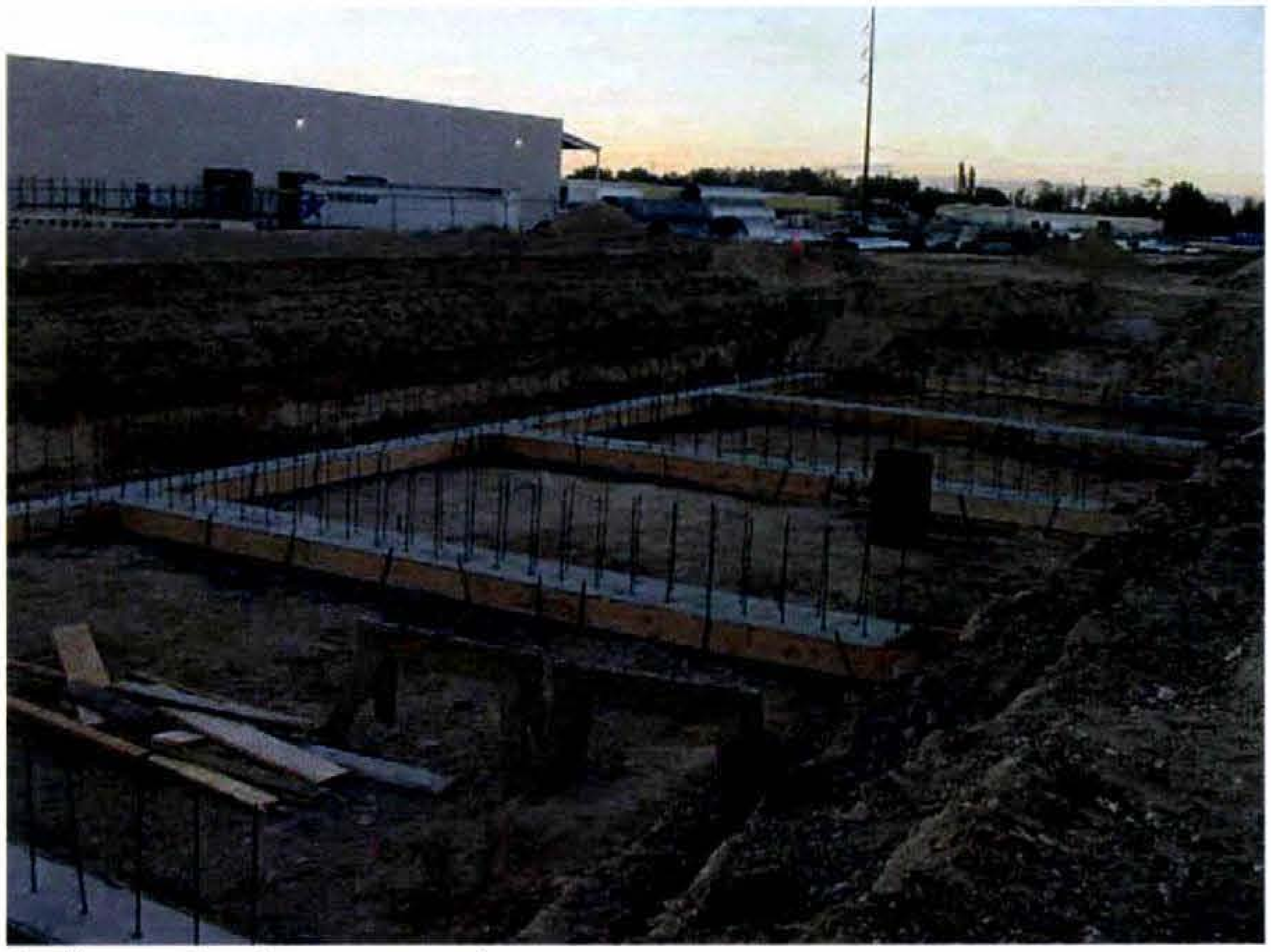
B1 Preconstruction

Cloverdale maintenance yard (photo taken facing east)

Cloverdale maintenance yard (photo taken northwest)

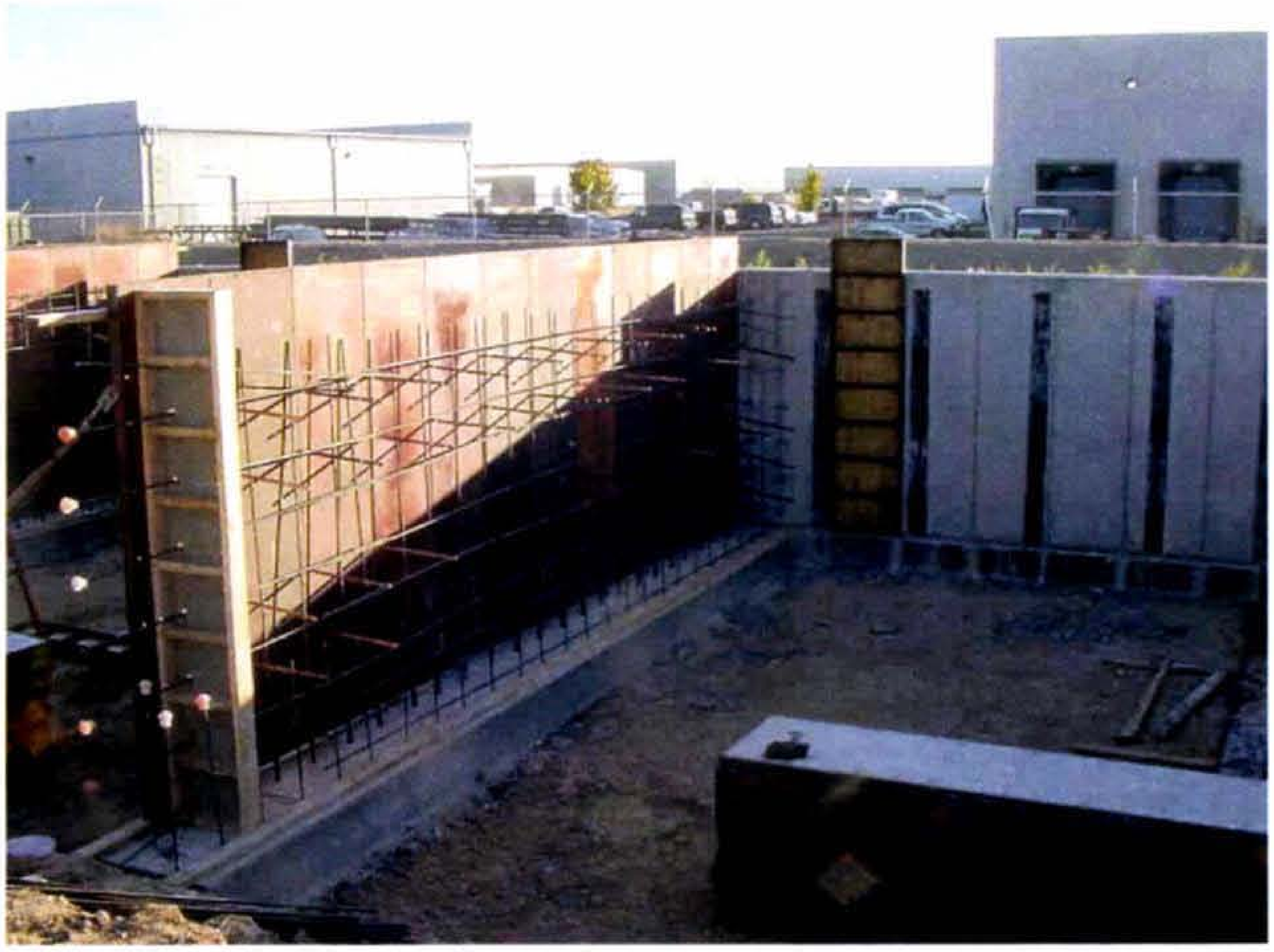
Evans Drain

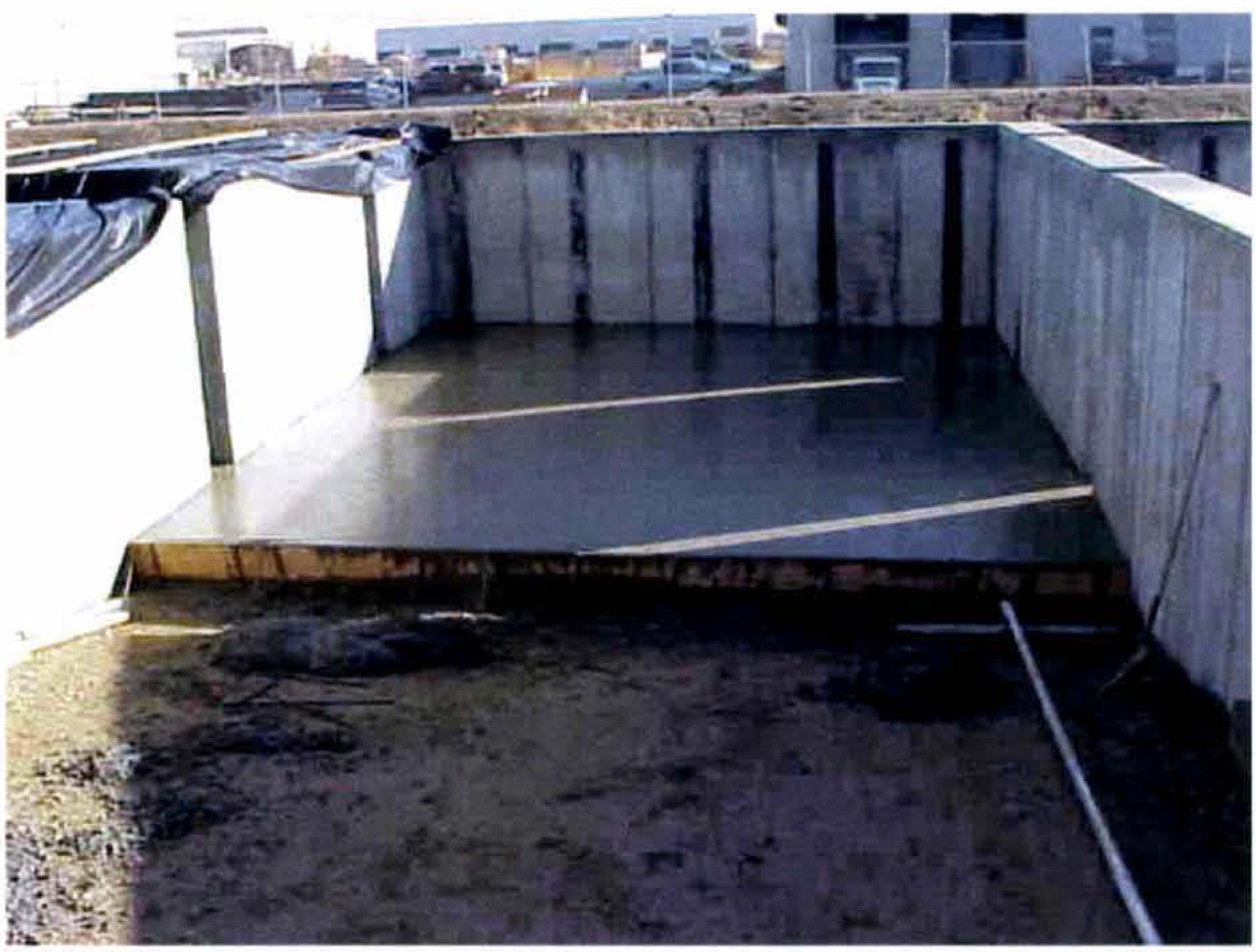
B2 Construction of Sediment Basin and Concrete Pad



Sediment basin construction

Sediment basin construction


Sediment basin construction


Sediment basin construction


Completed sediment basin


Sediment basin in operation


Sediment basin and concrete pad in operation

Sediment basin in operation

B3 Sand Filter Construction

Sand filter construction

Sand filter construction

Sand filter construction

Completed sand filter


Sand filter in operation

B4 Storm Drain Construction

Outfall construction under Union Pacific Railroad tracks


Outfall construction under Union Pacific Railroad tracks

Outfall construction under Union Pacific Railroad tracks

Outfall construction under Union Pacific Railroad tracks

Trenching for storm drain at Cloverdale maintenance yard.

Trenching for storm drain at Cloverdale maintenance yard

APPENDIX C
SAND FILTER SIZING AND DESIGN GUIDE

Stormwater Sand Filter Sizing and Design
A Unit Operations Approach

Ben R. Urbonas, P.E.
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Urban Drainage and Flood Control District,
2480 W. 26th Avenue, Suite 156-B; Denver, Colorado 80211

ABSTRACT

The use of sand and other media filters are gaining acceptance in the field of urban stormwater structural best management practice. Much work has been done to develop local design guidance such as in the State of Delaware and in Austin, Texas. Also, considerable field testing of these devices has occurred over the last 10 years. This paper consolidates much of the earlier work and provides the technical basis for the design of media filters for stormwater runoff treatment at any location in the United States. The approach utilizes the unit processes known to exist in urban stormwater runoff and within filter devices. The suggested design is based on hydraulic capacity of the filter media, which, in turn, is a function of the total suspended solids removed by the filter.

INTRODUCTION

Sand and other media filters remove constituents from stormwater runoff primarily through a physical process of filtering out particulates from the water. The type of media used and its grain size distribution determine how small of a particle is strained out. Coarser sands have larger pore spaces that have high flow-through rates but pass larger suspended particles. A very fine sand, or other fine media filter, has small pore spaces with slow flow-through rates and filter out smaller total suspended solids (*TSS*) particles. Some media, such as peat-sand mix, may also provide ionic adhesion or exchange for some dissolved constituents which further enhances effluent quality.

Laboratory and field tests have shown (Neufeld, 1996; EPA, 1983; Veenhuis, 1989; City of Austin, 1990) that a filter media consisting of concrete sand (ASTM C-33 mix) provides a good balance between flow-through rates and filtering efficiency. The filter performs like a classic slow sand filter that has been used to treat water for approximately 100 years. Initially the flow-through rates are high, but as the filtrate of fine sediment accumulates on its surface, flow-through rates diminish. In water treatment the quality of the effluent improves as the filtrate layer thickens. This may not be the case with stormwater. Some field tests suggest that the effluent quality improves initially, but may degrade over time, suggesting leaching out of constituents from the filtrate and a need for maintenance.

In water treatment plants, scarifying the "sealed" surface improves the filter's flow-through rates. Eventually the filter media is removed and replaced. Water treatment filters operate continuously and regular maintenance is a part of the water supply product that is sold to the consumers. However, slow sand filters are rarely used today because they are operationally inefficient and require very large land areas. Instead, multi-media rapid sand filters are the norm in this industry, but they require intense operation and frequent backwashing to keep in operation at design flow-through rates.

Stormwater filters located within a municipality have to operate occasionally, often infrequently. If they are used extensively, there will be a large number of such facilities in any given metropolitan area. As a result, simple economics and pragmatism precludes the use of rapid sand filters for urban stormwater treatment because of their intense operations and maintenance needs. Since there is likely to be a very large number of small filter sites throughout the municipality their operation and maintenance needs become overwhelming. What remains as an option is the use of slow sand filters which require only an occasional cleaning.

The challenge a designer of a stormwater filter faces is to find a design that will provide a sufficient flow-through rate to process most of the runoff events (Urbonas *et al.*, 1996a). The filter has to be made as small as possible for cost reasons, while large enough to pass through the design event(s) without backing up water onto streets, parking lots, etc. and creating nuisance or safety problems for a municipality or its private owners.

DESIGN HYDROLOGY AND *TSS* LOAD

Because of the stochastic nature and temporal variability of stormwater runoff, any stormwater media filter will need a detention storage volume upstream of it. This detention volume permits the capture of rapid runoff so as to buffer the flows that have to be processed through the filter. A filter without such a buffer would have to be very large to keep up with the instantaneous runoff rates during rainstorms. The amount of this detention volume is determined by local runoff characteristics. To deal with the stochastic nature of the runoff process, typically a *design storm* is selected. Also, the rate at which the runoff from this *design storm* is allowed to drain through the filter determines its size. This detention capture volume needs to be emptied out in a reasonable amount of time to provide volume for the next storm runoff event that may follow.

After an extensive literature search of practices in the United States in the 1980's, Urbonas and Ruzzo (1986) suggested that a capture volume upstream of a sand filter be equal to ½ watershed inch of runoff from the impervious surfaces in the tributary watershed. Subsequent studies of rainfall records in the United States and field performance of BMPs now suggest that, as a minimum, this storage volume be between the runoff from an average runoff producing storm depth (i.e., *mean* storm) shown in Figure 1 (Driscoll, et al., 1989) and the *maximized* volume (Guo and Urbonas 1996; Urbonas, *et al.*, 1996a). The *mean* and the *maximized* volumes are a function of how rapidly this volume is fully drained (i.e., evacuated) from the detention basin, or from the surcharge of a retention pond. If it takes a long time, say 48 hours to fully drain this volume, then the probability increases for another storm to occur before this volume is evacuated and a larger detention volume needs to be provided than would be needed if the design *drain time* for this capture volume is less, say 12 hours.

Guo and Urbonas suggested Equation 1 (Guo and Urbonas, 1996; Urbonas, *et al.*, 1996) that permits an engineer to make a first order estimate of the *maximized* volume P_o. This relationship and the values for coefficient *a* (see Figure 2) resulted from extensive runoff modeling performed by Guo using rainfall records from different regions of the United States. The author re-examined these rainfall records and has also developed values of coefficient *a* for the capture of the *mean* storm runoff volumes for use with Equation 1(see Figure 2).

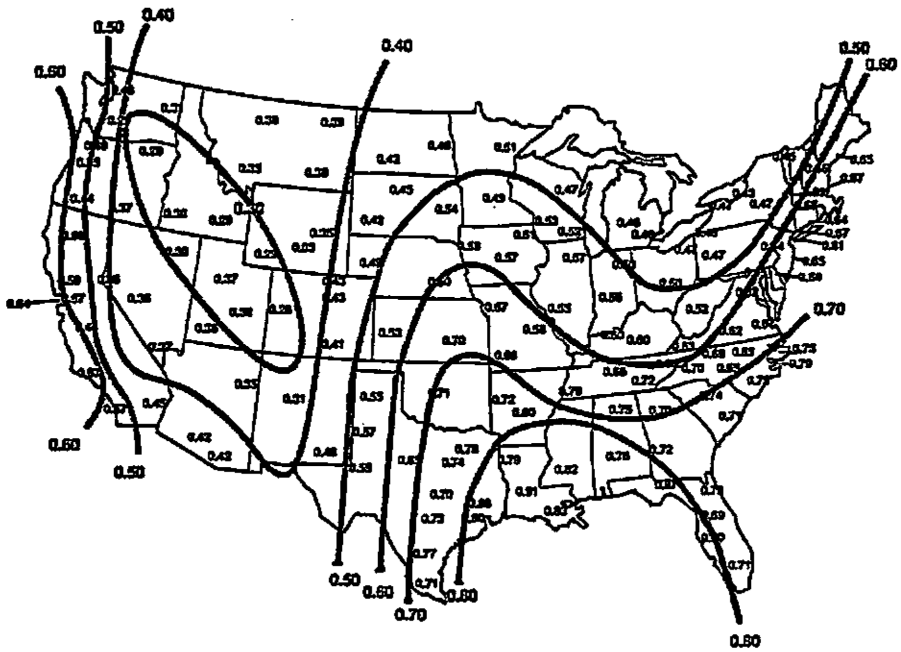
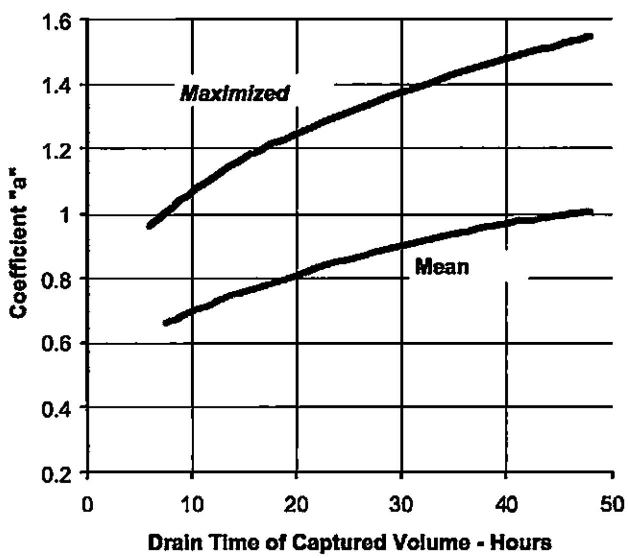


Figure 1. Mean Storm Depths in Inches of Precipitation in United States.
(Ref.: Driscoll, *et. al.*, 1989)



Drain Time in Hours	Determ. Coeff. r^2
12	0.97
24	0.91
48	0.85

Figure 2. Coefficient "*a*" to use in Equation 1.

$$P_o = a \cdot C \cdot P_6 \quad (1)$$

In which, a = coefficient taken for the *maximized* or *mean* runoff volume from Figure 2
C = catchment's runoff coefficient (see Equation 2)
P_6 = average runoff producing storm depth from Figure 1, in inches

P_0 = water quality capture volume (*maximized* or *mean* as appropriate), in inches

The catchment's runoff coefficient can be estimated using Equation 2 which was developed using rainfall and runoff data from 60 NURP sites across the United States (EPA, 1983).

$$C = 0.858 i_a^{\,3} - 0.78 i_a^{\,2} + 0.774 i_a + 0.04 \qquad (r^2 = 0.72) \qquad (2)$$

In which, i_a = I_a/100; fraction of the catchment's total area covered by impervious surfaces

I_a = percent of the catchment's area that is covered by impervious surfaces (use the <u>total</u> percent imperviousness rather than the hydraulically connected portion).

Because the filter's surface accumulates the strained-out materials over time, it is also necessary to know how much runoff can occur over an extended period of time, such as during an average year. This permits an estimate of the average annual load of the constituents in stormwater arriving at the filter and, knowing the filter's removal characteristics, the amount of the constituents removed by the filter during an average year. The annual runoff depth can be estimated using Equation 3.

$$P_A = n \cdot P_6 \cdot C \qquad (3)$$

In which, P_A = average annual total stormwater runoff from the catchment, in inches

n = average number of runoff producing storms per year from Figure 3

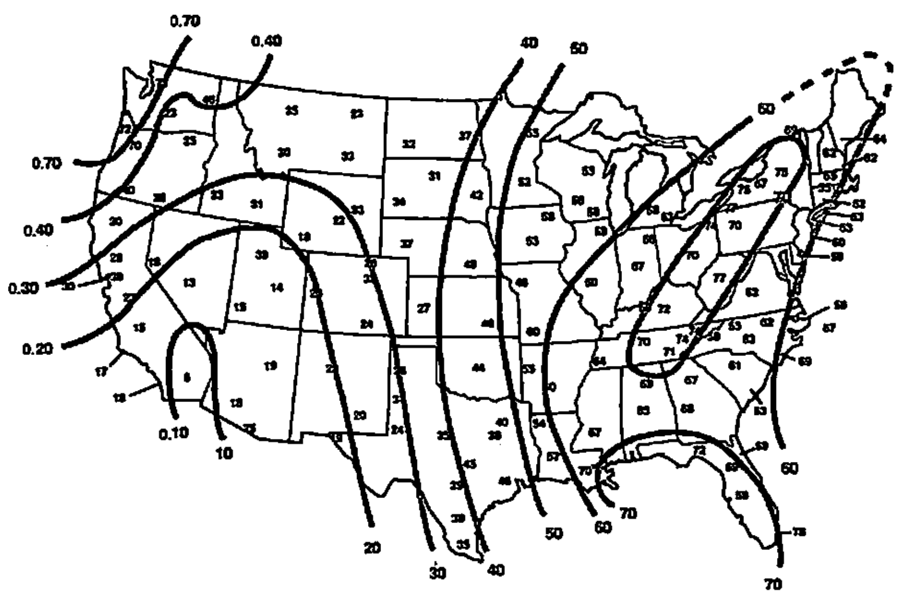


Figure 3. Number of Runoff Producing Storms in United States. (Ref.: Driscoll, *et. al.*, 1989)

Then the average annual load of *TSS* delivered by stormwater to the filter can be found using

$$L_a = \left[\left(A_c \cdot 43{,}560 \right) \cdot \left(\frac{P_A}{12} \right) \right] \cdot \left(\frac{E_s}{10^6} \cdot 62.4 \right)$$

Which can be reduced to:

$$L_a = 0.2265 \cdot A_c \cdot P_A \cdot E_s \qquad (4)$$

In which, L_a = average annual *TSS* load in stormwater runoff from the tributary catchment, in pounds
A_c = area of tributary catchment, in acres
E_s = average *EMC* of *TSS* at the site, in mg/l

This annual load of *TSS*, along with the removal rates by the upstream detention/retention and by the filter, plays a dominant role in determining the size needed for a media filter. In order to proceed further with the design it is necessary to first understand how different detention/retention basin and filter combinations interact in the removal of *TSS* from the water column. Also, it will be necessary to estimate the fraction of the annual *TSS* load, L_a, that will be processed through the filter facility and the fraction that will bypass it.

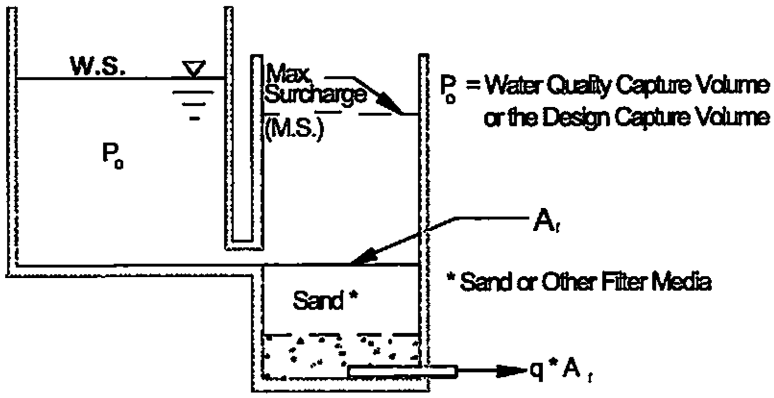
FILTER CONFIGURATIONS

There are three basic arrangements of upstream design volume storage (i.e., water quality capture volume - *WQCV*), and the filter media. Figure 4 schematically illustrates these configurations. The upstream detention captures and equalizes stormwater runoff rates to those compatible with the filter's flow-through capacity. This design volume temporarily stores the higher rates of runoff and permits stormwater to flow through a filter at rates that it is capable of handling, namely its available flow-through rate. When this design capture volume is exceeded by a larger runoff event, the excess volume ponds on the surface of the catchment immediately upstream of the filter, or it bypasses the filter.

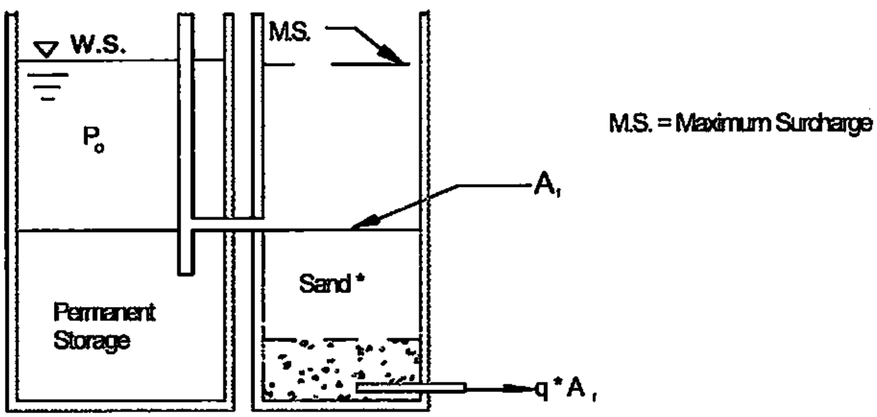
In Figure 4, Case 1 condition represents an arrangement where the filter is preceded by an extended detention basin, namely a basin that is totally evacuated of water after stormwater runoff ends. In Case 2 the filter is preceded by a retention pond with a surcharge extended detention volume above the permanent pool. In this case the permanent pool retains all or some of the runoff within it after storm runoff ends while the surcharge capture volume is totally evacuated after stormwater runoff ends. For Cases 1 and 2 the detained volume is evacuated through a flow control outlet. This outlet is designed to empty out the design capture volume over a desired time period, namely its *drain time*.

The detention outlet can also be oversized and the detention volume's evacuation rate can be governed by the size and flow-through rate of the filter itself. If this is the design condition, the filter will operate similarly to the one shown in Case 3, where at least a part of the detention volume resides directly above the filter's surface. Most common field examples for Case 1 can be found in Austin, Texas. The State of Delaware filter design is best represented by Case 3, as are the field conditions where the filter is incorporated into the banks of a retention pond above the permanent pool's surface. The latter design is commonly used in Florida. Case 3 was the condition tested in Lakewood, Colorado in 1995.

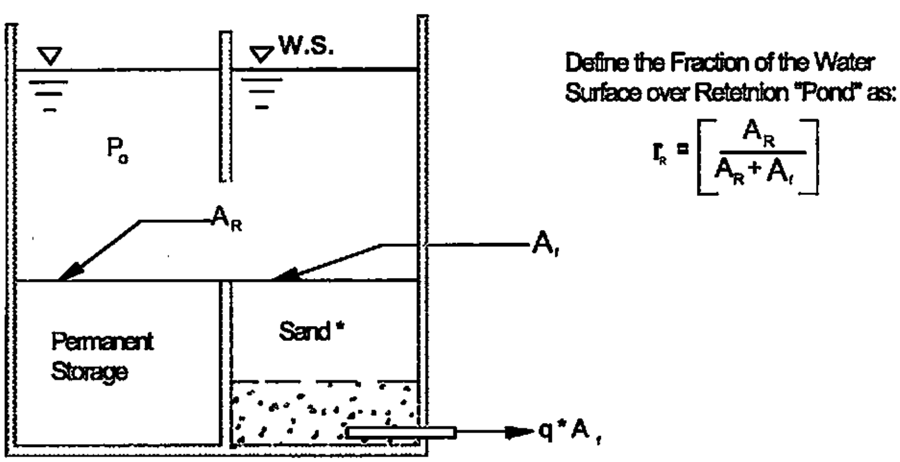
The detention/retention basin upstream of the filter also removes some of the solids since *TSS* can settle before the stormwater reaches the filter. The designer needs to estimate how much *TSS* is removed by the upstream detention/retention basin in order to estimate how much *TSS* may be left in the water column to be removed by the filter. This is not an easy estimate to make since there is much variability in the reported *TSS* removal rates by a detention or a retention basin.



Case 1: Detention Basin with Controlled Release Followed by a the Filter



Case : Retention Basin with Controlled Surcharge, Followed by a the Filter



Case 3: Combination Retention Pond & Media Filter Without Controlled Release to Filter

Figure 4. Three possible arrangements for a filter in relation to upstream detention basins.

A conservative design approach suggests that a lower value for *TSS* removals be used for design than the averages reported in literature for detention basins and retention ponds. For the same reason, *TSS* removal efficiencies used for the design of the filter itself should be based on higher removal rates than the average rates reported in the literature. The intent during the sizing of a filter is not to predict actual *TSS* removal rates accurately, but to use reasonable removal rates to arrive at realistic, possibly somewhat conservative filter size. Table 1 provides suggested design *TSS* removal rates for retention ponds and detention basins located upstream of the filter. These removal rates are somewhat lower than the averages reported in the literature. However, if locally collected information differs significantly, the designer should use such locally available data instead.

For Cases 1 and 2 defined in Figure 4 it is possible to assume that the concentration of *TSS* leaving the retention/detention basin can be estimated using :

$$E_{sd} = E_s \cdot \left(1 - \frac{R_D}{100}\right) \tag{5}$$

In which, E_{sd} = average concentration of *TSS* leaving the detention or retention basin, in mg/l
R_D = assumed percent removal rate for the retention or detention basin upstream of the filter bed (see Table 1)

The EMC of the effluent *TSS* leaving the filter after it has passed through retention or detention and the filter bed, is defined as:

$$E_{sf} = \left(1 - \frac{R_T}{100}\right) \cdot E_s \tag{6}$$

In which, E_{sf} = average annual EMC of *TSS* in the effluent from the filter bed, in mg/l
R_T = total system's average percent removal rate of *TSS*

Then the reduction in the EMC of *TSS* by the filter itself can be expressed as

$$E_{sfr} = E_{sd} - \left(1 - \frac{R_T}{100}\right) \cdot E_s$$

In which, E_{sfr} = the change in suspended solids concentration through the filter in milligrams per liter

After substituting Equation 5 into the above relationship and rearranging terms, we get

$$E_{sfr} = E_s \cdot \left(\frac{R_T - R_D}{100}\right) \tag{7}$$

For design purposes it is suggested that the value for R_T be equal to the highest reported rates of *TSS* removals by stormwater filters, namely R_T = 95 percent.

Table 1. Suggested Design Percent Removal Rates by Retention and Detention Upstream of a Media Filters for Sizing Them.

Detention Volume, P_o, Drain Time - T_d in hours	Suggested Percent Removal - R_D	
	Detention	Retention
48	60	90
24	55	85
12	50	80
6	40	75
3	30	70
1	20	50

For Case 3 shown in Figure 4 the above analysis needs to be modified. In Case 3 some of the detention storage volume is directly above the filter media. A first-order estimate of sediment removals ahead of the filter assumes that the water column that is not above the filter's surface acts as an independent retention pond. The water column that is above the filter's surface receives no pretreatment and all the *TSS* in this water is subject to removal by the filter.

Under the Case 3 scenario one can assume that the *TSS* concentration leaving the retention portion of the system can be expressed in terms of retention surface area and the total system surface area. Namely,

$$E_{sd} = r_R \cdot E_s \cdot \left(1 - \frac{R_D}{100}\right) \tag{8}$$

In which, r_R = $[A_R/(A_R+A_f)]$, ratio of the retention basin's surface area to the total system's surface area
A_R = surface area of the retention pond's permanent pool in square feet
A_f = surface area of the filter bed in square feet

Then the reduction in the EMC of *TSS* by the filter bed itself can be expressed by

$$E_{sfr} = E_s \cdot \left[\frac{R_T - r_R \cdot R_D}{100}\right] \tag{9}$$

Note that if all the detention storage is above the filter's surface, such as a basin with a sand filter bottom, $r_R = 0$ and all the *TSS* load is removed by the filter.

FILTER'S FLOW-THROUGH RATE

The classic relationship for water percolating through uniform soil media such as sand can be expressed as

$$q = k_h \cdot I \tag{10}$$

In which, q = flow velocity in inches per hour
k_h = hydraulic conductivity of the soil in inches per hour
I = hydraulic gradient in feet per foot

The relationship breaks down for a slow sand filter as fine sediment accumulates on top of its surface. In fact, field observation and laboratory tests (Neufeld, 1996; Urbonas *et al.*, 1996b) show that the flow-through rate for a sand filter (and other media as well) quickly becomes a function of the sediment being accumulated on the filter's surface. This relationship appears to be not very sensitive to the hydraulic surcharge on the filter's surface. It is represented graphically in Figure 5 and can be expressed mathematically as

$$q = k_i \cdot e^{-c \cdot L_m} \qquad (11)$$

In which, k_i = empirical flow-through constant (see Figure 5)

c = empirical exponential decay constant (see Figure 5)

L_m = cumulative unit *TSS* load accumulated on the filter's surface in pounds per square foot

It is this relationship that is used as the basis for the design procedure described later in this paper. Although the coefficients in Figure 5 are probably indicative of the expected performance for a sand filter, similar sets of coefficients can be developed for other filter media such as sand-peat mixes, etc. Namely, the procedure discussed here should be valid for other filter media provided appropriate empirical flow-through coefficients are employed . Examination of Figure 5 reveals that when the filter bed is new, the flow-through rates far exceed 12 inches per hour. As *TSS* is removed over the storm runoff season and the filtrate accumulates on the filter's surface, the flow-through rate rapidly drops off to approximately 0.9 inches per hour, after which it slowly continues to decrease to approximately 0.6 inches per hour.

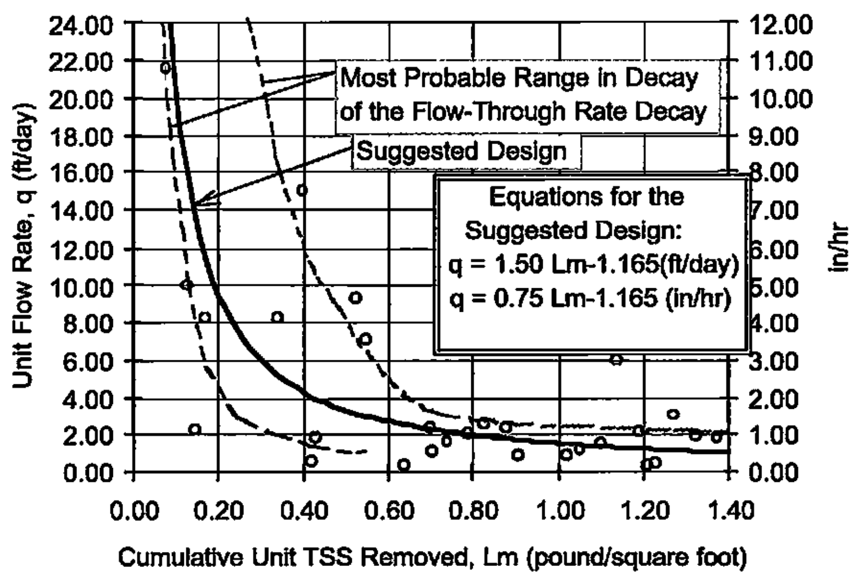


Figure 5. Flow Through Rate vs. Cumulative TSS Removed - Lakewood Sand Filter Test Site

The fraction of all runoff volume from the tributary area that will be treated through the filter facility is, in part, a function of the capture volume (i.e., detention) provided upstream of the filter. This detention volume can be bypassed by larger runoff flows, or the larger flows can first go through the detention basin before overtopping it and bypassing the filter itself. Depending on which condition occurs will also determine the amount of treatment provided to the excess volumes produced by larger storms. If the *maximized* capture volume is provided, approximately 80 to 90% of all runoff volume can be treated by the filter installation. If, however, the capture volume provided is based on the *mean* runoff volume, approximately 60% to 70% of all runoff volume will be fully processed through the filter. Approximate values of coefficient *a* to be used in Equation 1 can be found on Figure 2, which coefficient can be used to find the capture volume for the *mean* storm and the *maximized* storm.

The filter will need to be maintained to stay in operation. Its contaminated and clogged layers will need to be removed and replaced with new media. After a number of such surface cleanings (estimated at five to ten) the entire media filter will need to be replaced because lower pore spaces will also fill. The frequency of maintenance activities play a major, maybe a dominant role in the filter's design. It is appropriate then to define the *TSS* load removals in terms of the frequency of maintenance cycles the facility will experience each year. Also, since the flow-through rate in Equation 11 (i.e., Figure 5) is expressed as a function of the load removed by the unit area, it is appropriate to express the average *TSS* load removed during each maintenance cycle in terms of *TSS* load removed by each square foot of the filter. Thus,

$$L_m = \frac{L_{afr}}{A_{fm} \cdot m} \tag{13}$$

In which, L_m = average *TSS* load removed by each square foot of the filter during each maintenance cycle, in pounds per square foot per cycle

m = number of times per year the filter is cleaned and reconditioned (i.e., maintenance cycles per year). Use a fraction (i.e., 0.5) if more than one year between cleanings

A_{fm} = surface area of the filter sized on the basis of *TSS* for load removed, in square feet

SIZING THE FILTER

Rearranging the terms of Equation 13 yields an expression for estimating the filter's area, namely,

$$A_{fm} = \frac{L_{afr}}{L_m \cdot m} \tag{14}$$

which is one of two filter area relationships that have to be satisfied simultaneously. The other one is the ability of the filter to process the design storm's runoff volume (e.g., maximized volume) within the desired drain time. This condition can be expressed as

$$A_{fh} \cdot q \cdot T_d = P_o \cdot A_c \cdot 43{,}560$$

Rearranging terms the area of the filter is defined as

$$A_{fh} = \frac{P_o \cdot A_c \cdot 43{,}560}{q \cdot T_d} \tag{15}$$

In which, q = the design flow-through rate through the sand filter's surface, in inches/hour
T_d = the time it takes the volume P_o to totally drain out at the design flow-through rate q, in hours
A_{fh} = surface area of the filter based on hydraulic sizing, in square feet

The designer now has to find a filter's surface area that comes close to satisfying the condition

$$A_{fm} \approx A_{fh}$$

namely, the surface areas found using the *load removed* sizing equation and the *hydraulic sizing* equation are nearly identical.

The following design procedure is suggested for finding the required filter's surface area:

DESIGN PROCEDURE

1. *Determine the average EMC of TSS the tributary catchment will produce.*
Use local *TSS* stormwater characterization data when available. In absence of local data, use the closest regional averages of *TSS* found in stormwater reported in the Nationwide Urban Runoff Evaluation final report (EPA, 1983) or other, more current, data source. This will set a value for E_s for the design.

2. *Calculate the average annual TSS load in stormwater runoff from the design catchment.*
Use Equation 2 to find the catchment's runoff coefficient, C; Figures 1, 2 and 3 and Equation 3 to estimate the catchment's average annual runoff, P_A; and the value of E_s from Step 1 above, the catchment's tributary area, A_c, and the foregoing estimate of P_A in Equation 4 to estimate the average annual *TSS* load, L_a, being delivered by stormwater runoff to the filter installation.

3. *Select filter-detention/retention configuration and preselect its desired drain time (i.e., time it takes to fully evacuate the capture volume.*
It is suggested that Case 1 and 2 configurations (City of Austin, 1988) be used for tributary catchments with over one acre of impervious surface, while Case 3 be considered as a filter inlet for smaller sites (Shaver, 1994; City of Alexandria, 1992).

   It is necessary to assume or select the drain time, T_d, for the capture volume being used to size the filter. This is the determining factor for finding the "maximized" or the "mean" volume, P_o, whichever is used as the design water quality capture volume.

4. *Estimate the reduction in the EMC of TSS provided by the filter itself.*
Based on the filter's configuration being used (e.g., Case 1, 2 or 3 with a value for r_R), select the appropriate value from Table 1 for the removals by the detention or retention portion of the facility and use Equation 7 to calculate E_{sfr}.

5. *Estimate the average annual TSS load removed by the filter.*
Use Equation 12 to calculate a value for L_{afr}.
Assume b = 0.90 if a detention volume equal to P_o is provided.

6. *Determine the filter's annual maintenance frequency.*
Base this on how often the owner is willing and/or able to clean and restore the filter. For example, on the southwest coastal areas of the United States where almost all rainfall

takes place in a six-month period, if the owner is willing to clean the filter at least once a month during the wet weather months, set the value for $m = 6$. If, on the other hand the owner does not want to bother with frequent maintenance and will commit only to cleaning the filter once every two years, set $m = 0.5$.

7. *With the aid of Figure 5 select the acceptable unit TSS load before each cleaning.*
Initially it is necessary to assume a value for the unit *TSS* load removed, L_m. by the filter. This value will be used with Figure 5 to make the first estimate of the needed filter's surface area.

8. *Set the water quality capture volume for this installation.*
It is recommended that, as a minimum, a volume equal to the runoff from the "mean" average storm (see Figure 1) and the "maximized" volume be used for design. Using the drain time, T_d, assumed in Step 7 and Equation 1 to calculate a value for P_o.

9. *Make first estimates of the filter's area.*
Calculate the filter's area, A_{fm}, using Equation 14 and the values for L_a, E_s, and L_{afr} found in Steps 1, 2 and 5 respectively.

   Also, calculate the filter's area, A_{fh}, using Equation 15 and the values for P_o; the catchment's tributary area, A_c; the flow-through rate, q, using Equation 11 based on the value of L_m; and the assumed drain time T_d for P_o assumed in Step 3.

10. *Compare the two filter areas calculated in Step 9.*
If the two calculations give significantly different results, say more than 20% different; average the two areas; calculate a new value for the unit load removed by the filter, L_m; find a new flow-through rate using Equation 11 and repeat Step 9. Otherwise choose the larger surface area of the tow after rounding off, as the design area.

    Repeat this process as needed until the two area calculations are within 20% of each other. At that point use the larger of the two as the design surface area of the filter.

<u>EXAMPLES</u>

<u>Example 1</u>. A commercial site near Chicago, Illinois. The media filter will be preceded by an upstream extended detention basin. The known site conditions are as follows:

<u>Step 1</u>:

Tributary Area	A_c	=	1.5 acres
Expected EMC of *TSS*	E_s	=	120 mg/l
Average storm depth (Figure 1)	P_6	=	0.53 inches
Average number of storms per year $\geq$ 0.1 inches in depth (Figure 3)	n	=	55
Catchment's total imperviousness	I_a	=	85%

<u>Step 2</u>: Using Equation 2 find its runoff coefficient:

$$C = 0.858 \bullet 0.85^3 - 0.78 \bullet 0.85^2 + 0.77 \bullet 0.85 + 0.04 = 0.66$$

Using Equation 3 estimate the average annual runoff from the catchment:

$$P_A = 55 \bullet 0.53 \bullet 0.66 = 19.24 \ inches$$

Using Equation 4 calculate the annual *TSS* load from the catchment:

$$L_a = 1.5 \bullet 43{,}560 \bullet \frac{19.24}{12} \bullet \frac{120}{10^6} \bullet 62.4 = 784 \ lbs$$

<u>Step 3</u>: Select the filter's design configuration. Since the filter will be preceded by an upstream extended detention basin, we have Case 1 configuration. Also the outlet from the extended detention basin is designed to drain the capture volume in 12 hours.

<u>Step 4</u>: Using T_d = 12 hours, Table 1 gives for a detention basin a suggested removal rate R_D = 50 percent. Then, assuming an overall removal rate for the detention-filter system (i.e., R_T) is 95%, estimate the reduction in total solids concentration produced by the filter itself.

$$E_{sfr} = 120 \cdot \left(\frac{95-50}{100} \right) = 54 \ mg/l$$

<u>Step 5</u>: Using Equation 12 estimate the average annual *TSS* load removal by the filter itself.

$$L_{afr} = 0.90 \cdot \frac{54}{120} \cdot 784 = 318 \ lbs$$

<u>Step 6</u>: Determine the filter's annual maintenance frequency.
For this example assume m = 1 (i.e., once per year)

<u>Step 7</u>: To keep the size of the filter small while not imposing a very frequent maintenance schedule we choose to design the filter to drain at approximately 2.0 inches per hour. This means the corresponding value for L_m = 0.32 pound/square foot is found with the aid of Figure 5.

<u>Step 8</u>: Using T_d = 12 hours, the runoff coefficient from Step 2 and the coefficient from Figure 2 in Equation 1, find the "maximized" capture volume:

$$P_o = 1.12 \bullet 0.66 \bullet 0.53 = 0.39 \ watershed\ inches \ (2{,}124 \ cu.ft.)$$

<u>Step 9</u>: Using Equation 14:

$$A_{fm} = \frac{318}{0.32} = 994 \ sq.ft.$$

Using q = 2.0 *in./hr.* in Equation 15:

$$A_{fh} = \frac{0.39 \bullet 1.5 \bullet 43{,}560}{2.0 \bullet 12} = 1{,}062 \ sq.ft.$$

<u>Step 10</u>: Since the two areas calculated in Step 9 are well within 20% of each other, choose the larger of the two and round off. Namely the filter area scheduled for design is:

A_f = 1,060 *sq. ft.*

This design will require, on the average, one cleaning a year, each cleaning consisting of the removal and replacement of the top three inches of the sand bed. After five or more such cleanings, the entire filter bed will probably need to be replaced. A smaller filter could be used with additional cleanings each year. The designer may want to check to see if substantial savings in life-cycle costs could be achieved using higher maintenance frequencies and a smaller filter or using a larger filter with fewer maintenance cycles.

Example 2. Same as Example 1 except use a filter inlet, namely Case 3, with the retention pond's and filter's surface areas equal to each other, namely r_R = 0.5.

Steps 1 through 3 are the same as in Example 1.

Step 4. In Table 1 we find for a retention pond with T_d = 12 hours for its surcharge detention, the suggested TSS removal rate is R_D = 80 percent

then, using Equation 9

$$E_{sfr} = 120 \cdot \left[\frac{95 - 0.5 \cdot 80}{100}\right] = 66\ mg/l$$

Step 5. Using Equation 12 we find

$$L_{afr} = 0.9 \cdot \frac{66}{120} \cdot 784 = 388\ lbs.$$

Step 6. Assume m = 1.

Step 7. Using the same reasons stated in Example 1 we choose q = 2.0 *in./hr.* to begin the sizing process, thus

L_m = 0.32 lbs/ *sq. ft.*

Step 8: Same as in Example 1 @ T_d = 12 *hrs.*:

P_o = 0.39 inches (2,124 *cu. ft.*)

Step 9: Using Equation 14:

$$A_{fm} = \frac{388}{0.32} = 1{,}212\ sq. ft.$$

Using Equation 15:

A_{fh} = 1,062 *sq. ft.*

Step 10: Since these two are within 20% of each other, use the higher of the two. After rounding off recommend the following for design:

A_f = 1,200 *sq. ft.*

Again, one cleaning per year will be required to keep it operating as designed.

EXPECTED WATER QUALITY PERFORMANCE

What kind of hydraulic and water quality performance can one expect from a sand filter? The discussion above addressed the design of the filter based on hydraulic performance and how it varies as *TSS* was removed from stormwater runoff by the filter. The designer, planner and decision makers need to understand that stormwater runoff varies from zero to very large discharge numbers. It is a direct function of the precipitation, its duration and the tributary catchment's characteristics.

By providing a capture volume upstream of the filter that is in balance with the filter's flow-through capacity and after accounting for maintenance, it is possible to fully treat a large percentage of the storm runoff producing events through the filter, while treating some of the larger events only in part. The events that produce runoff at rates and volumes that exceed the capacity of the filter's physical plant will receive only partial treatment since the excess runoff will bypass the filter. Thus, the total system's performance is the composite of the filter's effluent water quality and the water quality of the bypass flow.

Hopefully, the worst polluted water will be captured by the filter's detention volume and will be treated through the filter, and only the cleaner "post first-flush" water will bypass the filter. The quality of the bypass water will also be affected by how the upstream detention or retention basin/pond is connected to the catchment's runoff.

If the basin/pond is in line with the flow after its capture volume is exceeded, stormwater will flow through the basin and the excess will overtop it. A properly designed extended detention basin or a retention pond should provide some treatment, through sedimentation, for the water that flows through it. Its efficiency may be diminished, but some sediment will be removed. A poorly designed or undersized basin may provide no water quality enhancement and may, in fact, cause some of the previously deposited sediment to resuspend and be flushed out.

If the detention/retention basin goes off-line when it is full, the excess runoff bypasses it. This arrangement is superior to in-line arrangement for high flows when the facility is not designed to handle high flows without resuspension of the previously settle solids. At the same time, it will generally produce lesser quality runoff during high flow events when the basin is properly designed to handle them.

The exact arrangement of water quality capture volume basin (i.e., retention or detention) in relation to the runoff system and the filter's size determine what one can expect the average annual EMCs that reach the receiving waters. Figure 6 illustrates the two cases, namely overflow of the excess and the bypass of the excess. To make a valid assessment of the average annual EMC for any constituent reaching receiving waters, the designer needs to flow-weight the concentrations of the effluent and the excess runoff from all the storms that occur, on the average, during a year. Namely, for Case 1 shown in Figure 6:

$$E_c = \left(k_T \cdot k_D \cdot E_i\right) \cdot \left(1 - r_{pf}\right) + E_f \cdot r_{pf} \qquad (14)$$

and for Case 2

$$E_c = \left(k_T \cdot E_i\right) \cdot \left(1 - r_{pf}\right) + E_f \cdot r_{pf} \qquad (15)$$

In which, E_c = average annual constituent's EMC downstream of the filter facility's installation, in mg/l

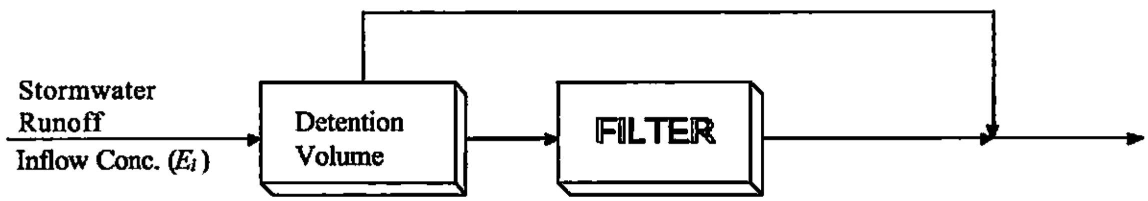
E_i = average annual constituent's EMC in the runoff inflow to the filter system, in mg/l

E_f = average annual constituent's concentration in the filter's effluent, in mg/l

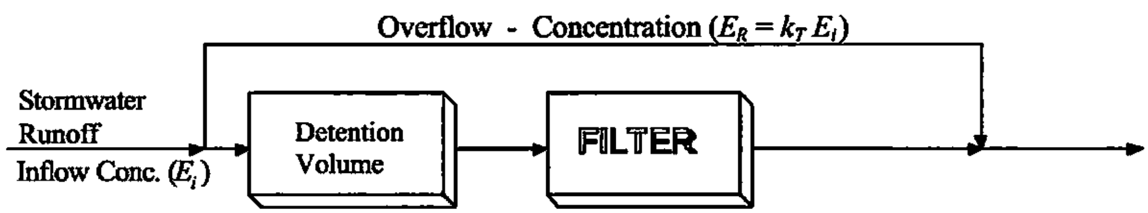
r_{pf} = fraction of the average annual runoff volume from the catchment that flows through the filter

k_D = fraction of the original constituent in the runoff that remains in the overflow water after the detention basin or retention pond overflows

k_T = coefficient of the reported constituent EMCs that represent the post "first-flush" fraction of the average EMC in stormwater runoff



Case 1. All runoff passes through the detention or retention basin upstream of the filter



Case 2. All runoff exceeding detention volume bypasses the filter and the detention/retention basin.

Figure 6. Two possible arrangements for a filter bypass with upstream detention volume.

Currently it is not possible to suggest definitive values for k_D and k_T, which coefficients depend on the constituent being considered and the actual design. However, a literature review suggests the following tentative ranges for *TSS*:

k_D = 0.3 to 0.5

and

k_T = 0.7 to 0.9

If the *maximized* coefficients suggested by Figure 2 for finding P_o are used, one can expect 80 to 90% of all runoff volume to be captured and treated through the filter, namely r_{pf} = 0.8 to 0.9. If, however, the runoff from the mean storm is used as the basis for design, one can expect approximately 60% to 70% of the runoff to be captured and treated through the filter, namely r_{pf} = 0.6 to 0.7.

Table 2 summarizes, after screening out the outliers, the findings of filter tests at four cities in the United States, namely, Alexandria, VA; Austin, TX; Anchorage, AK; and Lakewood, CO. Data for the first three were procured and consolidated into a single report by Bell et al. (1996) and the data for the Lakewood site were obtained by the Urban Drainage and Flood Control District in 1995. Note the high variability in the influent (i.e., stormwater runoff) measured concentrations for the six constituents reported here. Also note that the ratios between the high and the low concentrations are significantly less for the effluent. The variability in the influent appears to be primarily responsible for the large range in the report values of percent removed. However, most common removal rates for each constituent tend to cluster in a narrower range than the maximums. It is suggested that the designer look at the mean effluent (i.e., Out) concentrations in Table 2 to judge the filter's expected performance.

Table 2. Field Measured Performance Ranges of Sand Filters

Constituent	In or Out	Concentration *mg/l*			Percent Removed		
		Low	High	Mean	Low	High	MCR*
TSS	In	12	884	160			
	Out	4	40	16	8%	96%	80-94%
TP	In	0.05	1.4	0.52			
	Out	0.035	0.14	0.11	5%	92%	50-75%
TN	In	2.4	30	8.0			
	Out	1.6	8.2	3.8	(-130)%	84%	30-50%
TKN	In	0.4	28	3.8			
	Out	0.2	2.9	1.1	0%	90%	60-75%
TC_u	In	0.030	0.135	0.06			
	Out	0.016	0.035	0.025	0%	71%	20-40%
TZ_n	In	0.04	0.89	0.20			
	Out	0.008	0.059	0.033	50%	98%	80-90%

*MCR - Most Common Data Range

Returning to the earlier examples will illustrate the above discussion. In Example 1 an extended detention basin was used upstream of the filter. It is relatively easy to design this arrangement so that all runoff will pass through the detention basin and the excess runoff will overtop the pond. Let's further assume that k_D = 0.4 and k_T = 0.9. As a first order estimate we assume that 80% of the average annual runoff volume will pass through the basin and the filter and 20% will overflow the basin. If we assume that the filter will have an average effluent *TSS* concentration of 16 mg/l (see Table 2) then the average annual EMC of *TSS* downstream of the filter installation will be

$$E_c = (0.9 \bullet 0.4 \bullet 120) \bullet (1 - 0.8) + 16 \bullet 0.8$$

$$E_c = 21 \text{ mg/l}$$

Comparing this to the average EMC for *TSS* in stormwater runoff at that site (i.e., 120 mg/l) this installation will have 82% average annual removal efficiency for *TSS*. As a note of interest, it appears that the filter installation will produce only a marginal water quality improvement in *TSS* concentrations over a well-designed extended detention basin. Also, it appears that the filter's average effluent *TSS* and *TP* EMCs should be equivalent to one(s) produced by a well-designed retention pond. Similar estimates can be made for other constituents using the concentrations listed in Table 2.

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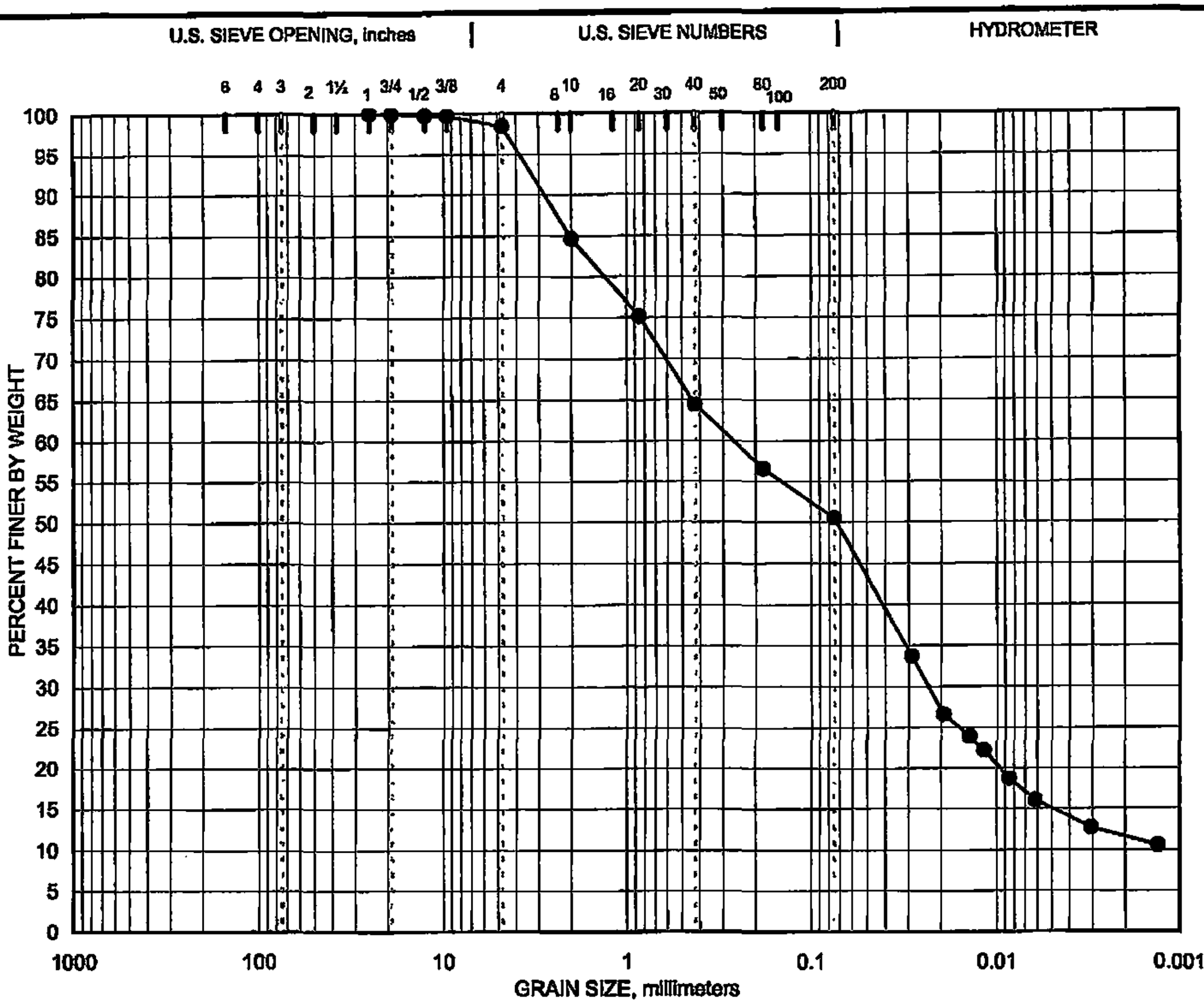
APPENDIX D
SAND FILTER MATERIAL SPECIFICATIONS

SAND SPECIFICATIONS:

Washed ASTM C33 Fine Aggregate Concrete Sand is utilized for stormwater management applications in Montgomery County. In addition to the ASTM C33 specification, sand must meet ALL of the following conditions:

1. Sand must meet gradation requirements for ASTM C-33 Fine Aggregate Concrete Sand. AASHTO M-6 gradation is also acceptable.
2. Sand must be silica based ... no limestone based products may be used. If the material is white or gray in color, it is probably not acceptable.
3. Sand must be clean. Natural, unwashed sand deposits may not be used. Likewise, sand that has become contaminated by improper storage or installation practices will be rejected.
4. Manufactured sand or stone dust is not acceptable under any circumstance.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
● L03262	**Sandy Silt (ML)**	9					

Specimen Identification	D100	D85	D60	D30	D15	D10	%Gravel	%Sand	%Silt	%Clay
● L03262	25.0	2.1	0.26				1.5	48	35.4	15.1

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PHYSICAL PROPERTY	UNIT US Values	TEST METHOD	MARV VALUES US Values
Weight (Typical)	oz./s.y.	ASTM D5261	4.5
Grab Tensile	lbs	ASTM D4632	120
Grab Elongation	%	ASTM D4632	50
Puncture Strength	lbs	ASTM D4833	70
Trapezoidal Tear	Lbs	ASTM D4533	50
Mullen Burst	psi	ASTM D3786	240
A.O.S.	U.S. Sieve	ASTM D4751	70
Water Permeability	cm/sec	ASTM D4491	0.22
Water Flow Rate	gpm/s.f.	ASTM D4491	120
Water Permittivity	(sec -1)	ASTM D4491	1.80
U.V. Resistance (500 Hours)	%	ASTM D4355	70

Note: Minimum average roll values are based on a 95% confidence level.

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