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Feasibility of Integrating Natural and Constructed Wetlands in Roadway Drainage System Design

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16. Abstract

Stormwater from roadways could have negative effects on the environment and aquatic ecosystems. Typical highway runoff pollutants include solids; heavy metals, particularly cadmium, copper, and zinc; petroleum hydrocarbons; gasoline constituents; PAHs (polyaromatic hydrocarbons); oxygen demanding compounds measured as COD (chemical oxygen demand) and BOD (biochemical oxygen demand); and road salts. Roadway runoff falls under the legislation of the Clean Water Act (CWA) via the National Pollutant Discharge Elimination System (NPDES). CWA regulates discharge of nonpoint source pollutants, such as roadway runoff, by issuing permits to public entities which manage Municipal Separate Storm Sewer Systems (MS4s). The objectives of volume I of this research were to characterize the pollutants in roadway runoff and determine the effectiveness of the existing stormwater BMPs at the study site. To accomplish these objectives, eleven rainfall events were sampled from November 2008 through November 2010. The objective of volume II was to fulfill the NDOR permitting requirement of creating a design guide for BMPs to remediate roadway runoff in Nebraska. BMPs which were most applicable to treating roadway runoff were those which removed 80% of the total solid load in the runoff, reduced metal concentrations to below acute toxicity levels, had low maintenance burden, were cost effective, did not pose a safety hazard to motorists, could be implemented within the right-of-way, did not negatively impact the road subgrade, and were aesthetically pleasing. The BMPs which best fit these criteria were vegetated filter strips, vegetated swales, bioretention, sand filters, and horizontal filter trenches. In this study fact sheets and design guides were compiled for each of these BMPs. The fact sheet provides background on the BMP including cost considerations, siting constraints, and predicted maintenance requirements. The design guide provides the process for sizing the BMP, design criteria the BMP must meet, and a design example which goes through the design process for a hypothetical application.

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Disclaimer

Abstract

Stormwater from roadways could have negative effects on the environment and aquatic ecosystems. Typical highway runoff pollutants include solids; heavy metals, particularly cadmium, copper, and zinc; petroleum hydrocarbons; gasoline constituents; PAHs (polyaromatic hydrocarbons); oxygen demanding compounds measured as COD (chemical oxygen demand) and BOD (biochemical oxygen demand); and road salts. Roadway runoff falls under the legislation of the Clean Water Act (CWA) via the National Pollutant Discharge Elimination System (NPDES). CWA regulates discharge of nonpoint source pollutants, such as roadway runoff, by issuing permits to public entities which manage Municipal Separate Storm Sewer Systems (MS4s). Part of the Nebraska Department of Roads (NDOR) permitting requirement is to create a design guide for Best Management Practices (BMPs) tailored to remediate roadway runoff in Nebraska.

The objectives of volume I of this research were to characterize the pollutants in roadway runoff and determine the effectiveness of the existing stormwater BMPs at the study site. To accomplish these objectives, eleven rainfall events were sampled from November 2008 through November 2010.

First flush and composite highway runoff samples were analyzed for heavy metals, anions, nutrients, particulates, BOD, COD, VOCs, and SVOCs. In addition to the concentrations, event loads were calculated using the hydrologic information from the study site. The results were compared to the Nebraska standards for water quality to establish which contaminants could have a negative impact on the environment. Additionally, an assessment of the effectiveness of the existing detention basin was completed, using the pollutant loads from the different outlet pipes.

Heavy metals, especially copper and zinc, total suspended solids (TSS), total dissolved solids (TDS), biological oxygen demand (BOD), and chemical oxygen demand (COD) were found to be the primary contaminants from the highway runoff. The current detention basin seems to be somewhat effective to reduce pollutant loads from small rainfall events. However, if pollutant reduction for all type of rainfall events is required, the basin should be modified into an extended detention basin which would provide better removal efficiency.

The objective of volume II was to fulfill the NDOR permitting requirement of creating a design guide for BMPs to remediate roadway runoff in Nebraska. BMPs which were most applicable to treating roadway runoff were those which removed 80% of the total solid load in the runoff, reduced metal concentrations to below acute toxicity levels, had low maintenance burden, were cost effective, did not pose a safety hazard to motorists, could be implemented within the right-of-way, did not negatively impact the road subgrade, and were aesthetically pleasing. The BMPs which best fit these criteria were vegetated filter strips, vegetated swales, bioretention, sand filters, and horizontal filter trenches. In this study fact sheets and design guides were compiled for each of these BMPs. The fact sheet provides background on the BMP including cost considerations, siting constraints, and predicted maintenance requirements. The design guide provides the process for sizing the BMP, design criteria the BMP must meet, and a design example which goes through the design process for a hypothetical application.

Chapter 1 Introduction

1.1 History of Roadway Stormwater Pollution Regulation

Since the early 1970s, an increasing awareness about the environmental impact of storm runoff resulted in the creation of environmental laws, executive orders, and policies that protect water quality. The acts and regulations that include highway runoff are: the National Environmental Policy Act (NEPA), Clean Water Act (CWA), the National Pollution Discharge Elimination System (NPDES), the Nonpoint Source (NPS) Management Programs, the Department of Transportation (DOT) National Transportation Policy, the Federal Highway Administration (FHWA) Environmental Policy Statement (EPS), and the Coastal Zone Reauthorization Amendment (CZRA). For purposes of this study, the CWA and its amendments are more relevant. More information about the other laws and regulations can be found in Appendix A.

The Clean Water Act (CWA) was originally called the Federal Water Pollution Control Act (FWPCA) of 1948. This law consisted of a regulatory system of water quality standards applicable to navigable waters. In 1972, the FWPCA included a system of standards, permits, and goals for fishable and swimmable waters to be achieved by 1983, and the total elimination of pollutant discharges into navigable waters by 1985. With the 1977 amendments, the name was changed to the Clean Water Act. The general purpose of this act was “to restore and maintain the chemical, physical, and biological integrity of the Nation’s waters” (Clean Water Act, 1977a), declaring unlawful the unregulated discharge of pollutants into all waters of the United States.

The amended section 402 of the CWA created a permit system, the National Pollutant Discharge Elimination System (NPDES). These discharge permits could be granted by the Environmental Protection Agency (EPA) or by EPA through approved state programs. These permits regulate discharges into navigable waters from point sources. According to the CWA, a point source is “any discernible, confined and discrete conveyance, including but not limited to any pipe, ditch, channel, tunnel, conduit, well, discrete fissure, container, rolling stock, concentrated animal feeding operation, or vessel or floating craft, from which pollutants are or may be discharged" (Clean Water Act, 1977b). The permits require that the permittee comply with discharge limits based on the use of “best technology” and with discharge and monitoring and reporting obligations.

The set of amendments passed in 1987 includes regulation of nonpoint source pollutants. According to these amendments, states are required to identify water bodies where water quality standards cannot be met without control of nonpoint source pollutants and to establish management programs for these water bodies. The plans need to include Best Management Practices (BMPs) for categories of sources, schedules of implementation milestones, and appropriate regulatory measures. Additionally, in Section 319 of the CWA, EPA is authorized to issue grants to states to help in the implementation of management programs that have been approved. Under these amendments, highway construction and operation was included as a source of pollution due to the accumulation of pollutants on the surface from oil, tires, dust, grease and other materials. These compounds may be washed into receiving waters during rain events. As a result, activities related with the construction, operation, and maintenance of roads and roadways need to be controlled and regulated by state regulations and programs related to nonpoint source pollution.

Section 319 of the Clean Water Act also established the Nonpoint Source (NPS) Program. Congress chose not to address nonpoint sources through a regulatory approach; instead, under Section 319, a federal grant program was created to provide money to states, tribes and territories for the development and implementation of NPS management programs.

Stormwater from roadways can contain several chemicals that could potentially have negative effects on the environment (FHWA and USGS, 2005). Therefore, several states have conducted studies to determine the contaminants and the potential impact that these pollutants may cause to the environment.

Chapter 2 Objectives and Scope

2.1 Objectives

The objectives of this project were to determine water quality of runoff from a selected Nebraska high-use roadway, evaluate the potential for existing stormwater BMPs to remove pollutants from stormwater from roadways, and improve the guidelines of roadside stormwater BMPs for pollutant removal. This thesis covers the first two objectives of the project, and the remaining objective was undertaken in the second stage of the project.

2.2 Scope

In order to accomplish these objectives, stormwater samples were collected in an existing detention basin located next to a high-traffic roadway. This detention basin receives water from a construction debris lot, two outlet pipes, and a grass section next to the roadway shoulder, and discharges the water to a nearby stream through an outlet pipe at the east end of the basin.

Concentrations from the roadway outlet pipes were compared to the basin outlet pipe to determine if concentrations were reduced when the stormwater passed through the detention basin. Loads for all the sampling sites were calculated using the hydrologic information obtained from the HEC-HMS model created for the project. The total pollutant loads deposited into the stream by the roadway were determined by subtracting the construction debris lot loads from the basin outlet pipe. Additionally, concentrations and loads were compared against factors affecting highway runoff (e.g., total suspended solids concentrations, antecedent dry period, volume of runoff, and total rainfall) to determine if correlations existed.

The first volume of this project is divided into seven sections and four appendices. Chapter 1 is an introduction to roadway stormwater pollution. Chapter 2 explains the purpose and scope of the project. Chapter 3 provides technical background on the topics covered in this thesis. Chapter 4 discusses the methods used in the project. Chapter 5 summarizes the results obtained. Chapter 6 contains the discussion of results. Chapter 7 contains the conclusions from this project.

Chapter 3 Literature Review

3.1 Typical Contaminants from Roadways

As part of the urbanization process, highways have become a potential source for an extensive variety of contaminants to surface and subsurface waters (Gupta et al. 1981; Barret et al. 1995; Chui et al. 1982; Mitton and Payne 1997).

3.1.1 Roadway Contaminant Sources

Sources of highway runoff pollutants can be classified into three different categories: a) vehicle traffic, b) snowmelt and ice-melt containing deicing chemicals, and c) chemicals used to manage roadside vegetation (U.S Environmental Protection Agency [EPA], 2005). Young et al. (1995) reported the major sources of pollutants on highways are vehicles, dustfall, and precipitation. Table 3.1 shows the major highway pollutant constituents and their sources (Kobriger, 1984).

Table 3.1 Primary contaminants and their major sources in highway runoff

Constituent	**Primary Source**
Particulates	Pavement wear, vehicles, atmosphere, maintenance, snow/ice abrasives, sediment disturbance.
Nitrogen, Phosphorus	Atmosphere, roadside fertilizer use, sediments.
Lead	Leaded gasoline, tire wear, lubricating oil and grease, bearing wear, atmospheric fallout.
Zinc	Tire wear, motor oil, grease.
Iron	Auto body rust, steel highway structures, engine parts.
Copper	Metal plating, bearing wear, engine parts, brake lining wear, fungicides and insecticides use.
Cadmium	Tire wear, insecticide application.
Chromium	Metal plating, engine parts, brake lining wear.
Nickel	Diesel fuel and gasoline, lubricating oil, metal plating, brake lining wear, asphalt paving.
Manganese	Engine parts.
Bromide	Exhaust.
Cyanide	Anticake compound used to keep deicing salt granular.
Sodium, Calcium	Deicing salts, grease.
Chloride	Deicing salts.
Sulphate	Roadway beds, fuel, deicing salts.
Petroleum	Spills, leaks, blow-by-motor lubricants, antifreeze, hydraulic fluids, asphalt surface leachate.
PCBs, pesticides	Spraying of highway right of way, atmospheric deposition, PCB catalyst in synthetic tires.
Pathogenic bacteria	Soil litter, bird droppings, truck hauling livestock/stockyard waste.
Rubber	Tire wear
Asbestos*	Clutch and brake lining wear.

*No mineral asbestos has been identified in runoff; however, some break-down products of asbestos have been measured.

3.1.2 Factors Affecting Highway Pollutants

Authors of different studies agree that runoff characteristics and pollutant accumulation are site-specific and are affected by several variables. Some of the factors affecting the pollutant accumulation (Gupta et al. 1981; Thiem et al. 1998; East-West 2000; Chui et al. 1982; Herrera 2007; EPA 2005; Barret et al. 1995) are:

- Traffic characteristics (volume, speed, braking);

- Climate conditions (intensity and form of precipitation, wind, temperature);
- Maintenance policies (sweeping, mowing, repair, deicing, herbicides);
- Surrounding land use (residential, commercial, industrial, rural);
- Percent pervious and impervious areas;
- Age and condition of vehicles;
- Anti-litter laws and regulations covering car emissions and delivery trucks;
- Use of special additives in vehicle operation;
- Vegetation types on the highway right-of-way; and
- Accidental spills.

The East-West Gateway Coordinating Council (2000) suggests that a stronger correlation of potential runoff impacts exists with storm characteristics such as the number of dry days preceding the storm event, the intensity of the storm, and the elapsed time of the total storm event. Of these three characteristics, the most critical is storm intensity because many of the pollutants attach themselves to particulate matter (or sediment); and the more intense the storm, the greater the movement of particulate matter. Long-duration storms usually reduce the pollutant concentrations due to the increased volume; however, the overall loading of pollutants is greater with these storms than with shorter but more intense storms.

In a study of particulates generated by traffic in Cincinnati (average daily traffic load was 150,000 vehicles), Sansalone and Buchberger (1997) found that 44% to 49% of particulates originated from pavement wear, 28% to 31% from tire wear, and 15% from engine and brake pad wear. The authors also found that 6% of particulates were deposited from settleable exhaust and 3% from atmospheric deposition.

Several studies (e.g., Gupta et al. 1981; Barret et al. 1995) have found that traffic volume is a factor affecting levels of pollutants in highway runoff. However, Driscoll et al. (1990) did not find any correlation between Total Suspended Solids (TSS), total solids, Biological Oxygen Demand (BOD), oil and grease, phosphorus, nitrate, Total Kjeldahl Nitrogen (TKN), or heavy metals with traffic density. Another study conducted by Kayhanian et al. (2003) at 83 highway sites in California, concluded that there is not an evident correlation between annual average daily traffic (AADT) and pollutant concentrations. This study found that a more consistent correlation emerged when mean and median concentrations were analyzed with medium and higher range of AADTs for urban highways. In another study, Kerri et al. (1985) also found that there was no statistically significant correlation between pollutant loads and the amount of traffic before the storm. Kerri et al. (1985) state that during dry periods there is a greater adherence of materials to the engine, undercarriage, and wheel walls of vehicles, while during a storm or wet period, there is more splashing and washing of materials from the vehicles. Therefore, they suggest that the number of vehicles during storm (VDS) may be a better predictor for constituent concentrations.

Hoffman et al. (1985) found that concentrations of various pollutants in highway runoff varied within the storm event. That is, suspended solids, petroleum hydrocarbons, lead, cadmium, copper, and iron concentrations were higher during the first flush. Additionally, during this period the most highway litter, such as Styrofoam cups, cigarette butts, beverage cans, and fast food wrappers, were present in the runoff. Hoffman et al. (1985) concluded that in general, peaks in pollutant concentrations occur during high flow rates when transport of contaminants is more efficient. However, peak concentrations may occur during lower flow conditions, due to reduced dilution.

Higher concentrations of pollutants are often observed during the first runoff from a storm. This phenomenon is often described as “first-flush” (Barret et al. 1995). According to the Center for Watershed Protection (1994) the first flush contains around 90% of the annual stormwater pollutant load; therefore, to treat 90% of the annual load, it would be necessary to treat the first half inch of runoff. Recent studies conducted by Chang et al. (1990) reveal that even though the first flush concentration is significantly higher, in many cases the first half inch does not carry most of the storm pollutant load. This study found that for developments with large areas of impervious cover the first half inch of runoff cannot transport as much of the pollutant load that is produced during larger storms. For example, for a development with 90% impervious cover, the first half inch of runoff of larger storms removed only about 40% of the total storm pollutant load on average.

Season and weather conditions may have a large impact on highway pollutant concentrations. Sansalone (1995) found that urban highway snowbanks can be significant sinks for metal elements and solids generated by traffic and maintenance activities. They also found that metal element concentrations are significantly correlated to solids concentrations in snowmelt. In contrast, metal element concentrations in rainfall events varied depending on the degree to which the metal element was dissolved, the presence of solid fractions, and the hydrology of the event and site. It is believed that physical entrapment and long residence times of metals and suspended solids in close proximity to the snow matrix may be the reason for a stronger correlation for snow wash-off events than rainfall-runoff events.

3.2 Sampling Methods for Roadway Pollutants

Studies done on highway runoff have used a variety of sampling methods. Some have used automatic samplers (Gupta et al. 1981; Granato and Smith, 1999; Horner et al. 1979; Desta et al. 2007; Kayhanian et al. 2003; Khan et al. 2006; Sansalone and Buchberger, 1997; Barrett et al. 1998; Wu et al. 1998). Others have used grab sampling methods (Horner et al. 1979; Kayhanian and Stenstrom, 2005; Khan et al. 2006; Hoffman et al. 1985; Shinya et al. 2000; Thiem et al. 1998; Little et al. 1983; Marsalek et al. 1997) and some have used a combination of two or more sampling techniques (Mitton and Payne, 1997; Wu et al. 1998).

Gupta et al. (1981) obtained data on traffic characteristics, highway maintenance, precipitation, air particulate fallout (dustfall), and runoff quantity and quality at numerous sites. The monitoring of storm events was conducted for a minimum 12 month period at each site. Instrument Speciality Company (ISCO) water quality samplers (models 1392 and 1680) were used to sample the storm events. Two ISCO samplers were installed at each site to collect samples at intervals of 5 and 15 min. The samplers were installed behind weirs and flumes where a maximum turbulence was experienced during the runoff events. Manual grab samples were required to analyze for oil and grease, pesticides/herbicides, and PCBs because plastic bottles used in the autosamplers may absorb some of these pollutants.

Gupta et al. (1981) recommend that during the first stages of the event, discrete samples should be collected at more frequent intervals. Drainage areas with a high proportion of paved area require sampling frequencies of 2 to 5 min during first flushes, and 15 to 30 min after that until runoff ends. Drainage areas with a high proportion of unpaved area require sampling frequencies of 5 to 15 min during first flushes and 15 to 60 min after that.

Granato and Smith (1999) used an automated sampler that took samples immediately downstream of the throat section of a Palmer-Bowles flume in order to take advantage of the mixing action of the flume throat. The automated sampler was triggered by changes in specific conductivity.

Desta et al. (2007) used a full-size portable automatic water quality sampler (ISCO 6712) with 24 glass bottles each with 350 mL capacity. The sampler was programmed to trigger during storm events and collect samples according to the volume of runoff passing the measuring section. The 24 bottles were divided into two parts: Part A (bottles 1 to 6) and Part B (bottles 7 to 24). Bottles in Part A were used to account for the first-flush effect sample, approximately 25% of the storm, and the last 18 were used in Part B for the rest of the event.

Barrett et al. (1998) used ISCO 3700 automatic samplers. The automatic samplers were programmed to sample based on the volume of runoff passing the sampler. At two sites, the sampler was programmed to draw samples at set volumes of flow. At a third location, the sampler was initially programmed to collect samples on a timed basis, but was later converted to collect flow-weighted composite samples.

Kayhanian and Stenstrom (2005) collected a series of grab samples during storm events. In the first year, five to six grab samples were collected during the first hour and a manually composited sample was collected over the following hours. In the second, third, and fourth years, five grab samples were collected in the first hour, followed by one grab sample per hour until the end of the 8th hour. If the storm lasted beyond the 8th hour, one or two additional grab samples were collected to characterize the tail of the runoff. Samples were collected in 4 L amber glass bottles.

Thiem et al. (1998) used glass and plastic (HDPE) bottles to collect runoff samples. Their major goal was to collect samples in the period immediately after the beginning of the storm. During this initial rainfall period (approximately 30 min in duration), samples were taken every five minutes. After this period, the sampling interval time was gradually increased to one sample every 30 min.

Shinya et al. (2000) collected runoff samples manually with a clean 20-L plastic bucket.

Samples were collected during four different periods during a rainfall event. At the beginning of the storm, samples were collected every 5 min for the first 15 min, and then every 15 min until one hour had elapsed from the beginning of the storm. Then two samples every half an hour were taken. Finally, samples were taken every hour until the end of water runoff or six hours had elapsed from the beginning of the storm.

Hoffman et al. (1985) collected samples manually from the outfall with buckets. A metal bucket was used for hydrocarbons and solids, and a plastic bucket was used for heavy metals analysis. Sampling was started at the beginning of the rainfall event and ended when the drain flow subsided following the end of the storm. Samples were usually collected every 30 min or more frequently (every 10 to 15 min) when flow rates were high or rapidly changing.

Little et al. (1983) used the composite sampling system and procedures developed by Clark et al. (1981). This system consisted of a calibrated flow-splitter and collection tanks. The flow-splitters were designed for each sampling site to capture a set proportion of the design storm flow, typically 1% to 2%.

Marsalek et al. (1997) used custom-made fluidic devices in the form of a stainless steel flow splitter, formed by a cylinder with closed bottom and 16 openings (D = 12 mm) in the side wall, just above the bottom. Each of the openings conveyed 1/16 of the inflow. Outflow from one of the openings was then directed to a second stage splitter of similar design. Outflows from the remaining 15 openings were discarded.

Wu et al. (1998) used ISCO or American Sigma automatic samplers to collect discrete samples from two of the three study sites. For site III, an elliptical flow divider was installed at the end of the drain pipe to collect composite samples. The divider provides accurate diversion for a fixed fraction of runoff, proportional to the flow rate. Water coming out the divider was collected in a holding of 48 X 47 X 36 in. capacity. The automatic samplers were programmed depending on the runoff event. Each automatic sampler was programmed to collect discrete samples at a preset time increment to ensure the collection of at least six to eight samples. Composite sampling was done by taking a fixed amount of sample at equal flow intervals during a storm.

Khan et al. (2006) collect both grab and composite samples. Grab samples, 4 L each, were collected at 15 min intervals during the first hour of runoff and at one hour intervals over the next seven hours. For storms lasting more than seven hours, additional grab samples were collected to capture the end of the storm. Grab samples were collected by bailing from a freefall of runoff exiting a discharge pipe. An American Sigma model 950 automatic sampler was used for flow-weighted composite samples. The automatic sampler was allowed to run until the end of runoff.

Horner et al. (1979) took three different types of samples where discrete sampling was the most common. For the discrete samples, one-liter portions of highway runoff were collected in plastic bottles. These samples were typically collected every 5 to10 min during the first hour of a storm, every 15 to 20 min during the second and third hours and every 30 to 60 min thereafter. Composite samples were taken directly from drums after the contents were thoroughly mixed with a stick. Composite sampling was used at the beginning of the storms when runoff began to collect before the technician reached the monitoring station. The third set of samples was collected using an ISCO Model 1680 and a Manning Model 4050 automatic samplers.

Mitton and Payne (1997) collected grab samples (usually at the upstream end of a flume) within 30 min of the onset of runoff, or as soon thereafter as possible. Remaining samples were collected using an automatic sampler that held 28 1-L bottles. The flow-composited samples were collected based on even increments of flow.

As discussed above, different techniques can be used to collect samples. Selection of a specific sampling technique depends on the specific sampling goals for the study. If the study requires analyzing VOCs, automatic samplers cannot be used because VOCs will likely volatilize as a result of agitation during automatic sampler collection (U.S. EPA, 1992). Table 3.2 shows a comparison between manual and automatic sampling techniques.

Table 3.2 Comparison of manual and automatic sampling techniques*

Sampling Method	**Advantages**	**Disadvantages**
Manual grab	• Appropriate for all pollutants • Minimum equipment required	• Labor-intensive • Environment possibly dangerous to field personnel • May be difficult to get personnel and equipment to the storm water outfall within the 30 min requirement • Possible human error
Manual flow-weighted composites (multiple grabs)	• Appropriate for all pollutants • Minimum equipment required	• Labor-intensive • Environment possibly dangerous to field personnel • Human error may have significant impact on sample representativeness • Requires flow measurements taken during sampling
Automatic grabs	• Minimizes labor requirements • Low risk of human error • Reduced personnel exposure to unsafe conditions • Sampling may be triggered remotely or initiated according to present conditions	• Samples collected for oil and grease may not be representative • Automatic samplers cannot properly collect samples for volatile organic compounds analysis • Costly if numerous sampling sites require the purchase of equipment • Requires equipment installation and maintenance • Requires operator training • May not be appropriate for pH and temperature • May not be appropriate for parameters with short holding times (for example, fecal streptococcus, fecal coliform, chlorine) • Cross-contamination of aliquot if tubing/bottles not washed
Automatic flow-weighted composites	• Minimizes labor requirements • Low risk of human error • Reduced personnel exposure to unsafe conditions • May eliminate the need for manual compositing of aliquots • Sampling may be triggered remotely or initiated according to on-site conditions	• Not acceptable for volatile organic compounds sampling • Costly if numerous sampling sites require the purchase of equipment • Requires equipment installation and maintenance, may malfunction • Requires initial operator training • Requires accurate flow-measurement equipment tied to sampler • Cross-contamination of aliquot if tubing/bottles not washed

*(Adapted from U.S. EPA 1992).

3.3 Flow Monitoring

Gupta et al. (1981) indicated that two components are required for the measurement of flow in an open channel:

- A calibrated device inserted in a channel.
- A level sensing instrument which measures the water level upstream of the calibrated device.

In this study, V-notch weirs and Palmer-Bowlus flumes were used as calibrated devices.

Flow measurements are used to develop composite water samples. According to the U.S. EPA (1992), flow rates for storm water discharges are most accurately measured using either primary or secondary flow measurement devices. Primary flow measurement devices are man-made flow control structure which is inserted into an open channel, creating a geometric relationship between the depth of the flow and the rate of the flow. The most common primary flow devices are weirs and flumes. Secondary flow measurement devices are automated forms of flow rate and volume measurement. Usually, a secondary device is used in conjunction with a primary device to automatically measure the flow depth or head. Some secondary devices are floats, ultrasonic transducers, pressure transducers, and bubblers.

It is also possible to estimate the flow rates using float methods, bucket and stopwatch, slope and depth, and runoff coefficient. These methods are not as accurate as the primary and secondary devices, but are appropriate for sites where primary or secondary devices are not practical or economically feasible. A full description of these methods can be found in U.S. EPA (1992).

Flow rates have been monitored using several devices or structures. Hoffman et al. (1985) used a Marsh-McBirney portable flow meter mounted on the drain headwall to monitor the flow velocities. A scale was painted on the interior of the drain to measure the water height during a rainfall event. Height measurements were taken at the time of sample collection.

Mitton and Payne (1997) used Parshall flumes with 3-in. or 6-in. throats to measure runoff flow rates. Flumes with 3-in. throats were used to measure runoff flows ranging from 0.03 to 0.89 ft^3/s, and flumes with 6-in. throats were used to measure flows ranging from 0.05 to 2.06 ft^3/s.

Wu et al. (1998) used an automatic sampler to continuously record flow stages that later were converted to flow rates. They also used a potentiometric water-level recorder to measure the level in the holding tank. These measurements were converted to flow rates.

Barrett et al. (1998) measured water-levels using bubble flow meters (ISCO 3230). These measurements were later converted into flow rates using a rating curve developed for each study site.

Horner et al. (1979) measured flow rates at least once during the period between samples. Flow rates were determined by measuring the level rise in the water collection drums with a meter stick over an interval timed by a stop watch. Measurement of the cross-sectional area of the drums permitted conversion of the timed level rise to units of volume per unit of time.

Desta et al. (2007) used two types of flow meters to measure flow rates. A bubble module (ISCO 730) was used to measure head over a weir which relates water level to flow rate. An area-velocity module (ISCO 750) was used to measure flow in all piped drainage systems where installation of a weir was not possible due to the risk of causing blockage.

Hallberg and Renman (2008) used two Parshall flumes to measure flows. The first Parshall flume was designed for flows between 1 l/s to 20 l/s, and the second flume was used for flows between 20 l/s and 600 l/s. The flow from 1 l/s to 20 l/s was registered with a Chanflo

Open Channel (Danfoss) flowmeter (0 to 0.3 m) with a Sonolev sensor (100 KHz). The flow between 20 l/s to 600 l/s was registered with Chanflo Open Channel (Danfoss) flowmeter (0 to 1 m) with a Sonolev sensor (100 KHz). The data from the flow measurements were collected every 60 s during the runoff event with a Campbell Scientific CR 10X data logger.

Thiem et al. (1998) installed a V-notch weir on a paved waterway where the samples were collected. The depth over the crest was measured with a ruler every time a sample was taken. At other sites, Manning's equation was used to determine the flow in partly full pipes.

3.4 Pollutant Concentrations and Loadings

The pollution discharging from a highway can be characterized in terms of concentration or in terms of loading rates. There are several ways to characterize the pollutant concentration in highway runoff. For example, pollutant concentrations can be reported as the concentrations in discrete sequential samples collected at intervals during a single storm event, or they can be reported as event mean concentrations (EMC) (Driscoll et al. 1990). The EMC is the average pollutant concentration found in the total volume of runoff from a storm event (Driscoll et al. 1990).

Loading rates for a storm are typically calculated by multiplying the EMC times the total volume of runoff during the storm. This method gives the mass of a pollutant discharged per time or per event. In a slightly different way, Wu et al. (1998) reported the long-term average pollutant loading rates (kg/ha-yr). To obtain these loads, the site mean loading rates are multiplied by the ratio of average storm duration to the average time between storms.

Shinya et al. (2000) reported initial concentration, EMC, and pollutant load. Initial concentration refers to the concentration in the first runoff water. Pollutant load was calculated using equation 3.1:

$$Cummulative\ load\ per\ drainage\ area = \frac{1}{A}\int C(t)Q(t)dt \qquad (3.1)$$

where A is the drainage area, C(t) is the time-variable concentration, Q(t) is the time-variable flow, and t is the time elapsed from the start of the event.

Barrett et al. (1998) reported the EMCs and the annual pollutant loads for three selected sites along the MoPac Expressway in the Austin, Texas area. The annual constituent loads were calculated based on the “simple method” described by Schueler (1987). The “simple method” is used to estimate stormwater runoff pollutant loads for urban areas. This method is based on regression correlations to predict loads based on land use. The information required in this method is the subwatershed drainage area and impervious cover, stormwater runoff pollutant concentrations, and annual precipitation. Equation 3.2 was used to estimate pollutant loads:

$$L = \left[\frac{(P)(CF)(R_v)}{20.4}\right](C_i) \qquad (3.2)$$

where L is the annual pollutant load in kg/ha, P is the annual precipitation (825 mm/yr), CF is the correction factor that adjusts for storms where no runoff occurs (0.9), R_v is the average runoff coefficient, and C_i is the event mean concentration.

The United States Geological Survey (USGS) developed a set of equations based on regression analyses of data from different sites around the country (Driver and Tasker, 1990). These equations can be used to predict storm-runoff loads and volumes, storm-runoff mean concentrations, and mean seasonal or annual loads.

Chapter 4 Methods

4.1 Site Selection

The site selected was in Omaha, Nebraska. The site is the property of the Nebraska Department of Roads (NDOR). This location was selected for two main reasons: 1) the site is next to the intersection of two major Interstates in Omaha (I-80 and I-680). In 2006, this intersection was ranked the third busiest intersection in the Omaha metro area (Metropolitan Area Planning Agency [MAPA], 2010); and 2) a detention basin is present to collect runoff from the site and to provide flood protection for the area. This basin might help to reduce some of the pollutant load coming from the highway. The location of the study site is shown on figure 4.1.

4.2 Site Description

The site is located in the southeast corner of the junction between Intestate I-80 and I-680. The Average Daily Traffic (ADT) on the road section under study is 145,100 vehicles per day (MAPA, 2008). For purposes of this study, the site was subdivided into 6 subbasins. Characteristics of each subbasin are given table 4.1.

Table 4.1 Characteristics of the study site

Subbasin	Area (mi^2)	Cover Type
Subbasin 1	0.00969	Construction debris material
Subbasin 2	0.00287	Grass, pavement
Subbasin 3	0.00143	Pavement
Subbasin 4	0.00125	Grass
Subbasin 5	0.006275	Pavement
Subbasin 6	0.00502	Grass

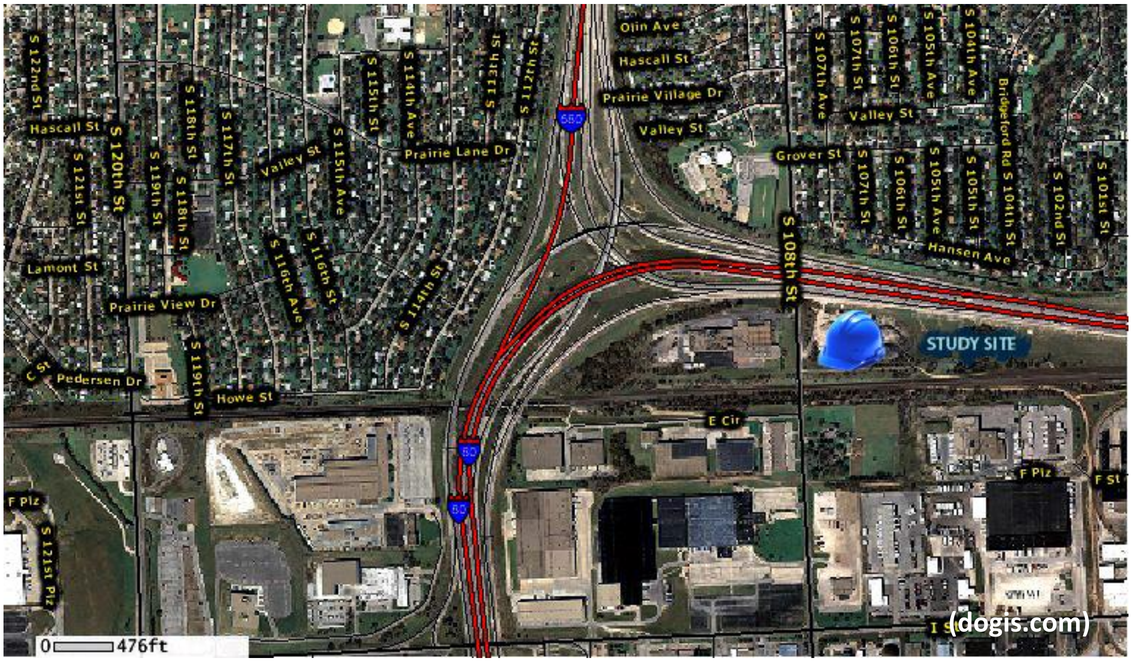

Figure 4.1 Zoom out to show I-680, I-80, 108th St (Location site)

4.3 Sampling

4.3.1 Sampling Goals

The sampling goals for this project were: obtain first flush and event mean concentrations (EMC) for the pollutants found in highway runoff, determine the pollutant loads discharging from the roadway at this site, and evaluate the impacts these contaminants may cause to the receiving waterway. There are five sampling sites (A, B, C, D, and E) shown in figure 4.2.

Figure 4.2 Study site, subbasins show drainage areas leading to sampling sites A-E

4.3.2 Storm Identification

Since this study required grab samples of the first flush from runoff events, potential runoff-producing storms had to be identified in advance so that preparations for sampling could be made.

To identify potential storms to be sampled, the Weather Channel (Weather, 2009), the National Oceanic and Atmospheric Administration (NOAA) (NOAA, 2009), Accuweather (Accuweather, 2009), and Weather Underground (Weather Underground, 2009) websites were monitored. The main data used to identify the storms were the predicted chance of precipitation and the Doppler weather radar. The sampling team was notified of a possible sampling event if the chance of precipitation approached 70%. After the team was notified, the weather was closely monitored to report any change that may affect the sample collection. Storms were targeted for potential sampling if they were predicted to be at least 1 hr long and have a minimum depth of 0.5 in.

4.3.3 Flow Measurement

4.3.3.1 Flow Monitoring

To measure the flow at the site, three different kinds of structures were built and installed. At site A (discharge point from the construction debris lot), a rectangular sharp-crested weir made of 1 in. thick treated wood supported by a 2 X 4 (treated wood) frame was constructed and covered by 1/16” plastic (to make impermeable and provide sharp crest). This weir was installed at the east side of the lot where discharge from the lot enters the basin. Figure 4.3 shows the weir being installed at site A. At site B (ditch), a V-notch weir was installed where flow in the ditch discharged to the basin. A 6-in. high berm lined with plastic sheeting (i.e., creating a long “sand bag”) was built along the fence to direct all flow toward weir and basin. Figure 4.4

shows the V-notch weir installed at site B. For sites A and B, a metallic ruler was used to determine the flow depth at the rectangular and V-notch weir, respectively. Measurements were taken when samples were collected at each site.

Figure 4.3 Rectangular weir installed at site A

For sites C and D (west pipe and east pipe outlets), 8-in. wide by 6-ft long rectangular flumes were installed. At site E (basin outlet pipe), a 24-in. by 6-ft long rectangular flume was installed (fig. 4.5). In sites C, D, and E, Isco 2150 Area Velocity (AV) Flow Modules and Sensors were installed. At first, the sensors were mounted 2 in. above the channel's bottom on a vertical PVC pipe placed at the end of each flume. After the first two sampling events, the pipes were removed because debris was getting trapped in the channel, affecting the sensor readings. From the third storm, sensors were mounted on the bottom of each flume. Flow modules were set to obtain a reading every five minutes. Also, during the first three storms, manual depth and velocity monitoring measurements were taken with a ruler and float to corroborate sensor readings.

Figure 4.4 V-notch weir installed at site B

Figure 4.5 Rectangular flume installed at site E

The flow at site A was calculated using the following equation for a rectangular sharp-crested weir with end contractions (U.S. Department of Interior Bureau of Reclamation, 2001).

$$Q = \frac{2}{3} C_d \sqrt{2g} (L - 0.1nH) \left[\left(H + \frac{v_0^2}{2g} \right)^{3/2} - \left(\frac{v_0^2}{2g} \right)^{3/2} \right] \tag{4.1}$$

Where:

C_d = Discharge coefficient, 0.6

g = Gravitational acceleration (32.2 ft/sec^2)

L = Weir length (ft)

n = Number of end contractions

H = Water height above the crest of the weir (ft)

v_0 = Approach velocity (ft/sec)

The weir installed at site A was contracted at both ends; therefore, the value of n was 2. The approach velocity was estimated by using equation 4.1 iteratively until the discharge (Q) and head (H) in the equation matched.

Methods to estimate the flow in site B have not been applied because there has not been flow in this site during the sampled storms. It was believed that the sampled storms had insufficient intensity to produce measurable runoff at site B (generally high intensity storms were not sampled because of dangerous lightning). However, on July 4, 2010, the sampling team had the chance to sample a high intensity storm that did not produce any lightning. During this storm, it was found that significant amounts of runoff were leaking through the berm at the fence and bypassing site B. As a result, the flow estimates for this site were developed from the HEC-HMS model.

For sites C, D, and E, the flow was measured by the AV flow modules and sensors. The AV sensor has an internal differential pressure transducer that measures the flow depth. The AV sensor has a pair of ultrasonic transducers which measure the average velocity using sound waves and the Doppler Effect. The first transducer sends the ultrasonic wave, and the second transducer receives the wave reflected by bubbles and particles in the stream (Isco, 2005). Using the information obtained by the AV sensor, the AV flow module internally estimates the flow rate in the channels based on the channel dimensions and the sensor's height above the channel bottom.

4.3.3.2 Hydrologic Model

An HEC-HMS model of the site was developed and calibrated using data obtained from 2008 and 2009 storm events. The six subbasins shown in figure 4.1 were further divided into eight different subbasins by splitting subbasin 1 and subbasin 5 into two subbasins each. Table 4.2 shows the identification name and the description of each subbasin created in the model.

Table 4.2 HEC-HMS subbasins

Identification Name	Description
Subbasin 1A	Hard packed driveway in the construction debris lot.
Subbasin 1B	Construction material piles and soil in the construction debris lot.
Subbasin 2	Area along the fence, interstate shoulder, and some of I-80 Eastbound entrance lane.
Subbasin 3	Tributary area for the west pipe inlet. This area includes the two outside lanes of the I-80 East bound and interstate shoulder.
Subbasin 4	Grass and shoulder area and some pavement from the center lanes that provide runoff directly into the detention basin.
Subbasin 5A	Tributary area for the east pipe inlet. This area includes the two inside lanes of the I-80 Eastbound and the four lanes of the I-80 Westbound.
Subbasin 5B	Tributary area for the East pipe inlet. This area includes a segment of the two outside lanes of the I-80 Westbound and the grass area between I-80 Westbound and the Exit 445 ramp.
Subbasin 6	Detention basin.

The construction debris area and the tributary area of the east pipe were each divided into two sub-areas because these sub-areas exhibit different runoff characteristics. For example, water coming from the hard packed driveway in the construction debris area moves faster compared to water coming from the construction material piles. In a similar way, water coming from the pavement on I-80 moves faster than water coming from the grass strip between I-80 and the Exit 445 ramp.

The HEC-HMS model provided hydrographs of the flows at each sampling point. These flows were used to determine the aliquots to use in the sample compositing process as described below. A full description of the HEC-HMS model and its application is given in Appendix B.

4.3.4 Sample Collection

Samples were collected using grab sampling methods. Nalgene and glass bottles were used to collect the samples. Bottles were rinsed twice with the stormwater prior to sample collection. In the construction debris lot, samples were collected downstream of the rectangular weir. For the east and west pipe inlet, samples were collected as close as possible to the pipe outlets, before the water reached the flumes. For the outlet pipe, samples were collected after water flowed through the flume and before it reached the outlet pipe.

Two sets of samples were taken during the first sampling round. The first set of samples captured the first flush, and the second set was used as part of the composite sample. To catch the first flush required the first sampling round to be taken within about 15 min after the beginning of the storm. Samples used in the composite were then taken approximately every 20 min from the beginning of the storm. Table 4.3 summarizes the date each sample was taken, type of bottles used, and sample preservation.

Table 4.3 Type of bottles and preservatives used for each sampling event

Date	Analytes	Type of Bottle Used	Preservative
11/10/2008	Metals, COD, BOD, TP, SP, TS, TSS,TDS, TVS, VSS, VDS, VOCs, PAH, F^-, Br, Cl^-, PO_4P, SO_4^{2-}, HCO_3^-, NO_3^{-2}, NO_2^-, O&G	2-L Nalgene	Ice
5/12/2009	Metals, COD, BOD, TP, SP, TS, TSS,TDS, TVS, VSS, VDS, VOCs, PAH, F^-, Br, Cl^-, PO_4P, SO_4^{2-}, HCO_3^-, NO_3^{-2}, NO_2^-, O&G	2-L Nalgene	Ice
6/1/2009	Metals, COD, BOD, TP, SP, TS, TSS,TDS, TVS, VSS, VDS, VOCs, PAH, F^-, Br, Cl^-, PO_4P, SO_4^{2-}, HCO_3^-, NO_3^{-2}, NO_2^-, O&G	2-L Nalgene	Ice
7/3/2009	Metals, COD, BOD, TP, SP, TS, TSS,TDS, TVS, VSS, VDS, PAH, F^-, Br, Cl^-, PO_4P, SO_4^{2-}, HCO_3^-,O&G	2-L Nalgene	Ice
	VOCs,	300-mL glass	Ice
7/31/2009	O&G, F^-, Br, Cl^-, PO_4P, SO_4^{2-},Si, HCO_3^-, TP, SP, TS, TSS, TDS, TVS, VSS, VDS, COD	2-L Nalgene	Ice
	PAH	1-L glass	Ice, H_2SO_4
	Heavy metals	500-mL glass	Ice, HNO_3
	TKN, NO_2N, NO_3N	250-mL glass	Ice, H_2SO_4
	VOCs	40-mL vials	Ice, HCL
9/3/2009	O&G, F^-, Br^-, Cl^-, PO_4P, SO_4^{2-},Si, HCO_3^-, TP, SP, TS, TSS, TDS, TVS, VSS, VDS, COD	2-L Nalgene	Ice
	PAH	1-L glass	Ice, H_2SO_4
	Heavy metals	500-mL glass	Ice, HNO_3
	TKN, NO_2N, NO_3N	250-mL glass	Ice, H_2SO_4
	VOCs	40-mL vials	Ice, HCL
3/27/2010 5/7/2010 5/20/2010 7/4/2010 9/13/2010 11/12/2010	Ca, Mg, Na	500-mL Nalgene	Ice, HNO_3
	BOD, TSS, TDS, Cl^-, Diss. Ortho P	1-L Nalgene	Ice
	COD, NO_3, TKN, TP	250-mL Nalgene	Ice, H_2SO_4
	Heavy metals	1-L glass	Ice
	VOCs	40-mL vials	Ice, H_2SO_4
	SVOCs	1-L glass	Ice

4.4 Pollutant Concentration Calculation

4.4.1 Chemical/Biological Analyses

The samples from the first six storms were analyzed by the University of Nebraska-Lincoln Water Sciences Laboratory, and the samples from the remaining six storms were analyzed by Midwest Laboratories. The specific analyte lists from these two labs are slightly different, but the primary analytes of interest were quantified by both labs. A complete list of the analytes and the methods used at each laboratory can be found in Appendix C.

4.4.2 Sample Compositing Method

There are generally two potential methods of sample composting, flow-weighted or time-weighted. In flow-weighted compositing, the time interval between samples is held constant, and the volume of each aliquot in the composited sample is proportional to the flow at the time each sample was taken. In time-weighted sampling, the sampler must monitor the flow through time, calculating the runoff volume at the sampling point during each sampling period. Then, the sampler would collect samples representing equal volumes (i.e., after a preset amount of runoff passes the sampling point).

For this project, the flow-weighted compositing process was used. Samples were taken approximately every 30 min. Each sample was assumed to be representative of the time period between half-way to the previous sample and half-way to the next sample. Flow rate data from the HEC-HMS model and time intervals were used to calculate the volume of stormwater for each sampling period. A volume fraction from each sample (i.e., aliquot) was taken such that the aliquot's fraction of the total composite sample was equivalent to that sample's fraction of the total storm volume. That is, if a sample represented 10% of the total storm volume, its aliquot would make up 10% of the total composite sample volume. Figure 4.6 shows a typical hydrograph used to determine the aliquot for each of the samples taken during this sample storm.

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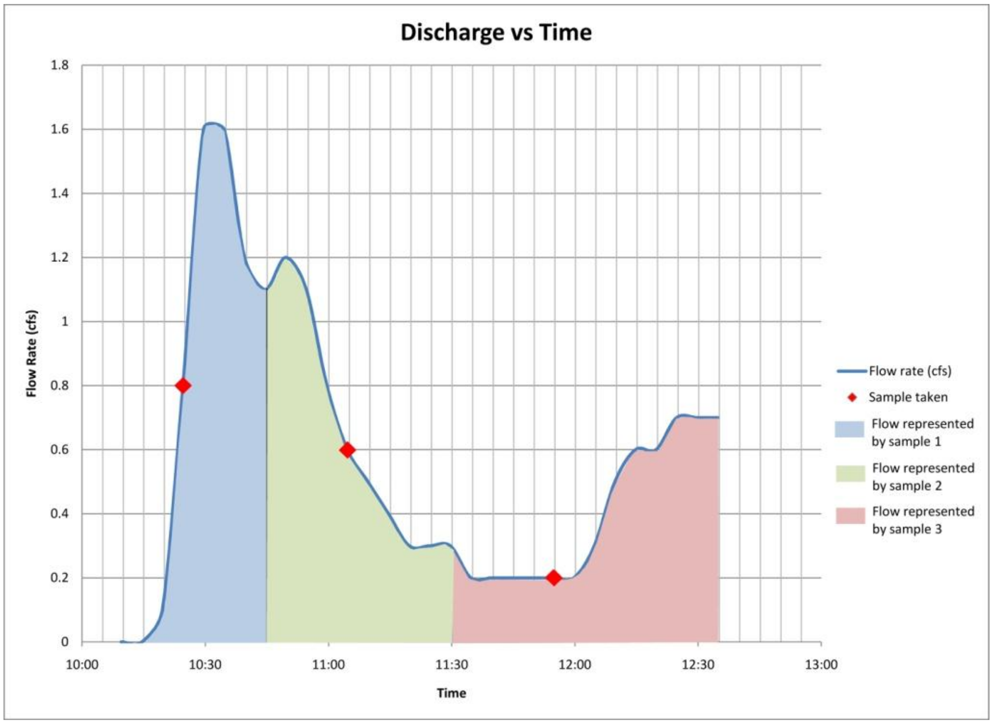

Figure 4.6 Typical hydrograph used in the compositing process

4.5 Pollutant Load Calculations

4.5.1 Event Pollutant Load Calculations

To determine the event pollutant loads, it is necessary to use the area of each subbasin, the volume of runoff, and the EMC from each individual storm. The volume of runoff can be obtained from the HEC-HMS hydrologic model. Equation 4.2 was used to calculate the pollutant loads.

$$L = C \times V \tag{4.2}$$

Where:

L = event pollutant load (Kg/event)

C = event mean concentration (Kg/m^3)

V = volume of runoff (m^3/event)

4.5.2 Annual Pollutant Load Calculations

To determine the annual pollutant loads, total precipitation for all rainfall events for 2009 and 2010 were collected from the weather station located at the Millard Airport, (located at 2.6 miles southwest from the study site) (Weather Underground, 2010). The volumes of runoff for the not-sampled rainfall events were obtained from a rating graph (total precipitation vs. volume of runoff) for the east and west outlet pipes for the sampled events. The volume of runoff from each individual storm was multiplied by the annual (2009 and 2010) mean EMC for the east and west outlet pipe; then, the pollutant loads were summed to obtain the annual pollutant load for each location. Finally, an average between the 2009 and 2010 pollutant loads were calculated for the east and west pipe outlets.

4.6 Efficiency of Existing Detention Basin

Loads from the different subbasins were used to evaluate the efficiency of the detention basin. The construction debris lot, the west and the east outlet pipes are the major pollutant contributors. Pollutants from the construction debris lot are not highway related; therefore, loads should be excluded from the total loads at the detention basin outlet. Once the loads from the construction debris lot are subtracted, the adjusted total pollutant loads are compared to the combined pollutant loads from the east and west pipe outlets. If the pollutant loads at the detention basin outlet are significantly smaller, the detention basin would have some effectiveness in reducing pollutant loads. If the pollutant loads are approximately the same, the detention would not be effective in reducing pollutant loads.

Chapter 5 Results

5.1 Highway Runoff Rates

5.1.1 Precipitation

Precipitation was measured using an ISCO 6700 rain gauge. Table 5.1 shows a summary of the 2009 and 2010 rainfall events, duration, antecedent dry days, and total rainfall. Antecedent dry days are defined as the number of days since a precipitation event of at least 0.10 in.

Table 5.1 Summary of rainfall events sampled

Date	Duration (hr)	Antecedent Dry Days	Total Rainfall (in)
November 10, 2008	3.8	1	0.17
May 12, 2009	3	10	0.18
June 1, 2009	2	6	0.03
July 3, 2009	9	7	0.50
July 31, 2009	2.5	10	0.32
September 3, 2009	7	7	5.1
March 27, 2010	3	7	0.14
May 7, 2010	2	4	0.24
May 20, 2010	7	8	0.38
July 4, 2010	2.8	11	1.38
September 13, 2010	1.5	11	0.19
November 12, 2010	2.5	17	0.74

5.2 Highway Runoff Pollutant Concentrations

Analyses to determine the First Flush (FF) and Event Mean Concentrations (EMC) were conducted. The mean and median values for the FF and EMC results for each site across all the storm events are shown in table 5.2 through table 5.6. The complete results can be seen in Appendix B.

Table 5.2 Arithmetic mean and median for first flush and event mean concentrations for site A for all sampled events

Analyte	**Unit**	First Flush		EMC	
		Mean	Median	Mean	Median
Antimony (total)	(mg/L)	0.0007	0.0007	0.001	0.0008
Arsenic (total)	(mg/L)	0.0063	0.006	0.006	0.0055
Beryllium (total)	(mg/L)	0.001	0.001	0.001	0.001
Cadmium (dissolved)	(mg/L)	0.002	0.002	0.004	0.004
Cadmium (total)	(mg/L)	N.D	N.D	N.D	N.D
Calcium (dissolved)	(mg/L)	91.9	58.6	56.1	13.6
Calcium (total)	(mg/L)	65.6	59.6	53.0	59.6
Chromium (dissolved)	(mg/L)	0.02	0.02	0.01	0.01
Chromium (total)	(mg/L)	0.03	0.03	0.02	0.02
Copper (dissolved)	(mg/L)	0.01	0.01	0.01	0.01
Copper (total)	(mg/L)	0.02	0.02	0.01	0.01
Iron (dissolved)	(mg/L)	0.009	0.004	0.042	0.002
Lead (dissolved)	(mg/L)	0.005	0.0017	0.005	0.004
Lead (total)	(mg/L)	0.01	0.01	0.01	0.01
Magnesium (dissolved)	(mg/L)	21.4	7.90	10.1	6.30
Magnesium (total)	(mg/L)	27.2	36.9	21.7	22.3
Mercury (dissolved)	(mg/L)	0.016	0.005	0.005	0.004
Mercury (total)	(mg/L)	N.D	N.D	N.D	N.D
Nickel (dissolved)	(mg/L)	0.013	0.008	0.006	0.007
Nickel (total)	(mg/L)	0.01	0.01	0.01	0.01
Potassium (dissolved)	(mg/L)	32.7	25.4	36.5	21.3
Selenium (total)	(mg/L)	0.0	0.0	0.0	0.00
Silver (total)	(mg/L)	N.D	N.D	N.D	N.D
Sodium (dissolved)	(mg/L)	239.5	252.0	253.4	183.0
Sodium (total)	(mg/L)	258.3	278.0	254.1	295.0
Thallium (total)	(mg/L)	N.D	N.D	N.D	N.D
Zinc (dissolved)	(mg/L)	0.062	0.085	0.055	0.037
Zinc (total)	(mg/L)	0.03	0.03	0.03	0.03
Silica	(mg/L)	19.1	18.9	16.6	14.9

N.D: Non-detected

[a] Total metal concentrations were analyzed for 2010 samples.

[b] Dissolved metal concentrations were analyzed for 2008 and 2009 samples.

Table 5.2 Arithmetic mean and median for first flush and event mean concentrations for site A for all sampled events (cont.)

Analyte	**Unit**	First Flush		EMC	
		Mean	Median	Mean	Median
Bromide	(mg/L)	0.15	0.10	0.21	0.10
Chloride	(mg/L)	315.3	247.0	340.7	328.0
Fluoride	(mg/L)	0.15	0.12	0.17	0.14
Nitrate	(mg/L)	1.10	1.29	1.43	1.36
Nitrite	(mg/L)	0.10	0.10	0.10	0.10
Nitrate/Nitrite Nitrogen	(mg/L)	1.30	1.30	0.90	1.00
Phosphate	(mg/L)	0.10	0.10	0.10	0.10
Sulfate	(mg/L)	142.9	101.2	155.1	170.2
Soluble Phosphate	(mg P/L)	0.19	0.02	0.12	0.03
Phosphorus (dissolved ortho)	(mg/L)	N.D	N.D	0.06	0.06
Total Phosphorus	(mg P/L)	0.17	0.10	0.17	0.15
Total Kjeldahl Nitrogen	(mg/L)	1.81	1.58	1.60	1.61
Total Dissolved Solids	(mg/L)	1030.7	1178.0	1012.5	1212.0
Total Suspended Solids	(mg/L)	748.3	377.0	393.9	223.5
Total Solids	(mg/L)	2544.4	2506.0	1608.8	1422.0
Volatile Dissolved Solids	(mg/L)	160.0	196.0	155.4	110.0
Volatile Suspended Solids	(mg/L)	144.4	136.0	69.4	54.0
Total Volatile Solids	(mg/L)	304.4	338.0	224.8	264.0
Alkalinity as $CaCO_3$	(mg/L)	142.2	137.1	116.5	142.2
Oil and Grease	(mg/L)	25.9	25.9	28.5	28.5
TEH as Diesel	(µg/L)	208.3	155.0	170.0	120.0
BOD	(mg/L)	8.3	7.0	7.5	8.0
COD	(mg/L)	109.5	101.2	97.3	83.6
n-Hexane	(µg/L)	N.D	N.D	N.D	N.D
Methyl t-Butyl Ether	(µg/L)	N.D	N.D	N.D	N.D
Benzene	(µg/L)	N.D	N.D	N.D	N.D
Toluene	(µg/L)	N.D	N.D	N.D	N.D
Ethylbenzene	(µg/L)	N.D	N.D	N.D	N.D
Napthalene	(µg/L)	0.114	0.11	0.1	0.1

N.D: Non-detected

Table 5.3 Arithmetic mean and median for first flush and event mean concentrations for site B for all sampled events

Analyte	**Unit**	First Flush		EMC	
		Mean	Median	Mean	Median
Antimony (total)	(mg/L)	0.0010	0.0010	0.0008	0.00075
Beryllium	(mg/L)	N.D	N.D	0.001	0.001
Arsenic	(mg/L)	0.002	0.002	0.0032	0.003
Lead	(mg/L)	0.0032	0.0032	0.0069	0.003
Copper	(mg/L)	0.01	0.010	0.0082	0.01
Chromium	(mg/L)	N.D	N.D	0.01	0.01
Zinc	(mg/L)	0.06	0.060	0.0617	0.04
Phosphorus (dissolved ortho)	(mg/L)	0.93	0.93	0.235	0.245
Phosphorus (Total)	(mg/L)	NA	NA	0.34	0.34
Nitrate/Nitrite Nitrogen	(mg/L)	0.7	0.70	0.55	0.45
Total Kjeldahl Nitrogen	(mg/L)	2.75	2.75	1.693	1.62
Magnesium(Total)	(mg/L)	2.09	2.0900	2.685	2.585
Bichemical Oxygen Demand	(mg/L)	N.D	N.D	6.33	6.6
Calcium (Total)	(mg/L)	11.4	11.4	20.683	17.0
Total Suspended Solids	(mg/L)	31.0	31.0	38.166	42.0
Chemical Oxygen Demand	(mg/L)	68.0	68.0	57.0	56.0
Sodium (total)	(mg/L)	54.0	54.0	79.75	84.45
Chloride	(mg/L)	38.0	38.0	95.8	97.5
TEH as Diesel	(µg/L)	172.0	172.0	119.5	119.5
Total Dissolved Solids	(mg/L)	288.0	288.0	394.0	396.0
2 - Methylphenol	(µg/L)	N.D	N.D	N.D	N.D
4 - Methylephenol	(µg/L)	N.D	N.D	N.D	N.D
1,2,4-Trichlorobenzene	(µg/L)	N.D	N.D	N.D	N.D
1,2-Dichlorobenzene	(µg/L)	N.D	N.D	N.D	N.D
1,2-Diphenylhydrazine	(µg/L)	NA	NA	N.D	N.D
1,3-Dichlorobenzene	(µg/L)	N.D	N.D	N.D	N.D
1,4-Dichlorobenzene	(µg/L)	N.D	N.D	N.D	N.D
2- methylnaphthalene	(µg/L)	N.D	N.D	N.D	N.D
2- Nitroaniline	(µg/L)	N.D	N.D	N.D	N.D
2,4 - Dichlorophenol	(µg/L)	N.D	N.D	N.D	N.D
2,4,5 -Trichlorophenol	(µg/L)	N.D	N.D	N.D	N.D
2,4,6 -Trichlorophenol	(µg/L)	N.D	N.D	N.D	N.D
2,4-Dichlorophenol	(µg/L)	N.D	N.D	N.D	N.D
2,4-Dinitrophenol	(µg/L)	N.D	N.D	N.D	N.D
2,4-Dinitrotoluene	(µg/L)	N.D	N.D	N.D	N.D
2,6-Dinitrotoluene	(µg/L)	N.D	N.D	N.D	N.D

N.D: Non-detected

Table 5.3 Arithmetic mean and median for first flush and event mean concentrations for site B for all sampled events (cont.)

Analyte	Unit	First Flush		EMC	
		Mean	Median	Mean	Median
2-Chloronapthalene	(µg/L)	N.D	N.D	N.D	N.D
2-Chlorophenol	(µg/L)	N.D	N.D	N.D	N.D
2-Nitrophenol	(µg/L)	N.D	N.D	N.D	N.D
3 - Nitroaniline	(µg/L)	N.D	N.D	N.D	N.D
3,3'-Dicholorobenzidine	(µg/L)	N.D	N.D	N.D	N.D
4 - Bromophenyl Phenyl Ether	(µg/L)	N.D	N.D	N.D	N.D
4- Chloroaniline	(µg/L)	N.D	N.D	N.D	N.D
4- Nitroaniline	(µg/L)	N.D	N.D	N.D	N.D
4,6 Dintro -2- methylphnol	(µg/L)	N.D	N.D	N.D	N.D
4-Chloro-3-methylphenol	(µg/L)	N.D	N.D	N.D	N.D
4-Chlorophenyl Phenyl Ether	(µg/L)	N.D	N.D	N.D	N.D
4-Nitrophenol	(µg/L)	N.D	N.D	N.D	N.D
Acenaphthene	(µg/L)	N.D	N.D	N.D	N.D
Acenaphthylene	(µg/L)	N.D	N.D	N.D	N.D
Anthracene	(µg/L)	N.D	N.D	N.D	N.D
Benzene	(µg/L)	N.D	N.D	N.D	N.D
Benzo (g,h,i)Perylene	(µg/L)	N.D	N.D	N.D	N.D
Benzo (a) Anthracene	(µg/L)	N.D	N.D	N.D	N.D
Benzo (a) Pyrene	(µg/L)	N.D	N.D	N.D	N.D
Benzo (b) Fluoranthene	(µg/L)	N.D	N.D	N.D	N.D
Benzo (k) Fluoranthene	(µg/L)	N.D	N.D	N.D	N.D
bis (2- Chloroethoxy) Methane	(µg/L)	N.D	N.D	N.D	N.D
Bis (2-ethylhexyl) Phthalate	(µg/L)	N.D	N.D	N.D	N.D
bis(2-Chloroethyl) Ether	(µg/L)	N.D	N.D	N.D	N.D
bis(2-Chloroisopropyl) Ether	(µg/L)	N.D	N.D	N.D	N.D
Butyl Benzyl Phthalate	(µg/L)	N.D	N.D	N.D	N.D
Cadmium	(mg/L)	N.D	N.D	N.D	N.D
Chrysene	(µg/L)	N.D	N.D	N.D	N.D
Crabazole	(µg/L)	N.D	N.D	N.D	N.D
Di -n-octyl Phthalate	(µg/L)	N.D	N.D	N.D	N.D
Dibenz (a,h) Anthracene	(µg/L)	N.D	N.D	N.D	N.D
Dibenzofarun	(µg/L)	N.D	N.D	N.D	N.D
Diethyl Phthalate	(µg/L)	N.D	N.D	N.D	N.D
Dimethyl Phtalate	(µg/L)	N.D	N.D	N.D	N.D

N.D: Non-detected

Table 5.3 Arithmetic mean and median for first flush and event mean concentrations for site B for all sampled events (cont.)

Analyte	Unit	First Flush		EMC	
		Mean	Median	Mean	Median
Di-n-butyl Phthalate	(µg/L)	N.D	N.D	N.D	N.D
Ethylbenzene	(µg/L)	N.D	N.D	N.D	N.D
Fluoranthene	(µg/L)	N.D	N.D	N.D	N.D
Fluorene	(µg/L)	N.D	N.D	N.D	N.D
Hexachlorobenzene	(µg/L)	N.D	N.D	N.D	N.D
Hexachlorobutadiene	(µg/L)	N.D	N.D	N.D	N.D
Hexachlorocyclopentadiene	(µg/L)	N.D	N.D	N.D	N.D
Hexachlorocyclopentadiene	(µg/L)	NA	NA	N.D	N.D
Hexachloroethane	(µg/L)	N.D	N.D	N.D	N.D
Indeno (1,2,3,-cd) Pyrene	(µg/L)	N.D	N.D	N.D	N.D
Isophorone	(µg/L)	N.D	N.D	N.D	N.D
Mercury	(mg/L)	N.D	N.D	N.D	N.D
Methyl t-Butyl Ether	(µg/L)	N.D	N.D	N.D	N.D
Naphtalene	(µg/L)	N.D	N.D	N.D	N.D
Napthalene	(µg/L)	N.D	N.D	N.D	N.D
n-Hexane	(µg/L)	N.D	N.D	N.D	N.D
Nickel	(mg/L)	N.D	N.D	N.D	N.D
Nitrobenzene	(µg/L)	N.D	N.D	N.D	N.D
N-Nitroso-di-n-propylamine	(µg/L)	N.D	N.D	N.D	N.D
N-Nitrosodiphenylamine	(µg/L)	N.D	N.D	N.D	N.D
Pentacholorophenol	(µg/L)	N.D	N.D	N.D	N.D
Phenanthrene	(µg/L)	N.D	N.D	N.D	N.D
Phenol	(µg/L)	N.D	N.D	N.D	N.D
Pyrene	(µg/L)	N.D	N.D	N.D	N.D
Selenium	(mg/L)	N.D	N.D	N.D	N.D
Silver	(mg/L)	N.D	N.D	N.D	N.D
TEH as Gasoline	(µg/L)	N.D	N.D	N.D	N.D
TEH as Waste Oil	(µg/L)	N.D	N.D	N.D	N.D
Thallium	(mg/L)	N.D	N.D	N.D	N.D
Toluene	(µg/L)	N.D	N.D	N.D	N.D
Total Purgeable Hydrocarbons	(µg/L)	N.D	N.D	N.D	N.D
Total Xylenes	(µg/L)	N.D	N.D	N.D	N.D

N.D: Non-detected

Table 5.4 Arithmetic mean and median for first flush and event mean concentrations for site C for all sampled events

Analyte	Unit	First Flush		EMC	
		Mean	Median	Mean	Median
Antimony (total)	(mg/L)	0.0053	0.0031	0.0051	0.0037
Arsenic (total)	(mg/L)	0.005	0.002	0.005	0.005
Beryllium (total)	(mg/L)	0.001	0.001	0.001	0.001
Cadmium (dissolved)	(mg/L)	0.004	0.004	0.003	0.003
Cadmium (total)	(mg/L)	0.004	0.004	0.0015	0.0015
Calcium (dissolved)	(mg/L)	23	21.4	14.4	15.4
Calcium (total)	(mg/L)	82.4	56.6	77.4	57.7
Chromium (dissolved)	(mg/L)	0.02	0.005	0.015	0.006
Chromium (total)	(mg/L)	0.05	0.03	0.04	0.03
Copper (dissolved)	(mg/L)	0.04	0.02	0.02	0.01
Copper (total)	(mg/L)	0.100	0.060	0.048	0.030
Iron (dissolved)	(mg/L)	0.3	0.03	0.05	0.04
Lead (dissolved)	(mg/L)	0.01	0.001	0.008	0.004
Lead (total)	(mg/L)	0.039	0.018	0.019	0.007
Magnesium (dissolved)	(mg/L)	1.2	0.8	0.7	0.4
Magnesium (total)	(mg/L)	8.77	4.00	8.52	9.1
Mercury (dissolved)	(mg/L)	0.004	0.004	0.004	0.004
Mercury (total)	(mg/L)	N.D	N.D	N.D	N.D
Nickel (dissolved)	(mg/L)	0.012	0.006	0.008	0.008
Nickel (total)	(mg/L)	0.03	0.03	0.01	0.01
Potassium (dissolved)	(mg/L)	4.3	2.5	1.133	1.4
Selenium (total)	(mg/L)	0.001	0.001	0.001	0.001
Silver (total)	(mg/L)	N.D	N.D	N.D	N.D
Sodium (dissolved)	(mg/L)	53.5	61.8	39.5	18.4
Sodium (total)	(mg/L)	120.8	159.0	290.4	103.0
Thallium (total)	(mg/L)	N.D	N.D	N.D	N.D
Zinc (dissolved)	(mg/L)	0.04	0.02	0.05	0.03
Zinc (total)	(mg/L)	0.057	0.029	0.027	0.012
Silica	(mg/L)	4.22	2.51	3.20	2.60

N.D: Non-detected

[a] Total metal concentrations were analyzed for 2010 samples.

[b] Dissolved metal concentrations were analyzed for 2008 and 2009 samples.

Table 5.4 Arithmetic mean and median for first flush and event mean concentrations for site C for all sampled events (cont.)

Analyte	**Unit**	First Flush		EMC	
		Mean	Median	Mean	Median
Bromide	(mg/L)	0.10	0.10	0.10	0.10
Chloride	(mg/L)	141	124	207	93
Fluoride	(mg/L)	0.460	0.430	0.342	0.275
Nitrate	(mg/L)	1.00	1.19	0.88	0.90
Nitrite	(mg/L)	0.680	0.000	0.020	0.000
Nitrate/Nitrite Nitrogen	(mg/L)	1.33	0.40	0.90	0.50
Phosphate	(mg/L)	0.64	0.49	0.43	0.35
Sulfate	(mg/L)	47.4	43.7	29.5	18.4
Soluble Phosphate	(mg P/L)	0.150	0.143	0.230	0.120
Phosphorus (dissolved ortho)	(mg/L)	0.107	0.100	0.095	0.090
Total Phosphorus	(mg P/L)	0.82	0.60	0.39	0.27
Total Kjeldahl Nitrogen	(mg/L)	5.02	5.57	2.53	1.64
Total Dissolved Solids	(mg/L)	207.0	106.0	514.3	122.0
Total Suspended Solids	(mg/L)	520.9	321.0	258.6	116.0
Total Solids	(mg/L)	637.3	584.0	379.0	223.0
Volatile Dissolved Solids	(mg/L)	83.3	83.5	37.8	23.0
Volatile Suspended Solids	(mg/L)	117.5	72.0	101.2	52.5
Total Volatile Solids	(mg/L)	157.7	139.0	116.0	94.0
Alkalinity as $CaCO_3$	(mg/L)	80.0	77.9	51.4	51.5
Oil and Grease	(mg/L)	10.4	10.4	14.0	14.0
TEH as Diesel	(µg/L)	963.7	880.0	411.3	455.0
BOD	(mg/L)	32.3	28.0	14.3	12.5
COD	(mg/L)	120.1	82.4	95.6	47.0
n-Hexane	(µg/L)	N.D	N.D	N.D	N.D
Methyl t-Butyl Ether	(µg/L)	N.D	N.D	N.D	N.D
Benzene	(µg/L)	N.D	N.D	N.D	N.D
Toluene	(µg/L)	N.D	N.D	N.D	N.D
Ethylbenzene	(µg/L)	N.D	N.D	N.D	N.D
Napthalene	(µg/L)	0.09	0.08	0.0833	0.08

N.D: Non-detected

Table 5.5 Arithmetic mean and median for first flush and event mean concentrations for site D for all sampled events

Analyte	**Unit**	First Flush		EMC	
		Mean	Median	Mean	Median
Antimony (total)	(mg/L)	0.005	0.005	0.004	0.003
Arsenic (total)	(mg/L)	0.005	0.005	0.003	0.002
Beryllium (total)	(mg/L)	0.0010	0.0010	0.0010	0.0010
Cadmium (dissolved)	(mg/L)	0.0028	0.0028	0.0028	0.0028
Cadmium (total)	(mg/L)	0.003	0.003	0.0015	0.0015
Calcium (dissolved)	(mg/L)	23.2	21.4	14.2	13.4
Calcium (total)	(mg/L)	65.6	93.7	47.7	46.5
Chromium (dissolved)	(mg/L)	0.013	0.004	0.011	0.005
Chromium (total)	(mg/L)	0.035	0.035	0.027	0.020
Copper (dissolved)	(mg/L)	0.04	0.02	0.026	0.017
Copper (total)	(mg/L)	0.05	0.04	0.03	0.02
Iron (dissolved)	(mg/L)	0.0046	0.0037	0.0045	0.003
Lead (dissolved)	(mg/L)	0.008	0.002	0.006	0.003
Lead (total)	(mg/L)	0.023	0.014	0.014	0.006
Magnesium (dissolved)	(mg/L)	1.28	1.3	0.82	1.00
Magnesium (total)	(mg/L)	5.27	4.03	3.60	2.16
Mercury (dissolved)	(mg/L)	0.0034	0.0030	0.0033	0.0030
Mercury (total)	(mg/L)	N.D	N.D	N.D	N.D
Nickel (dissolved)	(mg/L)	0.011	0.003	0.007	0.002
Nickel (total)	(mg/L)	0.03	0.03	0.009	0.009
Potassium (dissolved)	(mg/L)	3.6	3.2	1.9	1.6
Selenium (total)	(mg/L)	0.0025	0.0025	N.D	N.D
Silver (total)	(mg/L)	N.D	N.D	N.D	N.D
Sodium (dissolved)	(mg/L)	79.9	75.7	56.5	41.5
Sodium (total)	(mg/L)	172.0	106.0	166.7	128.2
Thallium (total)	(mg/L)	N.D	N.D	N.D	N.D
Zinc (dissolved)	(mg/L)	0.011	0.008	0.009	0.011
Zinc (total)	(mg/L)	0.041	0.030	0.024	0.012
Silica	(mg/L)	4.57	2.09	3.40	2.30

N.D: Non-detected

[a] Total metal concentrations were analyzed for 2010 samples.

[b] Dissolved metal concentrations were analyzed for 2008 and 2009 samples.

Table 5.5 Arithmetic mean and median for first flush and event mean concentrations for site D for all sampled events (cont.)

Analyte	Unit	First Flush		EMC	
		Mean	Median	Mean	Median
Bromide	(mg/L)	N.D	N.D	N.D	N.D
Chloride	(mg/L)	234.5	136.3	139	66
Fluoride	(mg/L)	0.44	0.44	0.29	0.24
Nitrate	(mg/L)	0.4	0.4	0.4	0.4
Nitrite	(mg/L)	1.20	1.20	N.D	N.D
Nitrate/Nitrite Nitrogen	(mg/L)	1.10	0.50	0.97	0.70
Phosphate	(mg/L)	0.42	0.35	0.29	0.23
Sulfate	(mg/L)	30.1	27.3	24.0	9.9
Soluble Phosphate	(mg P/L)	0.090	0.075	0.153	0.086
Phosphorus (dissolved ortho)	(mg/L)	0.107	0.100	0.095	0.090
Total Phosphorus	(mg P/L)	0.28	0.25	0.22	0.18
Total Kjeldahl Nitrogen	(mg/L)	2.63	2.63	1.76	1.18
Total Dissolved Solids	(mg/L)	497.8	328.0	336.6	157.0
Total Suspended Solids	(mg/L)	231.0	197.5	142.5	92.0
Total Solids	(mg/L)	529.0	560.0	432.7	370.0
Volatile Dissolved Solids	(mg/L)	47.2	48.0	60.0	37.0
Volatile Suspended Solids	(mg/L)	68.0	63.0	72.7	66.0
Total Volatile Solids	(mg/L)	103.7	106.0	109.0	101.0
Alkalinity as $CaCO_3$	(mg/L)	58.1	55.8	44.0	39.4
Oil and Grease	(mg/L)	9.7	9.7	9.6	9.6
TEH as Diesel	(µg/L)	610.7	260.0	401.0	420.0
BOD	(mg/L)	19.7	18.0	13.3	11.5
COD	(mg/L)	72.7	29.2	53.5	29.0
n-Hexane	(µg/L)	N.D	N.D	N.D	N.D
Methyl t-Butyl Ether	(µg/L)	N.D	N.D	N.D	N.D
Benzene	(µg/L)	N.D	N.D	N.D	N.D
Toluene	(µg/L)	N.D	N.D	N.D	N.D
Ethylbenzene	(µg/L)	N.D	N.D	N.D	N.D
Napthalene	(µg/L)	N.D	N.D	N.D	N.D

N.D: Non-detected

Table 5.6 Arithmetic mean and median for first flush and event mean concentrations for site E for all sampled events

Analyte	Unit	First Flush		EMC	
		Mean	Median	Mean	Median
Antimony (total)[a]	(mg/L)	0.002	0.002	0.003	0.003
Arsenic (total)[a]	(mg/L)	0.0035	0.0035	0.0028	0.003
Beryllium (total)[a]	(mg/L)	N.D	N.D	N.D	N.D
Cadmium (dissolved)[b]	(mg/L)	0.0010	0.0010	0.0023	0.0023
Cadmium (total)[a]	(mg/L)	N.D	N.D	N.D	N.D
Calcium (dissolved)[b]	(mg/L)	41.4	22.8	29.2	21.7
Calcium (total)[a]	(mg/L)	70.4	85.7	41.4	32.8
Chromium (dissolved)[b]	(mg/L)	0.008	0.01	0.009	0.008
Chromium (total)[a]	(mg/L)	0.020	0.020	0.018	0.020
Copper (dissolved)[b]	(mg/L)	0.01	0.01	0.01	0.01
Copper (total)[a]	(mg/L)	0.02	0.02	0.014	0.03
Iron (dissolved)[b]	(mg/L)	0.04	0.01	0.03	0.01
Lead (dissolved)[b]	(mg/L)	0.04	0.003	0.04	0.005
Lead (total)[a]	(mg/L)	0.007	0.005	0.003	0.007
Magnesium (dissolved)[b]	(mg/L)	5.14	2.60	4.50	3.25
Magnesium (total)[a]	(mg/L)	8.83	7.84	6.56	4.27
Mercury (dissolved)[b]	(mg/L)	0.0032	0.0027	0.0031	0.0027
Mercury (total)[a]	(mg/L)	N.D	N.D	N.D	N.D
Nickel (dissolved)[b]	(mg/L)	0.010	0.010	N.D	N.D
Nickel (total)[a]	(mg/L)	0.00	0.00	0.00	0.00
Potassium (dissolved)[b]	(mg/L)	7.62	5.00	8.7	8.2
Selenium (total)[a]	(mg/L)	0.0020	0.0020	0.0300	0.0300
Silver (total)[a]	(mg/L)	N.D	N.D	N.D	N.D
Sodium (dissolved)[b]	(mg/L)	207	107	133	107
Sodium (total)[a]	(mg/L)	343.9	380.0	171.6	128.5
Thallium (total)[a]	(mg/L)	N.D	N.D	N.D	N.D
Zinc (dissolved)[b]	(mg/L)	52.3	10.0	31.3	2.3
Zinc (total)[a]	(mg/L)	0.11	0.07	0.077	0.08
Silica	(mg/L)	7.25	2.34	6.28	4.70

N.D: Non-detected

[a] Total metal concentrations were analyzed for 2010 samples.

[b] Dissolved metal concentrations were analyzed for 2008 and 2009 samples.

Table 5.6 Arithmetic mean and median for first flush and event mean concentrations for site E for all sampled events (cont.)

Analyte	Unit	First Flush		EMC	
		Mean	Median	Mean	Median
Bromide	(mg/L)	N.D	N.D	N.D	N.D
Chloride	(mg/L)	246	196	207	149
Fluoride	(mg/L)	0.39	0.38	0.26	0.22
Nitrate	(mg/L)	0.6	0.7	0.6	0.6
Nitrite	(mg/L)	0.37	0.37	0.37	0.37
Nitrate/Nitrite Nitrogen	(mg/L)	0.73	0.40	0.78	0.50
Phosphate	(mg/L)	0.32	0.38	0.26	0.23
Sulfate	(mg/L)	55.5	36.6	55.2	40.6
Soluble Phosphate	(mg P/L)	0.187	0.069	0.066	0.060
Phosphorus (dissolved ortho)	(mg/L)	0.055	0.055	0.105	0.105
Total Phosphorus	(mg P/L)	0.17	0.15	0.18	0.16
Total Kjeldahl Nitrogen	(mg/L)	1.79	1.77	1.50	1.19
Total Dissolved Solids	(mg/L)	717.8	537.0	516.7	470.0
Total Suspended Solids	(mg/L)	239.4	172.5	248.6	190.0
Total Solids	(mg/L)	871.7	785.0	843.0	825.0
Volatile Dissolved Solids	(mg/L)	59.8	45.0	98.7	52.0
Volatile Suspended Solids	(mg/L)	92.2	84.0	69.2	55.0
Total Volatile Solids	(mg/L)	128.0	97.0	117.0	91.0
Alkalinity as $CaCO_3$	(mg/L)	83.9	88.3	84.3	68.8
Oil and Grease	(mg/L)	11.5	11.5	8.8	8.8
TEH as Diesel	(μg/L)	310.3	160	407.8	275.0
BOD	(mg/L)	11.0	11.0	13.8	11.0
COD	(mg/L)	66.8	49.6	69.5	52.3
n-Hexane	(μg/L)	N.D	N.D	N.D	N.D
Methyl t-Butyl Ether	(μg/L)	N.D	N.D	N.D	N.D
Benzene	(μg/L)	0.06	0.06	0.06	0.06
Toluene	(μg/L)	0.11	0.1	0.08	0.08
Ethylbenzene	(μg/L)	N.D	N.D	N.D	N.D
Napthalene	(μg/L)	0.105	0.105	0.08	0.08

N.D: Non-detected

5.3 Pollutant loads

The mean and median values for the pollutant loads from the east, west, and basin outlet pipes are shown in table 5.7 and table 5.8. Table 5.7 included the pollutant loads for the September 3, 2009 rainfall event. This event was the largest rainfall event sampled in this study; therefore, runoff from subbasin 2 (ditch), subbasin 4 (grass strip), and subbasin 6 (detention basin) contributed to the pollutant loads at the detention basin outlet. Table 5.8 excluded the pollutant loads from this rainfall event. The average annual loads for the east, west, basin outlet pipes are shown in table 5.9. The complete results can be seen in Appendix D.

Table 5.7 Arithmetic mean and median for pollutant loads for the west, east, and basin outlet pipes for all sampled events

Analyte	Load (Kg/event)					
	West Outlet Pipe		East Outlet Pipe		Basin Outlet Pipe	
	Mean	Median	Mean	Median	Mean	Median
Antimony (total)[a]	3.12E-05	3.24E-05	2.74E-05	3.24E-05	1.25E-04	7.76E-05
Arsenic (total)[a]	3.81E-05	3.81E-05	1.68E-05	1.36E-05	5.62E-04	1.03E-04
Beryllium (total)[a]	N.D	N.D	N.D	N.D	N.D	N.D
Cadmium (dissolved)[b]	1.62E-05	1.62E-05	2.03E-05	2.03E-05	7.66E-05	7.66E-05
Cadmium (total)[a]	1.84E-05	1.84E-05	1.84E-05	1.84E-05	N.D	N.D
Calcium (dissolved)[b]	1.27	0.306	1.22	0.251	18.3	1.77
Calcium (total)[a]	0.675	0.241	0.35	0.193	5.53	1.72
Chromium (dissolved)[b]	2.07E-04	1.17E-04	2.78E-04	1.21E-04	0.00308	0.00065
Chromium (total)[a]	2.59E-04	3.67E-04	1.73E-04	2.45E-04	0.00280	0.00176
Copper (dissolved)[b]	4.71E-04	2.43E-04	4.84E-04	2.13E-04	0.00370	0.00114
Copper (total)[a]	2.88E-04	2.98E-04	2.28E-04	2.11E-04	0.00293	0.00155
Iron (dissolved)[b]	0.00134	0.0013	0.00671	0.0015	0.017	0.0011
Lead (dissolved)[b]	6.98E-05	2.89E-05	6.66E-05	3.58E-05	7.16E-04	7.66E-05
Lead (total)[a]	1.02E-04	4.76E-05	7.48E-05	4.46E-05	0.00159	4.08E-04
Magnesium (dissolved)[b]	0.0481	0.0112	0.106	0.0122	4.48	0.242
Magnesium (total)[a]	0.033	0.0156	0.0203	0.0123	0.825	0.135
Mercury (dissolved)[b]	N.D	N.D	N.D	N.D	N.D	N.D
Mercury (total)[a]	N.D	N.D	N.D	N.D	N.D	N.D
Nickel (dissolved)[b]	1.22E-04	1.03E-04	1.48E-04	1.22E-04	0.0016	0.000240
Nickel (total)[a]	1.84E-04	1.84E-04	1.22E-04	1.22E-04	0.0016	0.000240
Potassium (dissolved)[b]	0.153	0.0298	0.147	0.0276	14.8	0.675
Selenium (total)[a]	1.36E-05	1.36E-05	N.D	N.D	0.00228	0.00228
Silver (total)[a]	N.D	N.D	N.D	N.D	N.D	N.D
Sodium (dissolved)[b]	1.62	0.64	2.39	1.04	94.1	7.54
Sodium (total)[a]	2.85	0.265	1.35	0.228	14.0	5.5
Thallium (total)[a]	N.D	N.D	N.D	N.D	N.D	N.D
Zinc (dissolved)[b]	0.00159	1.02E-03	0.001374	6.84E-04	0.00714	0.00256
Zinc (total)[a]	0.00157	1.04E-03	0.00143	1.19E-03	0.00915	0.00465
Silica	0.0419	0.0529	0.0531	0.0427	0.347	0.278
Bromide	0.0476	0.0476	N.D	N.D	N.D	N.D
Chloride	3.35	0.417	2.46	1.13	74.0	10.6
Fluoride	0.0193	0.00651	0.0164	0.00727	0.116	0.0203
Nitrate	0.0606	0.0180	0.0313	0.00858	0.590	0.0422
Nitrite	N.D	N.D	N.D	N.D	N.D	N.D
Nitrate/Nitrite Nitrogen	0.01014	0.00306	0.01018	0.00809	0.099	0.075
Phosphate	0.0416	0.0104	0.0323	0.00892	0.254	0.0351
Sulfate	0.856	0.338	0.907	0.329	43.4	3.21
Soluble Phosphate	0.0125	0.0026	0.0109	0.0034	0.034	0.00717
Phosphorus (dissolved ortho)	6.83E-04	5.51E-04	6.34E-04	7.22E-04	0.0515	0.0041
Total Phosphorus	0.00829	0.00356	0.00758	0.00350	0.0471	0.0107
Total Kjeldahl Nitrogen	0.103	0.02	0.0712	0.0150	0.603	0.0731
Total Dissolved Solids	12.5	2.50	5.46	3.06	274	19.2
Total Suspended Solids	7.04	1.28	4.70	1.21	184	7.20
Total Solids	26.1	4.19	17.5	7.16	721	32.1
Volatile Dissolved Solids	2.43	0.82	2.33	0.84	103	39.9
Volatile Suspended	11.4	0.61	6.58	1.45	17.8	3.94

Analyte	Load (Kg/event)					
	West Outlet Pipe		East Outlet Pipe		Basin Outlet Pipe	
	Mean	Median	Mean	Median	Mean	Median
Solids						
Total Volatile Solids	13.4	1.18	8.31	2.30	69.3	5.23
Alkalinity as $CaCO_3$	3.19	0.810	3.43	0.76	61.4	4.98
Oil and Grease	N.D	N.D	N.D	N.D	N.D	N.D
TEH as Diesel	1.21	0.00318	1.10	0.0035	0.0348	0.0522
BOD	0.101	0.0651	0.100	0.07	1.60	0.67
COD	1.13	0.298	0.523	0.41	30.5	2.48
n-Hexane	N.D	N.D	N.D	N.D	N.D	N.D
Methyl t-Butyl Ether	N.D	N.D	N.D	N.D	N.D	N.D
Benzene	N.D	N.D	N.D	N.D	N.D	N.D
Toluene	N.D	N.D	N.D	N.D	N.D	N.D
Ethylbenzene	N.D	N.D	N.D	N.D	N.D	N.D
Napthalene	N.D	N.D	N.D	N.D	N.D	N.D

N.D: Non-detected

[a] Total metal concentrations were analyzed for 2010 samples.

[b] Dissolved metal concentrations were analyzed for 2008 and 2009 samples.

Table 5.8 Arithmetic mean and median for pollutant loads for the west, east, and basin outlet pipes for all sampled events (excluding 9/03/09 event)

Analyte	Load (Kg/event)					
	West Outlet Pipe		East Outlet Pipe		Basin Outlet Pipe	
	Mean	Median	Mean	Median	Mean	Median
Antimony (total)[a]	3.12E-05	3.24E-05	2.74E-05	3.24E-05	1.25E-04	7.76E-05
Arsenic (total)[a]	3.81E-05	3.81E-05	1.68E-05	1.36E-05	5.62E-04	1.03E-04
Beryllium (total)[a]	N.D	N.D	N.D	N.D	N.D	N.D
Cadmium (dissolved)[b]	1.62E-05	1.62E-05	2.03E-05	2.03E-05	7.66E-05	7.66E-05
Cadmium (total)[a]	1.84E-05	1.84E-05	1.84E-05	1.84E-05	N.D	N.D
Calcium (dissolved)[b]	0.244	0.258	0.193	0.246	1.66	1.77
Calcium (total)[a]	0.675	0.241	0.354	0.193	4.51	1.72
Chromium (dissolved)[b]	1.48E-04	1.16E-04	1.14E-04	9.32E-05	1.55E-03	6.53E-04
Chromium (total)[a]	2.59E-04	3.67E-04	1.73E-04	2.45E-04	0.00280	0.00176
Copper (dissolved)[b]	2.43E-04	2.02E-04	2.23E-04	2.01E-04	0.00163	0.00114
Copper (total)[a]	2.88E-04	2.98E-04	2.28E-04	2.11E-04	0.00237	0.00155
Iron (dissolved)[b]	8.87E-04	1.02E-03	6.71E-03	8.30E-04	0.0202	0.00109
Lead (dissolved)[b]	6.17E-05	2.02E-05	5.81E-05	3.53E-05	6.28E-04	7.66E-05
Lead (total)[a]	1.02E-04	4.76E-05	7.48E-05	4.46E-05	0.001276	4.08E-04
Magnesium (dissolved)[b]	0.0125	0.00973	0.0137	0.0117	0.329	0.242
Magnesium (total)[a]	0.0332	0.0156	0.0203	0.0123	0.687	0.135
Mercury (dissolved)[b]	7.84E-05	5.42E-05	9.63E-05	8.14E-05	4.33E-04	3.34E-04
Mercury (total)[a]	N.D	N.D	N.D	N.D	N.D	N.D
Nickel (dissolved)[b]	8.03E-05	2.24E-05	1.14E-04	7.60E-05	5.06E-04	2.40E-04
Nickel (total)[a]	1.84E-04	1.84E-04	1.22E-04	1.22E-04	#DIV/0!	#NUM!
Potassium (dissolved)[b]	0.0251	0.0260	0.0290	0.0267	0.636	0.675
Selenium (total)[a]	1.36E-05	1.36E-05	N.D	N.D	0.00228	0.00228
Silver (total)[a]	N.D	N.D	N.D	N.D	N.D	N.D
Sodium (dissolved)[b]	0.531	0.506	0.866	0.942	8.24	7.54
Sodium (total)[a]	2.85	0.265	1.35	0.228	11.5	5.53
Thallium (total)[a]	N.D	N.D	N.D	N.D	N.D	N.D
Zinc (dissolved)[b]	0.00159	0.00102	0.00137	6.84E-04	0.00722	0.00455
Zinc (total)[a]	0.00157	0.00104	0.00143	0.00119	0.00744	0.00465
Silica	0.0524	0.0555	0.0664	0.0643	0.521	0.456
Bromide	N.D	N.D	N.D	N.D	N.D	N.D
Chloride	3.00	0.407	1.75	0.786	12.2	10.6
Fluoride	0.00633	0.00603	0.00623	0.00638	0.0214	0.0203
Nitrate	0.0141	0.0134	0.00821	0.00755	0.0416	0.0422
Nitrite	N.D	N.D	N.D	N.D	N.D	N.D
Nitrate/Nitrite Nitrogen	0.0101	0.00306	0.0102	0.00809	0.0821	0.0748
Phosphate	0.00785	0.00773	0.00659	0.00746	0.0308	0.0351
Sulfate	0.295	0.313	0.258	0.277	3.94	3.21
Soluble Phosphate	0.00531	0.00185	0.00399	0.00240	0.00783	0.00717
Phosphorus (dissolved ortho)	6.83E-04	5.51E-04	6.34E-04	7.22E-04	0.0347	0.00413
Total Phosphorus	0.00255	0.00303	0.00277	0.00312	0.0357	0.0107

Analyte	**Load (Kg/event)**					
	West Outlet Pipe		East Outlet Pipe		Basin Outlet Pipe	
	Mean	Median	Mean	Median	Mean	Median
Total Kjeldahl Nitrogen	0.0171	0.0184	0.0167	0.0117	0.162	0.0731
Total Dissolved Solids	6.86	2.17	3.85	2.52	37.9	19.2
Total Suspended Solids	1.58	1.07	0.990	0.759	30.4	7.20
Total Solids	4.02	3.88	7.41	5.94	89.1	32.1
Volatile Dissolved Solids	0.697	0.749	2.33	0.843	21.9	21.9
Volatile Suspended Solids	0.701	0.554	1.08	1.09	3.64	3.94
Total Volatile Solids	1.23	1.12	2.77	1.68	14.6	5.23
Alkalinity as $CaCO_3$	0.726	0.719	0.717	0.681	6.68	4.98
Oil and Grease	N.D	N.D	N.D	N.D	N.D	N.D
TEH as Diesel	1.21	0.00318	1.10	0.00349	1.80	0.0522
BOD	0.101	0.0651	0.0996	0.0683	1.37	0.673
COD	1.13	0.298	0.523	0.409	7.15	2.48
n-Hexane	N.D	N.D	N.D	N.D	N.D	N.D
Methyl t-Butyl Ether	N.D	N.D	N.D	N.D	N.D	N.D
Benzene	N.D	N.D	N.D	N.D	N.D	N.D
Toluene	N.D	N.D	N.D	N.D	N.D	N.D
Ethylbenzene	N.D	N.D	N.D	N.D	N.D	N.D
Napthalene	N.D	N.D	N.D	N.D	N.D	N.D

N.D: Non-detected

[a] Total metal concentrations were analyzed for 2010 samples.

[b] Dissolved metal concentrations were analyzed for 2008 and 2009 samples.

Table 5.9 Annual pollutant loads for the west, east, and basin outlet pipes

Analyte	Annual Load (Kg/yr)		
	West Outlet Pipe	East Outlet Pipe	Total Pollutant Load
Antimony (total)	0.0091	0.21	0.217
Arsenic (total)	0.010	0.17	0.185
Beryllium (total)	0.0020	0.058	0.060
Cadmium (dissolved)	0.0029	0.16	0.168
Cadmium (total)	0.0060	0.17	0.180
Calcium (dissolved)	19.1	50.1	69.1
Calcium (total)	131	2368	2499
Chromium (dissolved)	0.029	0.77	0.796
Chromium (total)	0.074	1.55	1.62
Copper (dissolved)	0.040	0.92	0.964
Copper (total)	0.085	1.89	1.97
Iron (dissolved)	0.070	0.88	0.953
Lead (dissolved)	0.012	0.27	0.280
Lead (total)	0.032	0.67	0.702
Magnesium (dissolved)	0.90	2.89	3.79
Magnesium (total)	8.55	177	186
Mercury (dissolved)	0.0049	0.012	0.0165
Nickel (dissolved)	0.017	0.59	0.605
Nickel (total)	0.060	1.16	1.22
Potassium (dissolved)	2.99	6.69	9.68
Selenium (total)	0.0020	N.D	0.0020
Sodium (dissolved)	52.3	199	251
Sodium (total)	475	7934	8409
Zinc (dissolved)	0.25	5.97	6.21
Zinc (total)	0.47	11.9	12.3
Silica	3.02	8.26	11.3
Chloride	432	6189	6621
Fluoride	0.38	0.87	1.25
Nitrate	0.85	1.60	2.45
Nitrate/Nitrite Nitrogen	1.85	53.8	55.6
Phosphate	0.57	1.03	1.60
Sulfate	41.5	95.1	137
Soluble Phosphate	0.33	0.58	0.906
Phosphorus (dissolved ortho)	0.20	5.09	5.29
Total Phosphorus	0.44	8.58	9.02
Total Kjeldahl Nitrogen	3.82	51.5	55.3
Total Dissolved Solids	1074	12826	13899
Total Suspended Solids	378	4154	4532
Total Solids	578	1191	1769
Volatile Dissolved Solids	27.4	214	241
Volatile Suspended Solids	147	237	384
Total Volatile Solids	147	349	496
Alkalinity as $CaCO_3$	64.9	157	222
Oil and Grease	18.5	34	52.3
TEH as Diesel	0.826	22.96	23.8
BOD	26.6	721	748
COD	168	2341	2509

N.D: Non-detected

Chapter 6 Data Analysis and Discussion of Results

6.1 Correlations Between Pollutant Concentrations and Factors Affecting Highway Runoff

Previous studies on highway runoff pollution found that several factors may have an influence on the pollutant concentrations and loads (Hoffman et al. 1985; Kerri et al. 1985; Dricoll et al. 1991). For this study, concentration of total suspended solids, antecedent dry period (ADP), rainfall amount, and volume of runoff were compared against pollutant concentrations and loads to see if they correlated. Chromium (Cr), copper (Cu), lead (Pb), chloride (Cl^-), and total phosphorus (TP) were selected because these pollutants were present in all the rainfall events sampled.

6.1.1 Pollutant Relationship with Total Suspended Solids

Concentrations and loads for chromium (Cr), copper (Cu), lead (Pb), chloride (Cl^-), and total phosphorus (TP) were compared to the concentration of total suspended solids for each of the sampled events for the west, east, and basin outlet pipes. These comparisons are shown in figures 6.1 through 6.15. From these figures, it is noticeable that most of the samples have a TSS concentration between 0 to 300 mg/L. Results from the literature showed a high correlation between TSS and the runoff pollutants (Driscoll et al. 1990; Sansalone et al. 1995). A poor linear correlation can be observed for the EMC for heavy metals (Cr, Cu, and Pb) and TP concentrations from the West and East outlet pipes. This correlation is not evident for the Basin outlet pipe, or for Cl^- in any of the pipes.

A strong correlation between TSS and heavy metals was expected because these elements tend to adsorb onto suspended solids. This relationship is important because if the metals are adsorbed to suspended solids, treating or removing solids would reduce the concentrations and loads of the pollutants associated with suspended solids. This relationship may not be well observed in this study due to the high variability of TSS concentrations at the east and west pipe outlets.

6.1.2 Pollutant Relationship with Antecedent Dry Period

Concentrations and loads for chromium (Cr), copper (Cu), lead (Pb), chloride (Cl^-), and total phosphorus (TP) were compared to the antecedent dry periods for each of the sampled events for the west, east, and basin outlet pipes. This comparison is shown in figures 6.16 through 6.30. It was expected that concentrations, especially the first flush concentrations, would be higher for longer ADP; however, the figures do not show any type of trend. It seems that concentrations are independent from ADP.

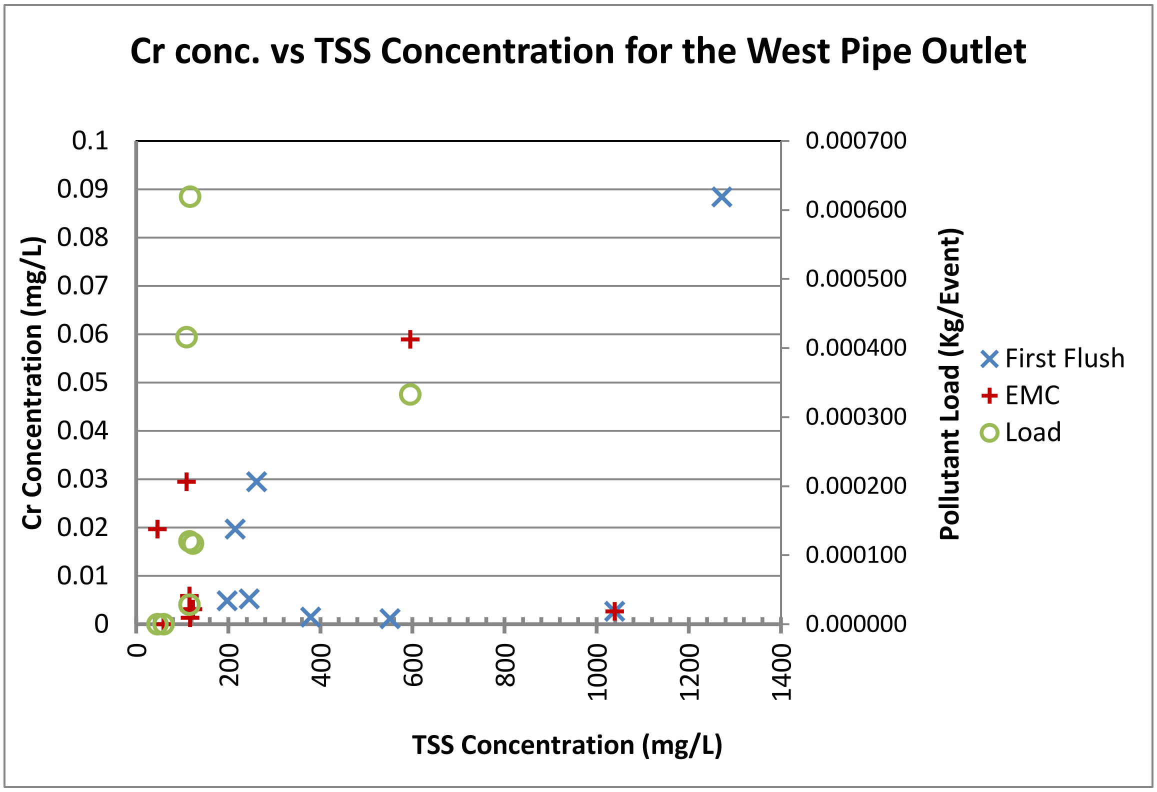

Figure 6.1 Cr and TSS concentrations for the west pipe outlet

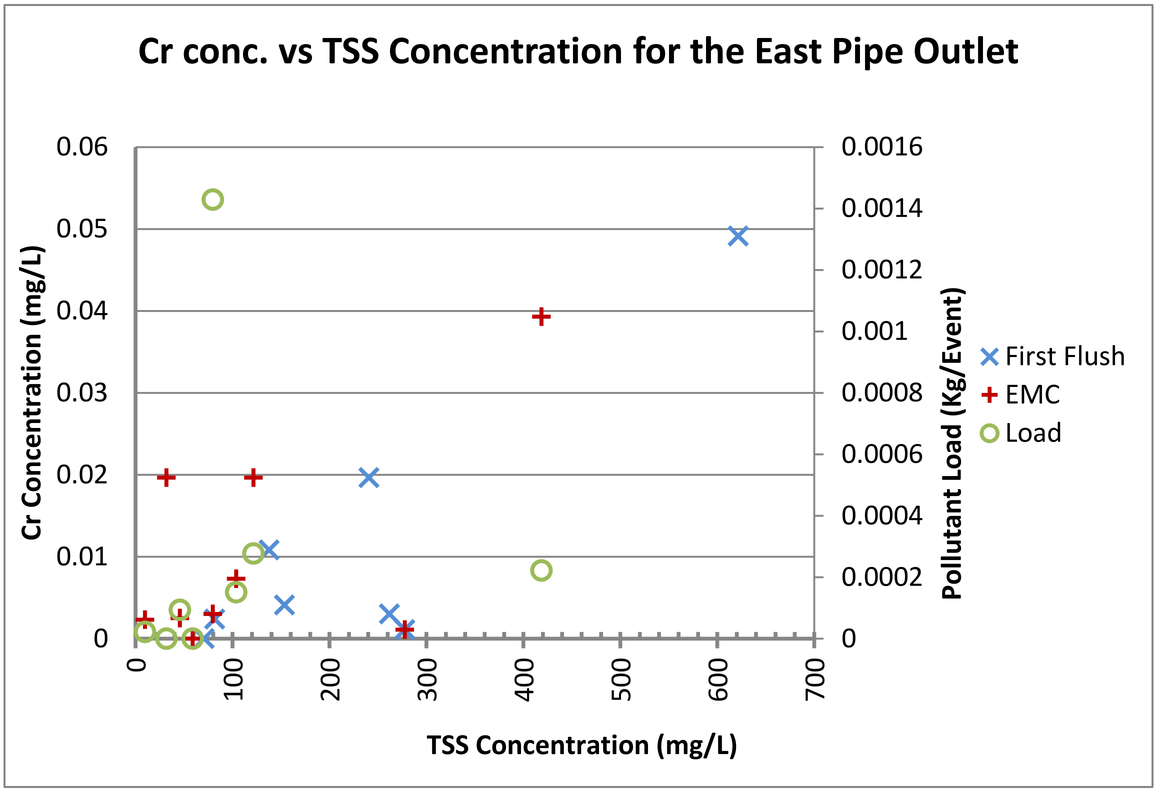

Figure 6.2 Cr and TSS concentrations for the east pipe outlet

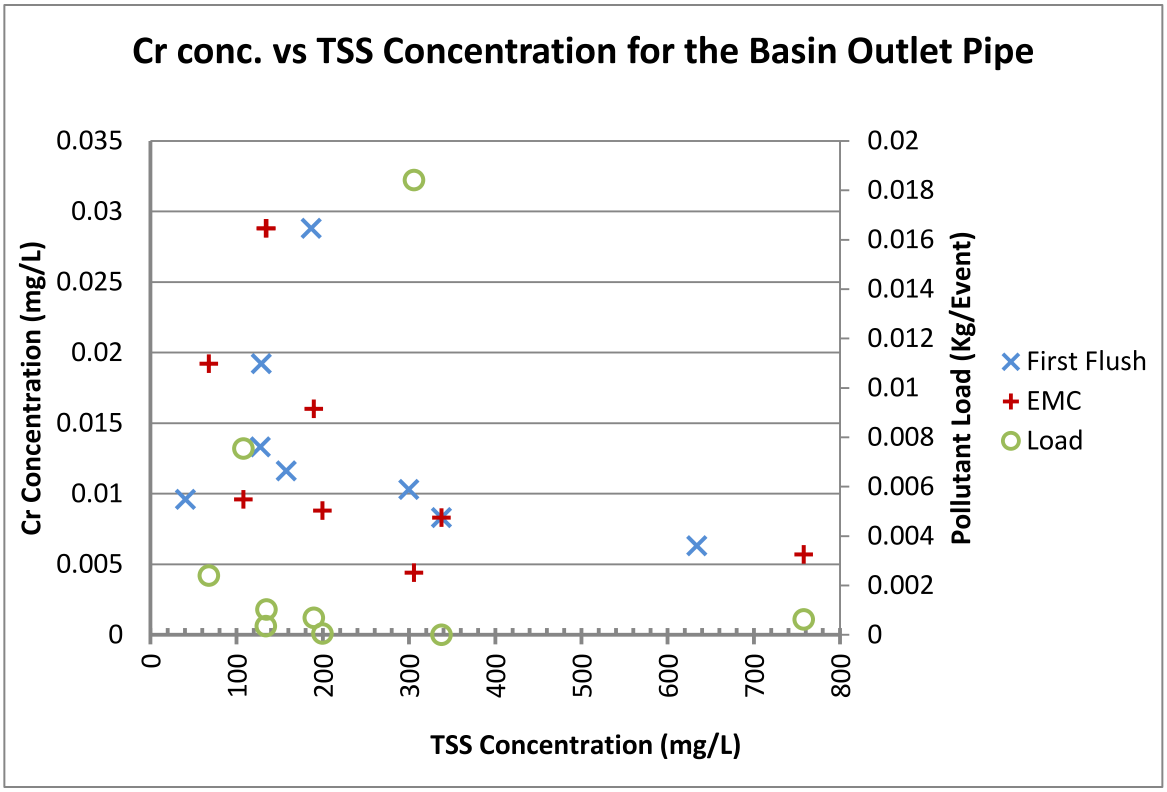

Figure 6.3 Cr and TSS concentrations for the basin outlet pipe

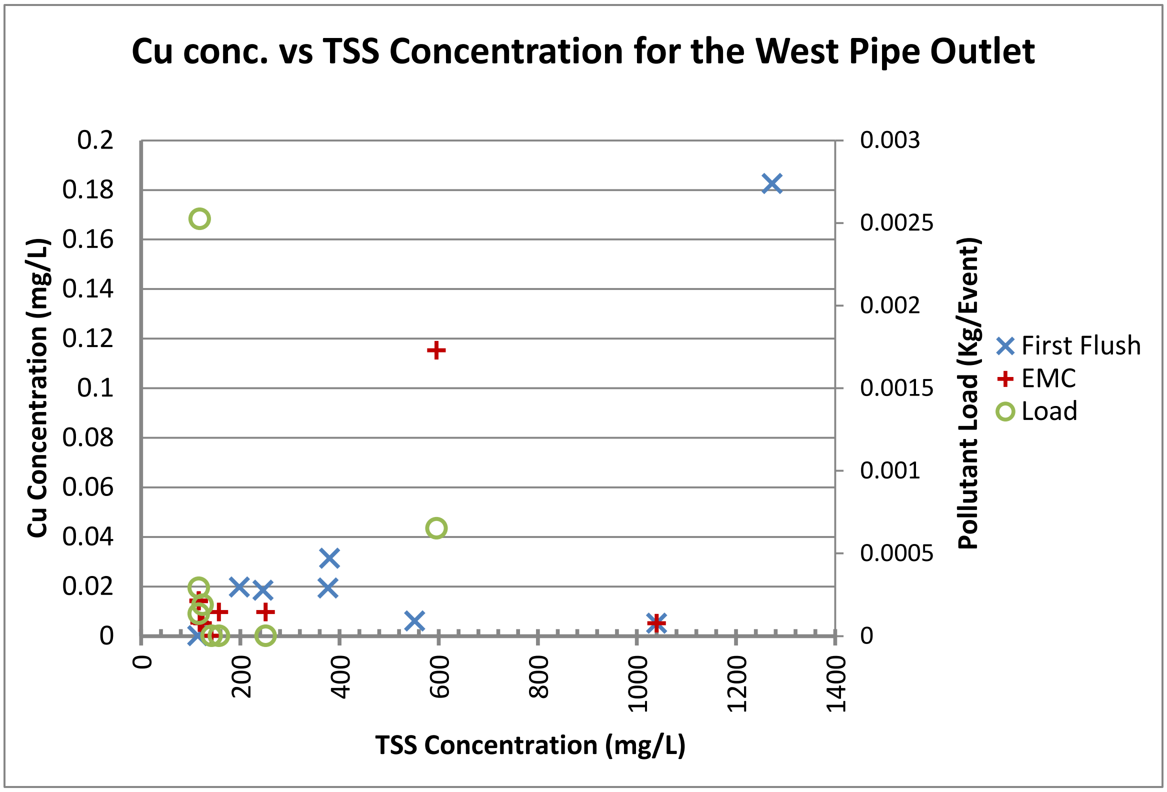

Figure 6.4 Cu and TSS concentrations for the west pipe outlet

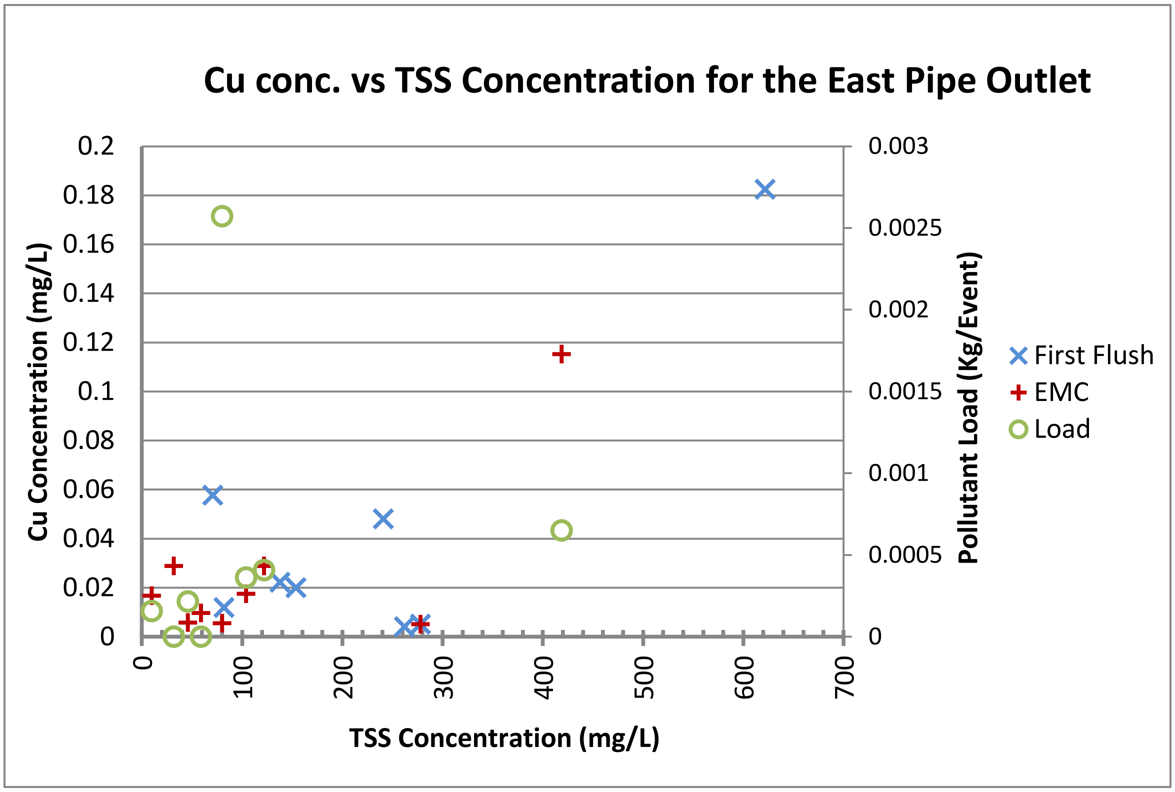

Figure 6.5 Cu and TSS concentrations for the east pipe outlet

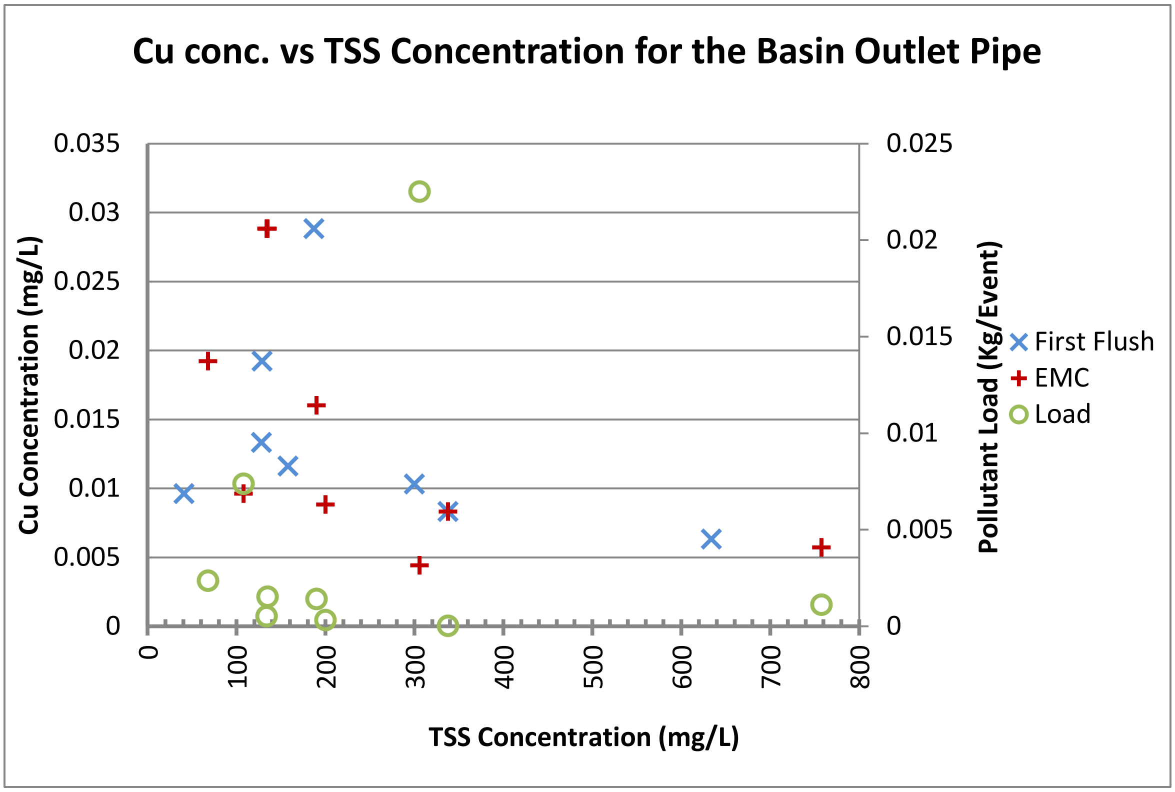

Figure 6.6 Cu and TSS concentrations for the basin outlet pipe

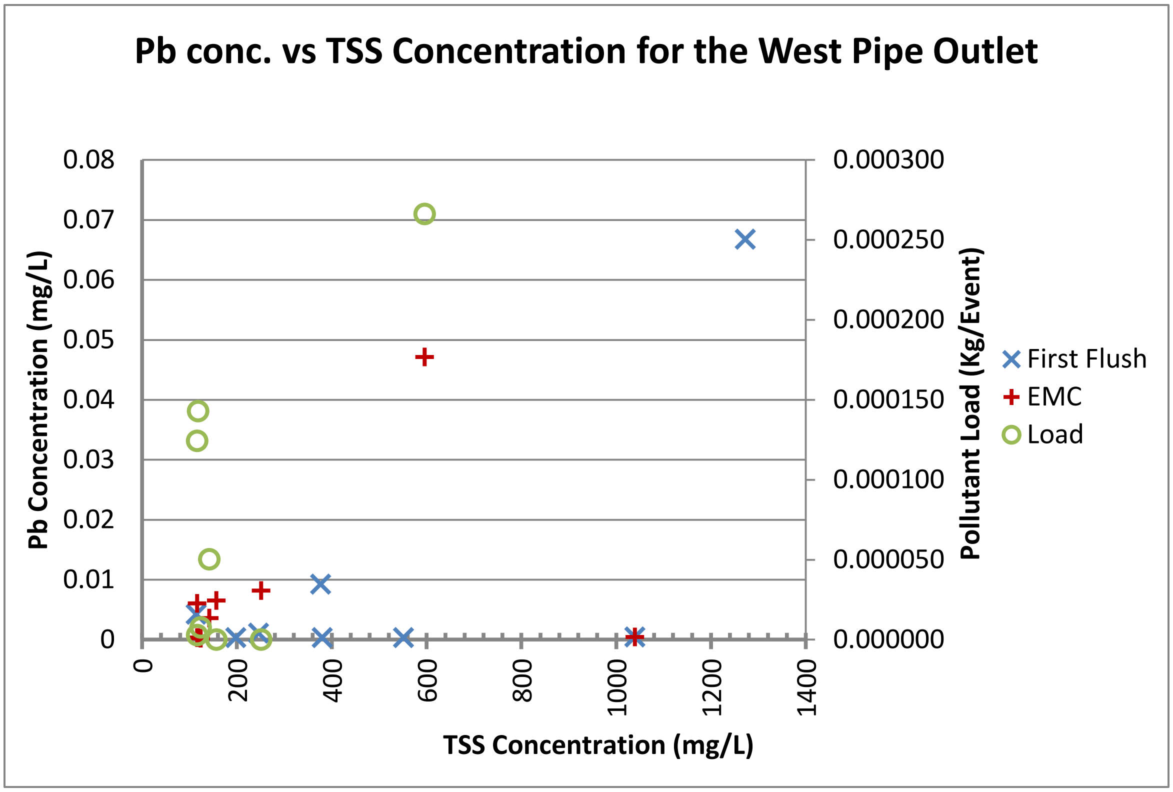

Figure 6.7 Pb and TSS concentrations for the west pipe outlet

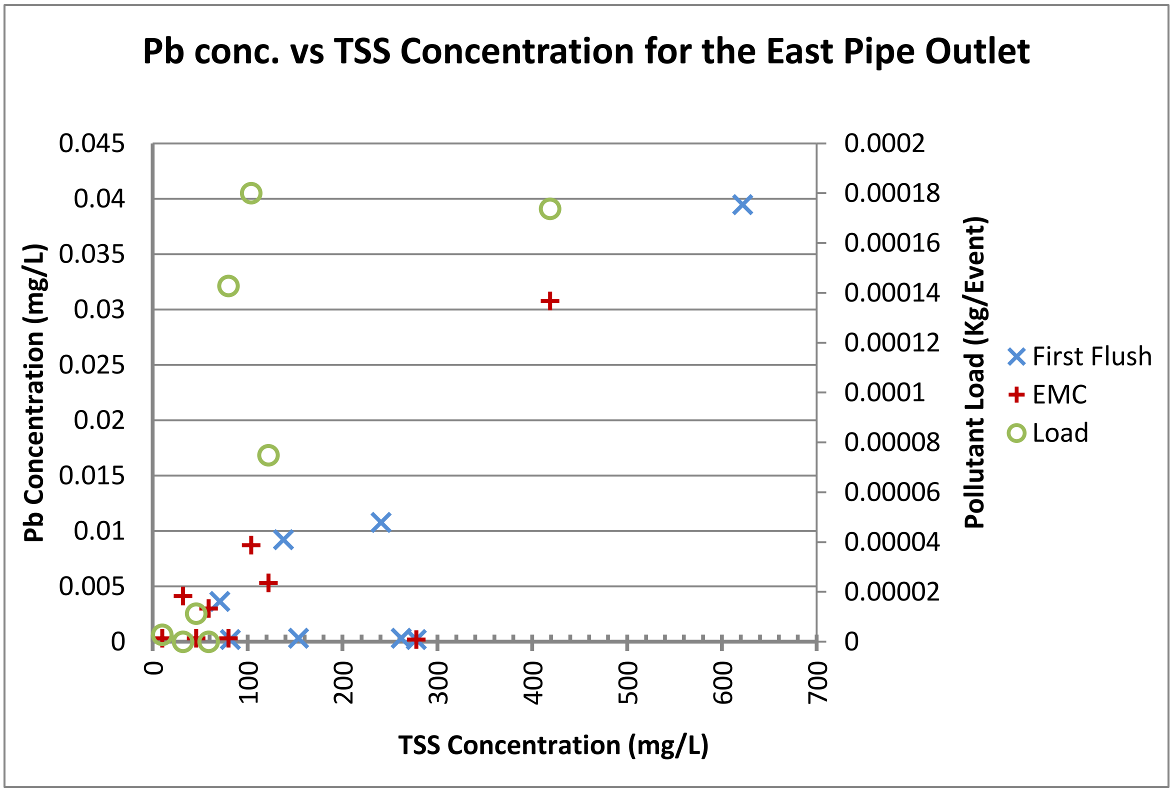

Figure 6.8 Pb and TSS concentrations for the east pipe outlet

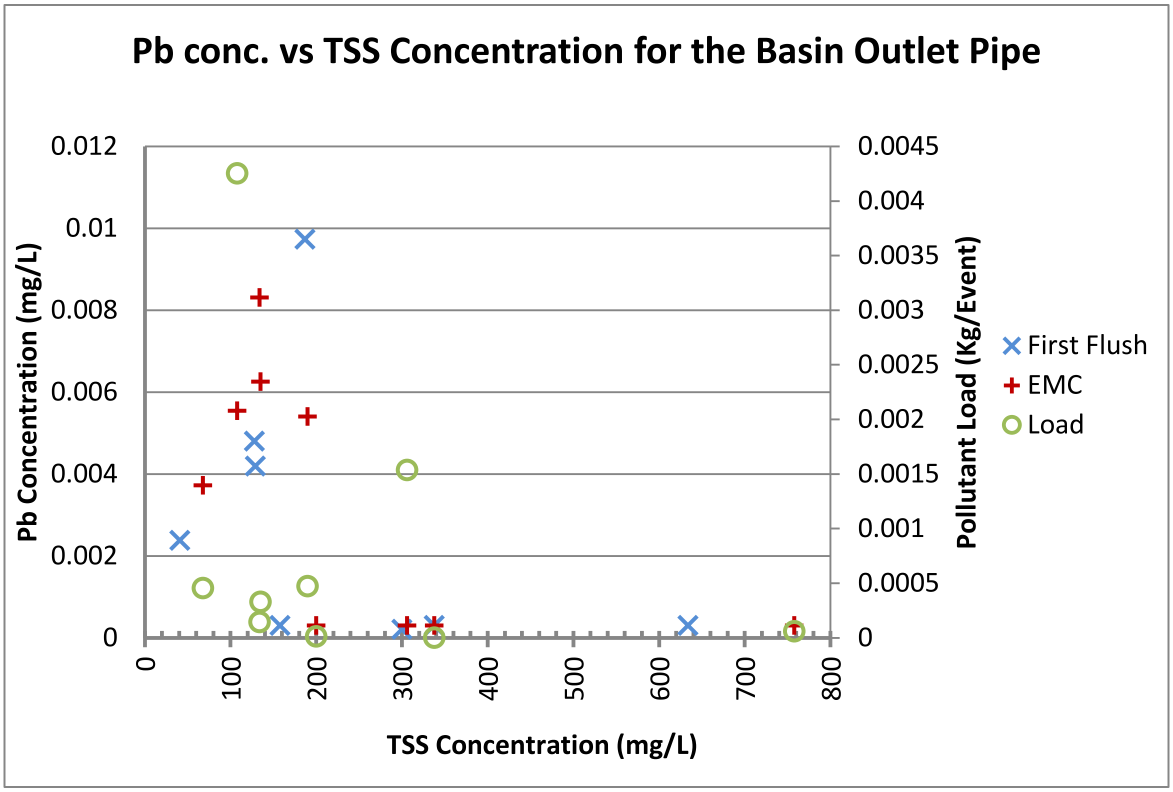

Figure 6.9 Pb and TSS concentrations for the basin outlet pipe

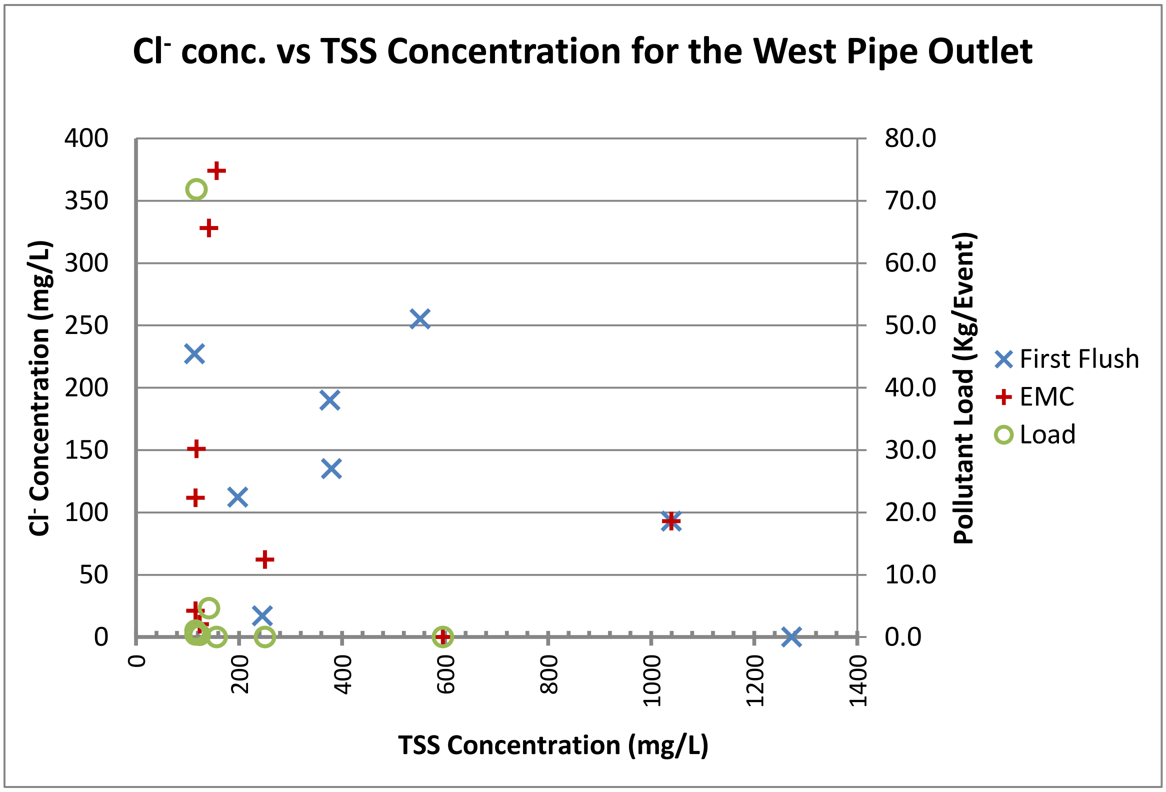

Figure 6.10 Cl^- and TSS concentrations for the west pipe outlet

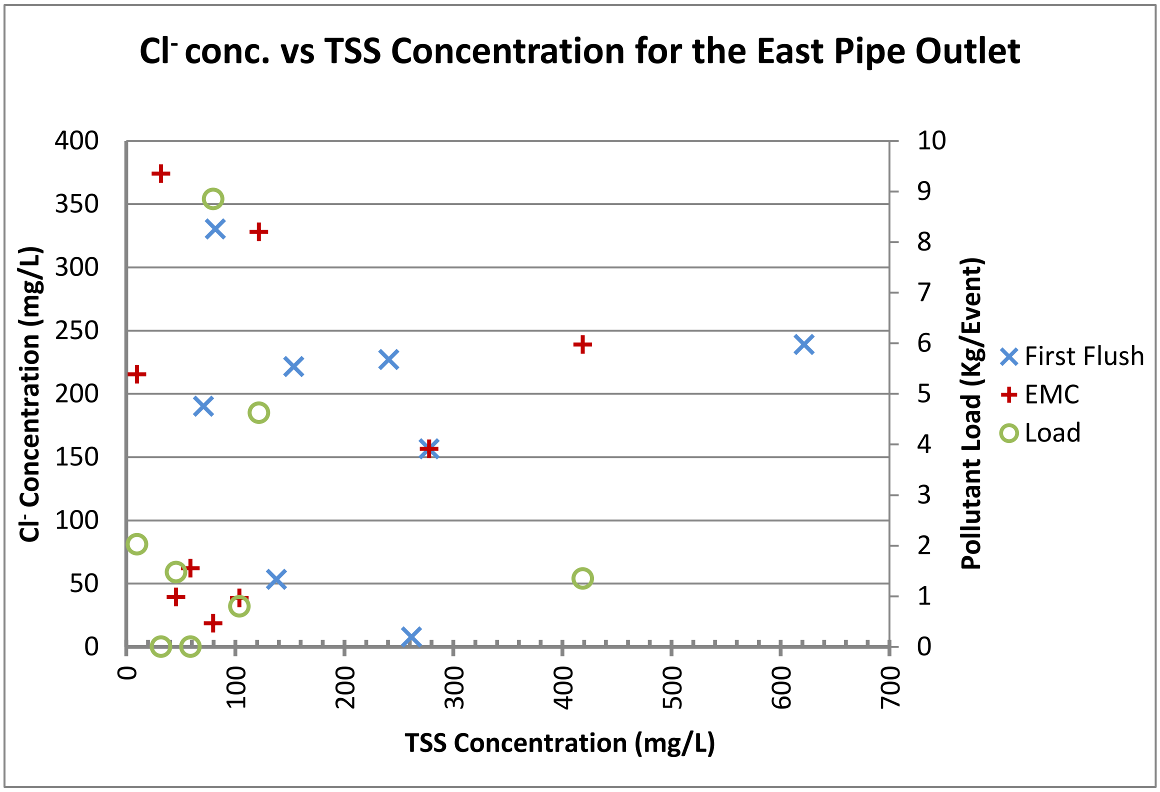

Figure 6.11 Cl$^-$ and TSS concentrations for the east pipe outlet

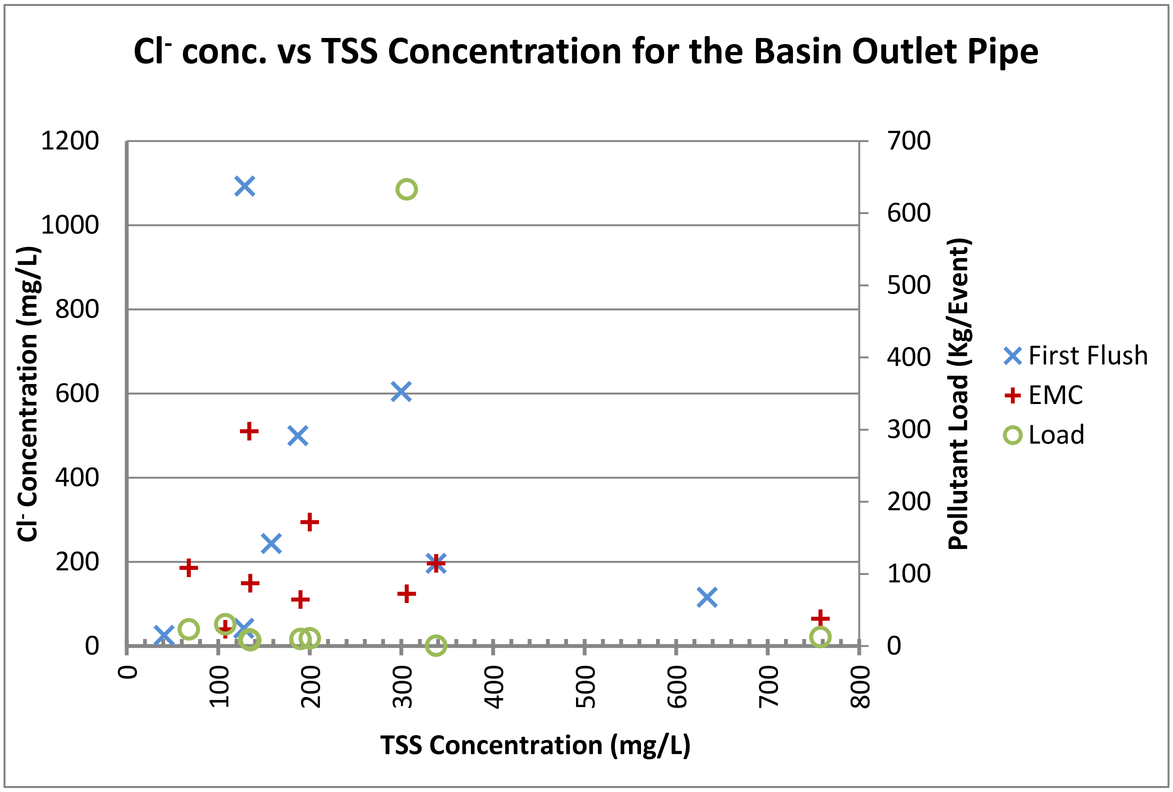

Figure 6.12 Cl$^-$ and TSS concentrations for the basin outlet pipe

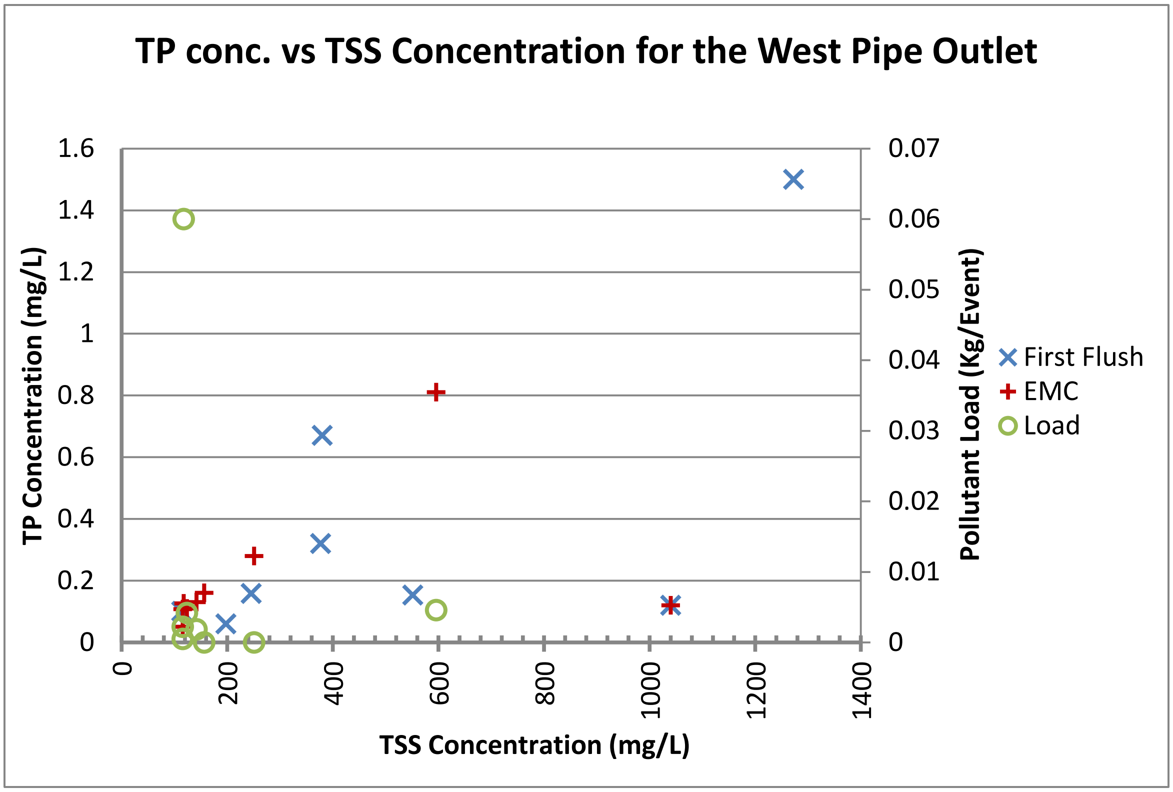

Figure 6.13 TP and TSS concentrations for the west pipe outlet

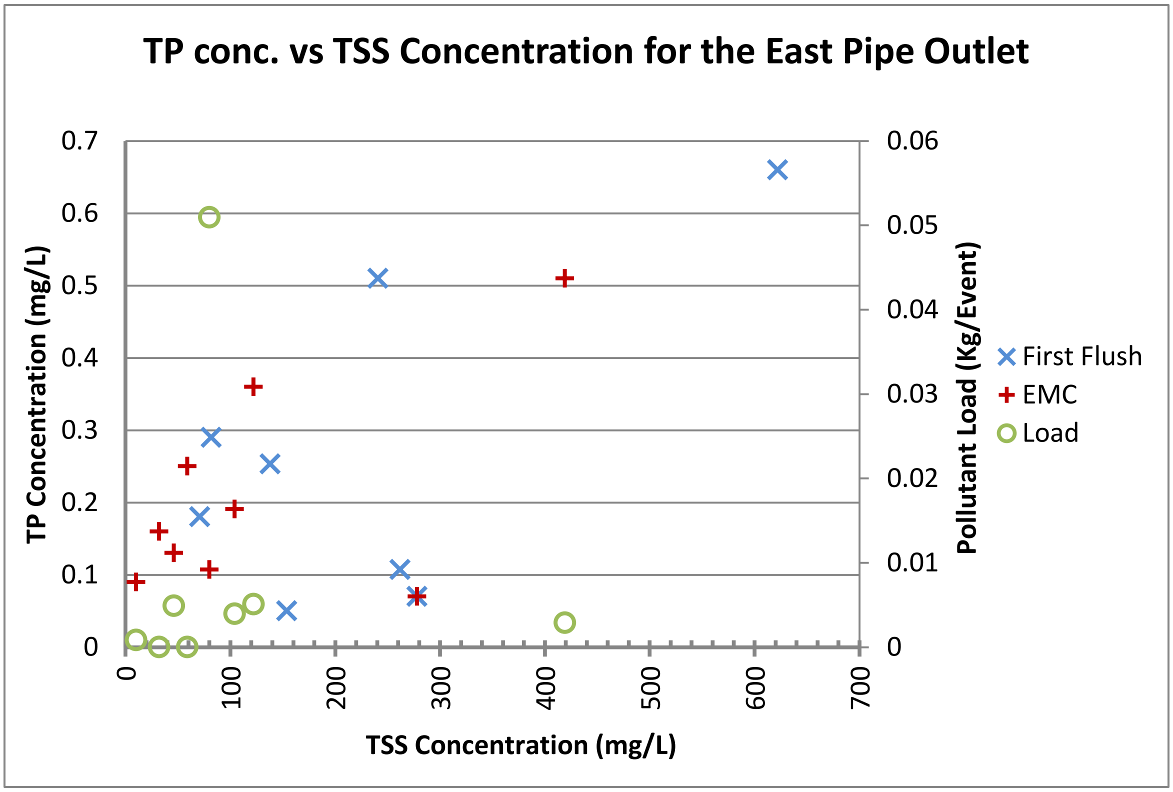

Figure 6.14 TP and TSS concentrations for the east pipe outlet

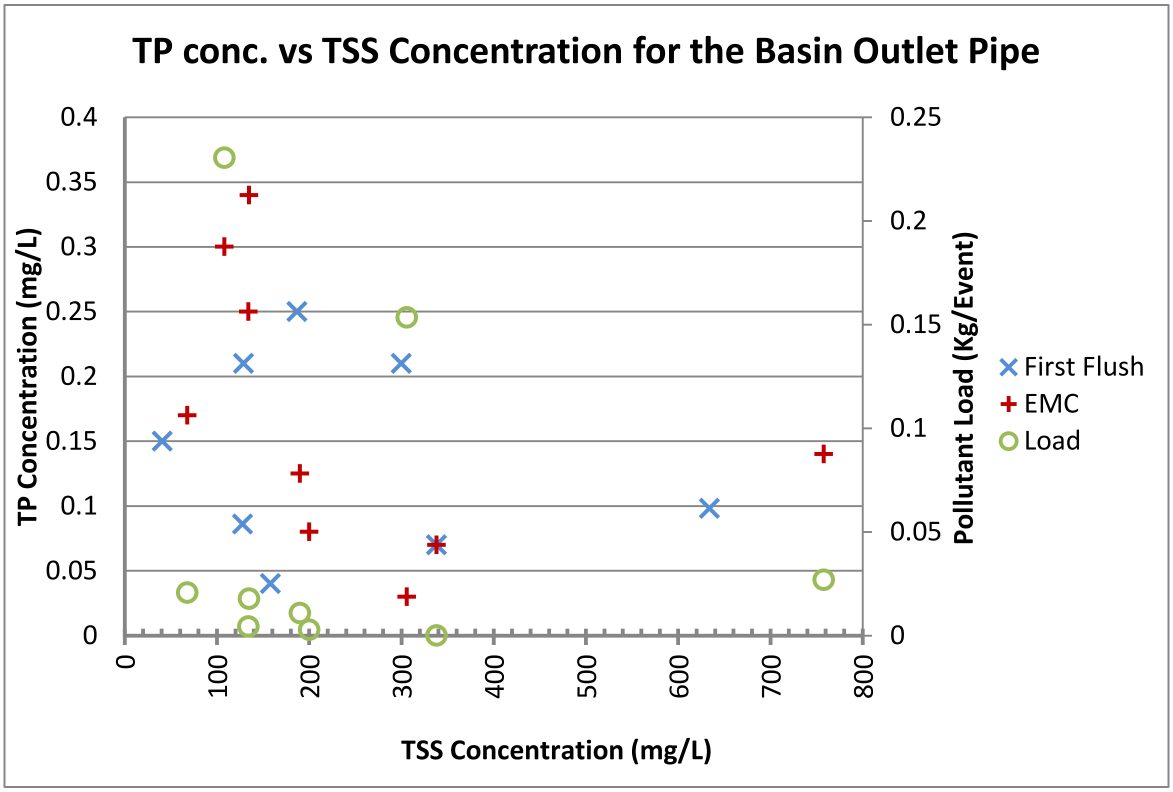

Figure 6.15 TP and TSS concentrations for the basin outlet pipe

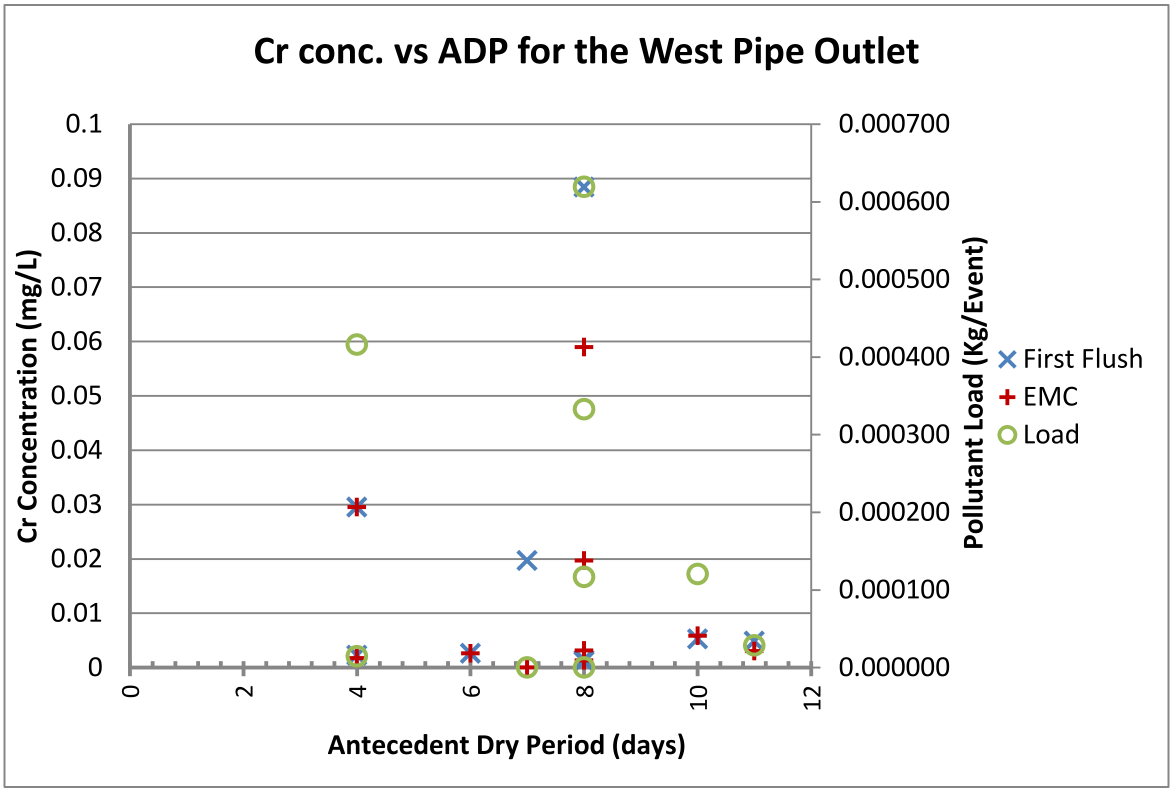

Figure 6.16 Cr and ADP for the west pipe outlet

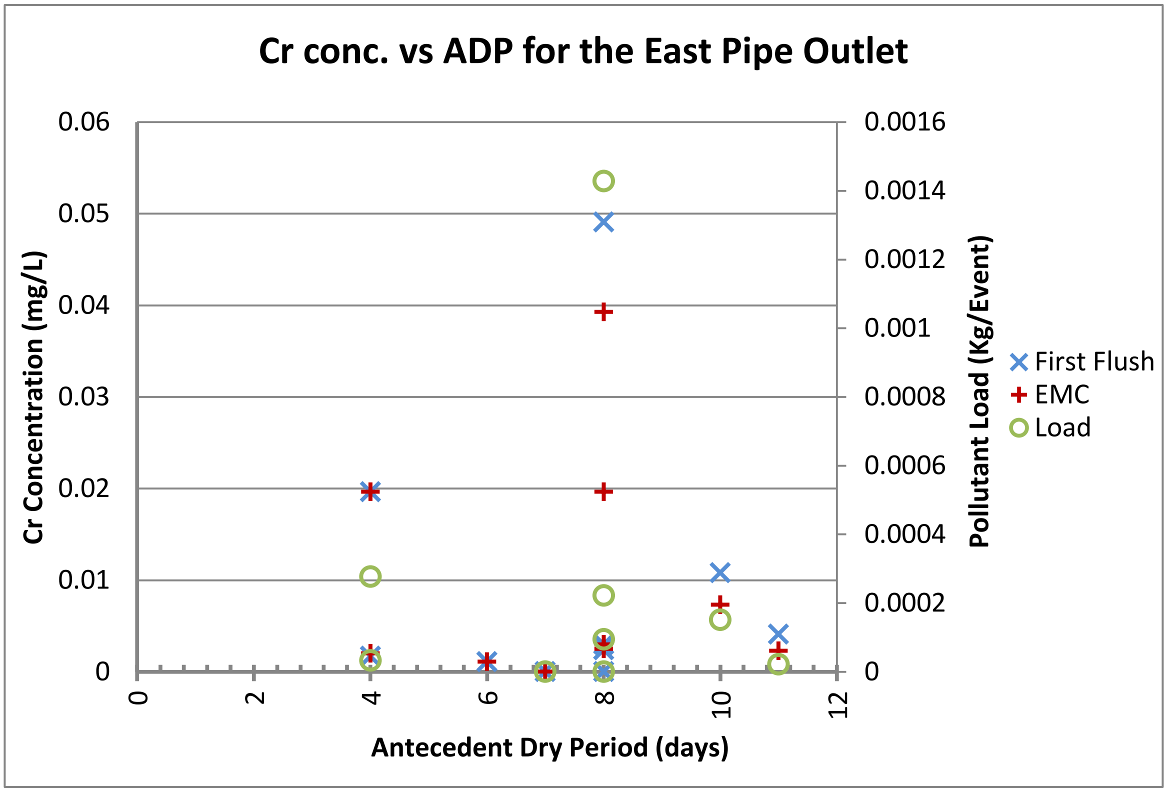

Figure 6.17 Cr and ADP for the east pipe outlet

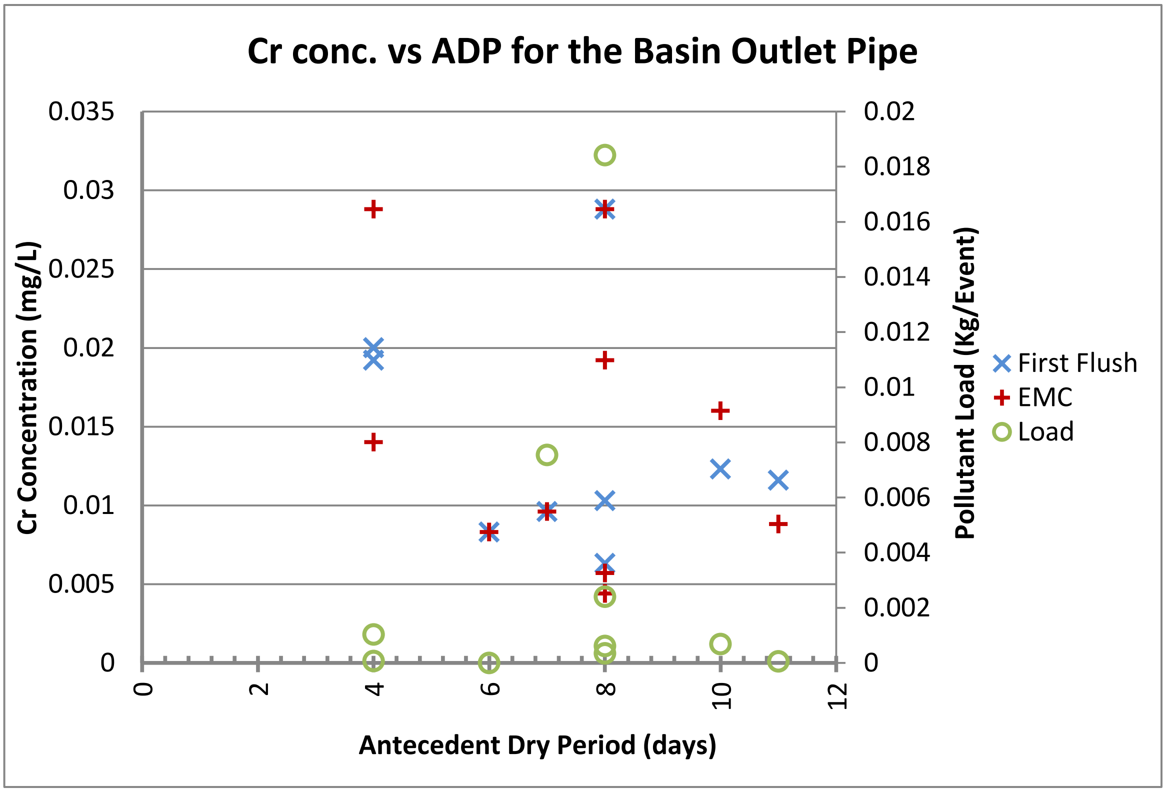

Figure 6.18 Cr and ADP for the basin outlet pipe

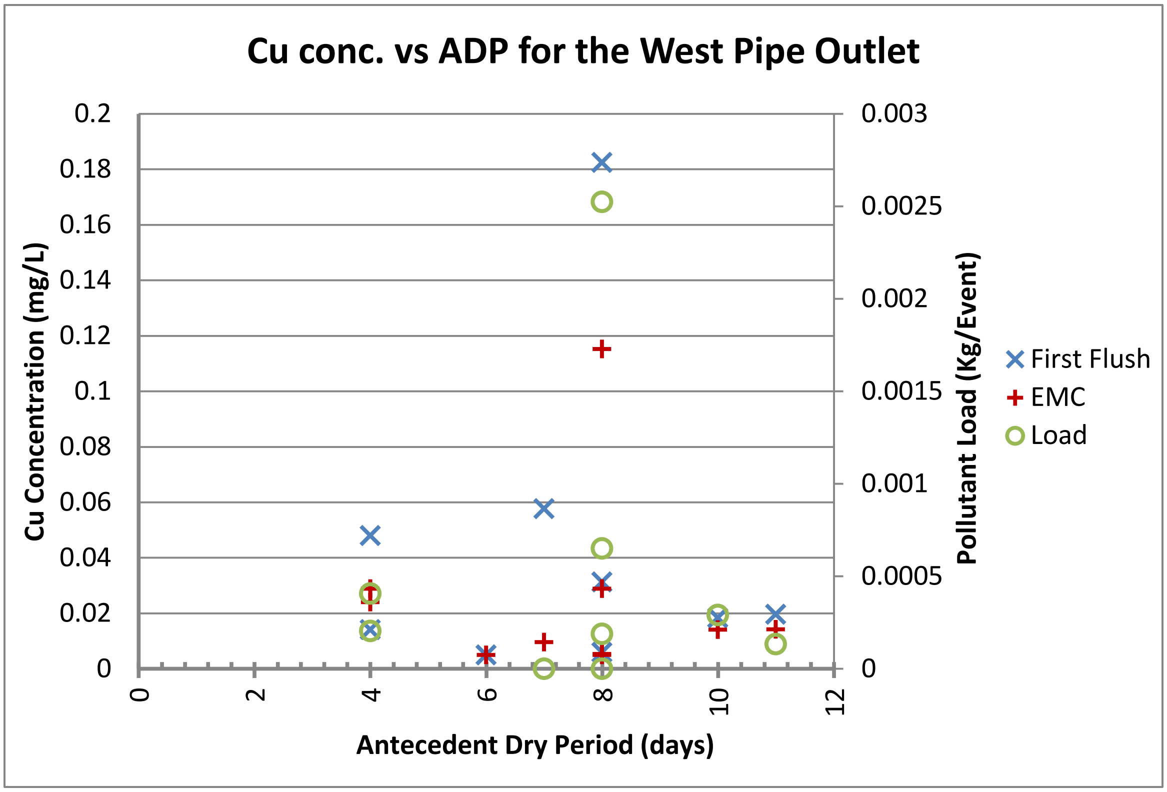

Figure 6.19 Cu and ADP for the west pipe outlet

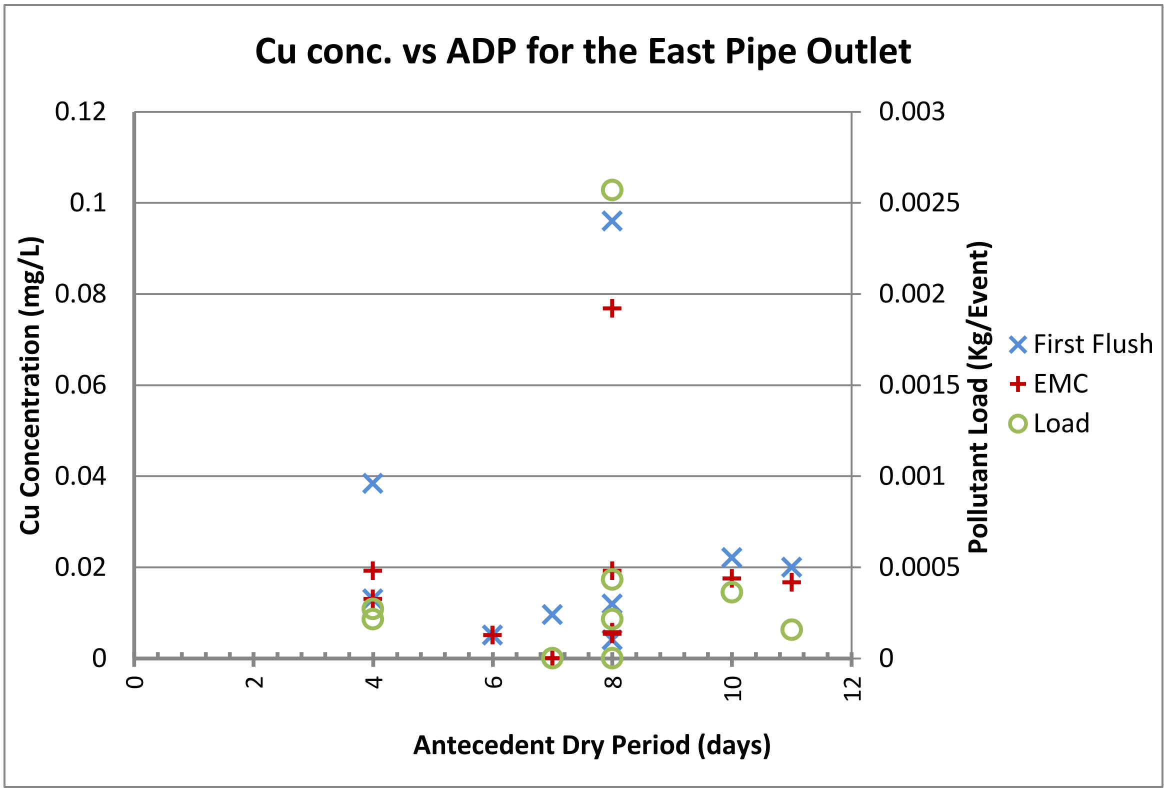

Figure 6.20 Cu and ADP for the east pipe outlet

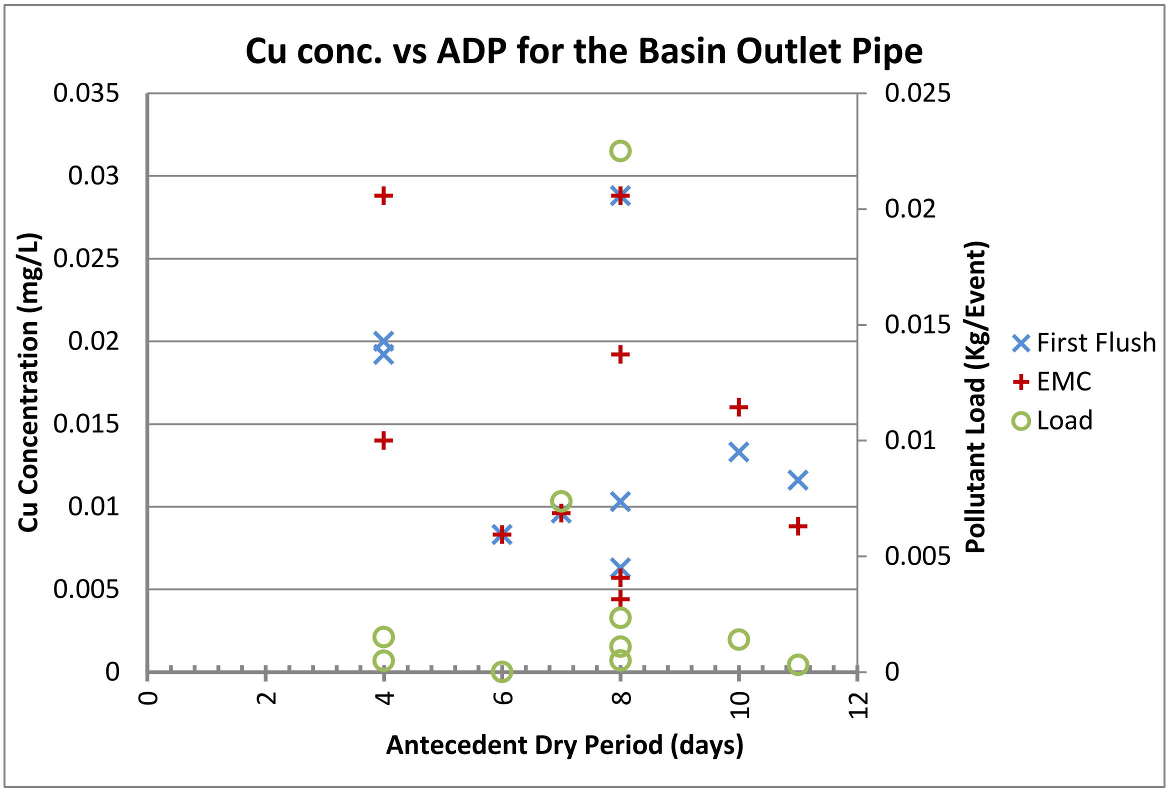

Figure 6.21 Cu and ADP for the basin outlet pipe

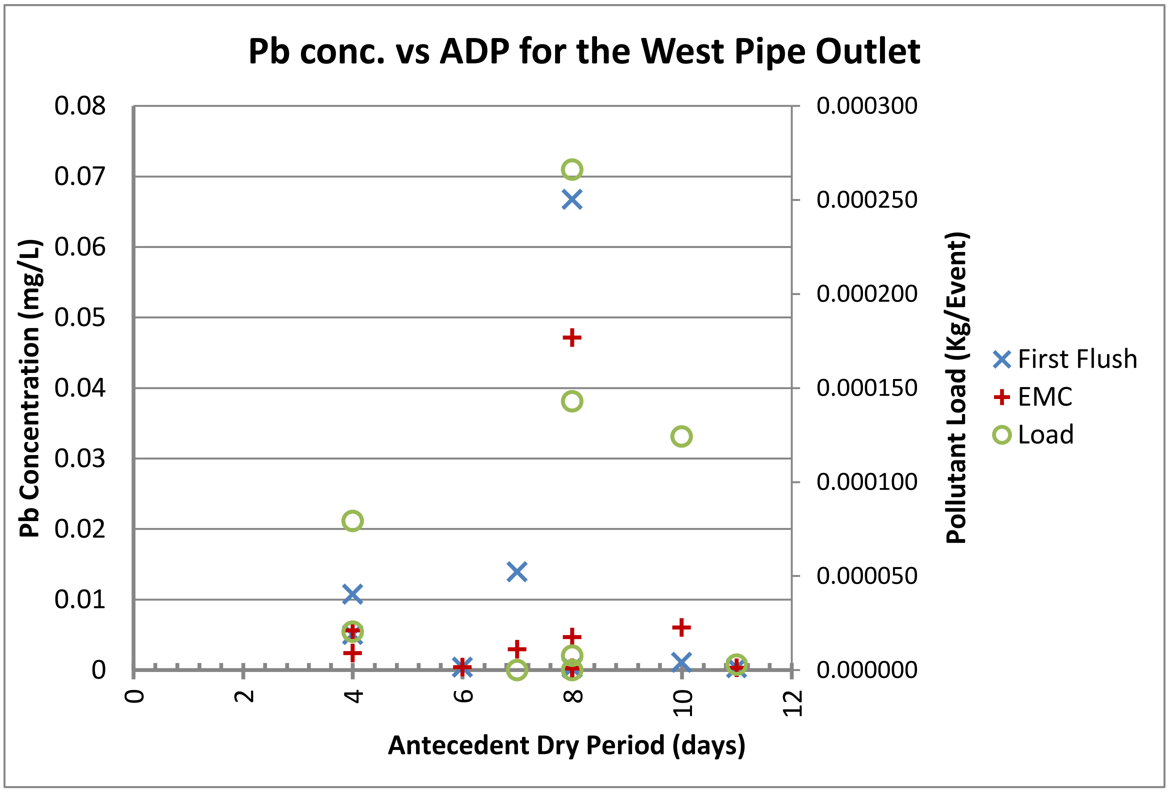

Figure 6.22 Pb and ADP for the west pipe outlet

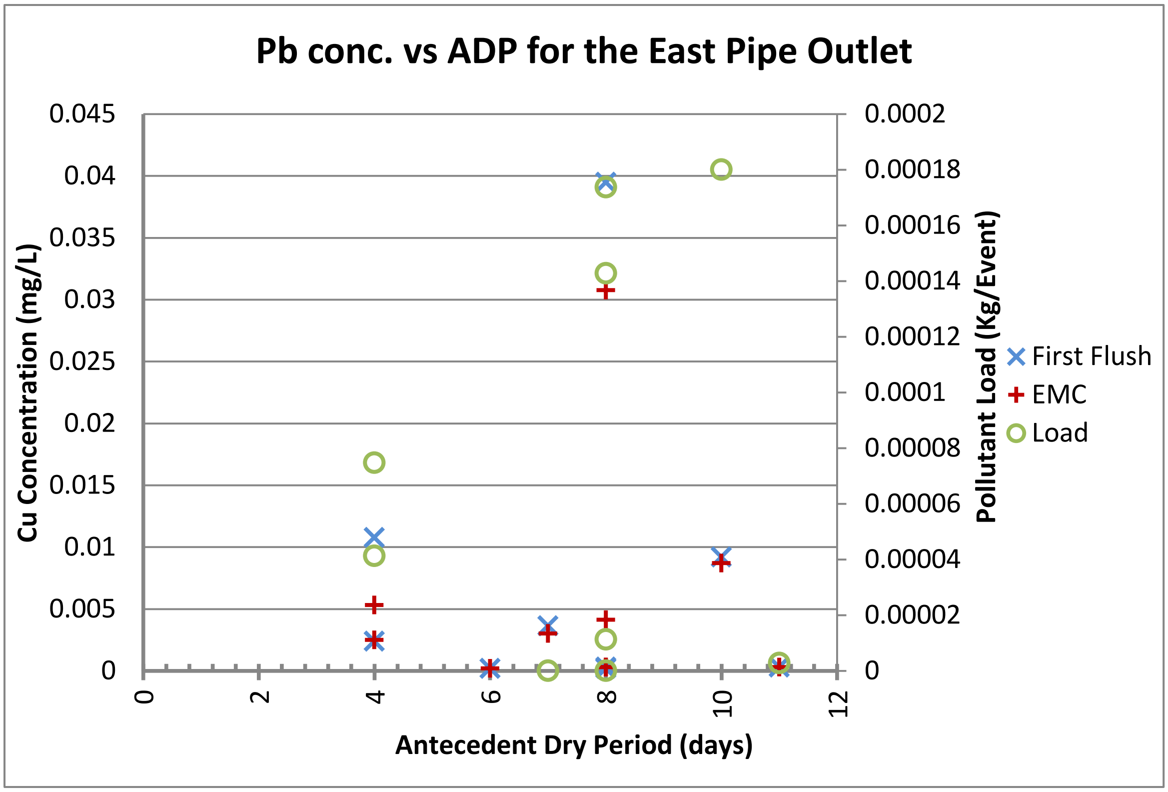

Figure 6.23 Pb and ADP for the east pipe outlet

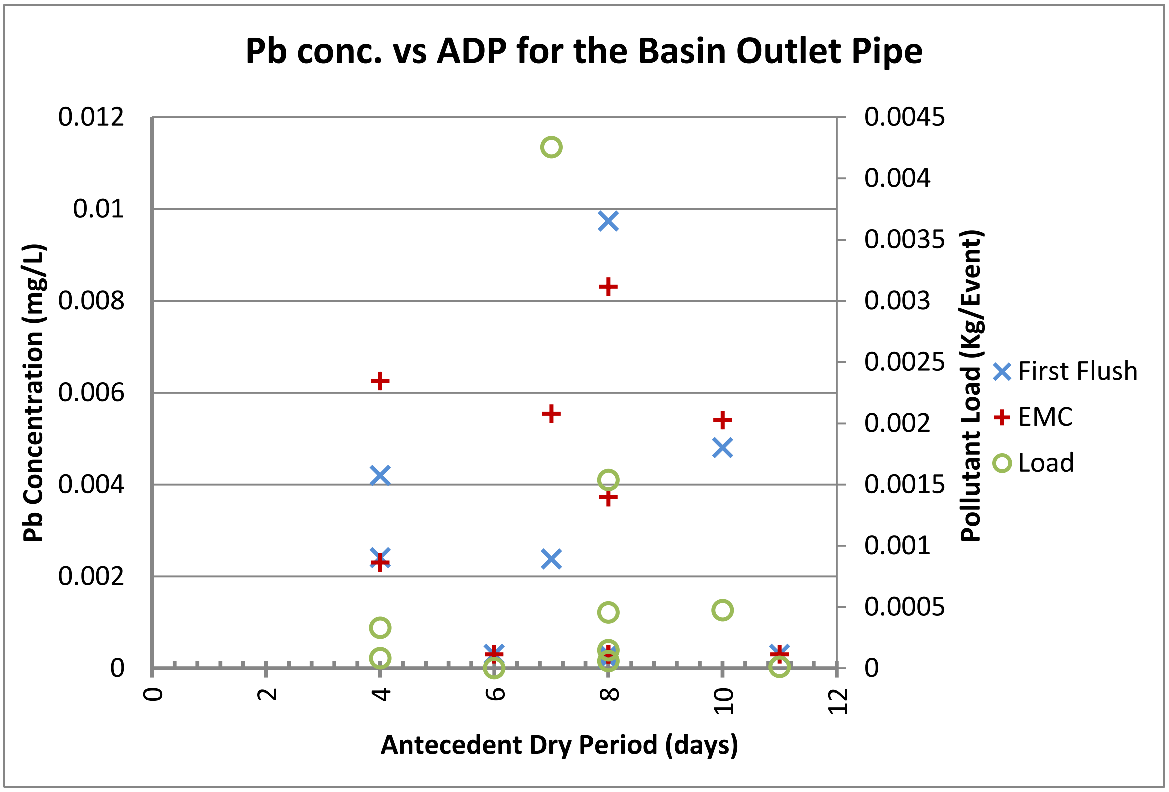

Figure 6.24 Pb and ADP for the basin outlet pipe

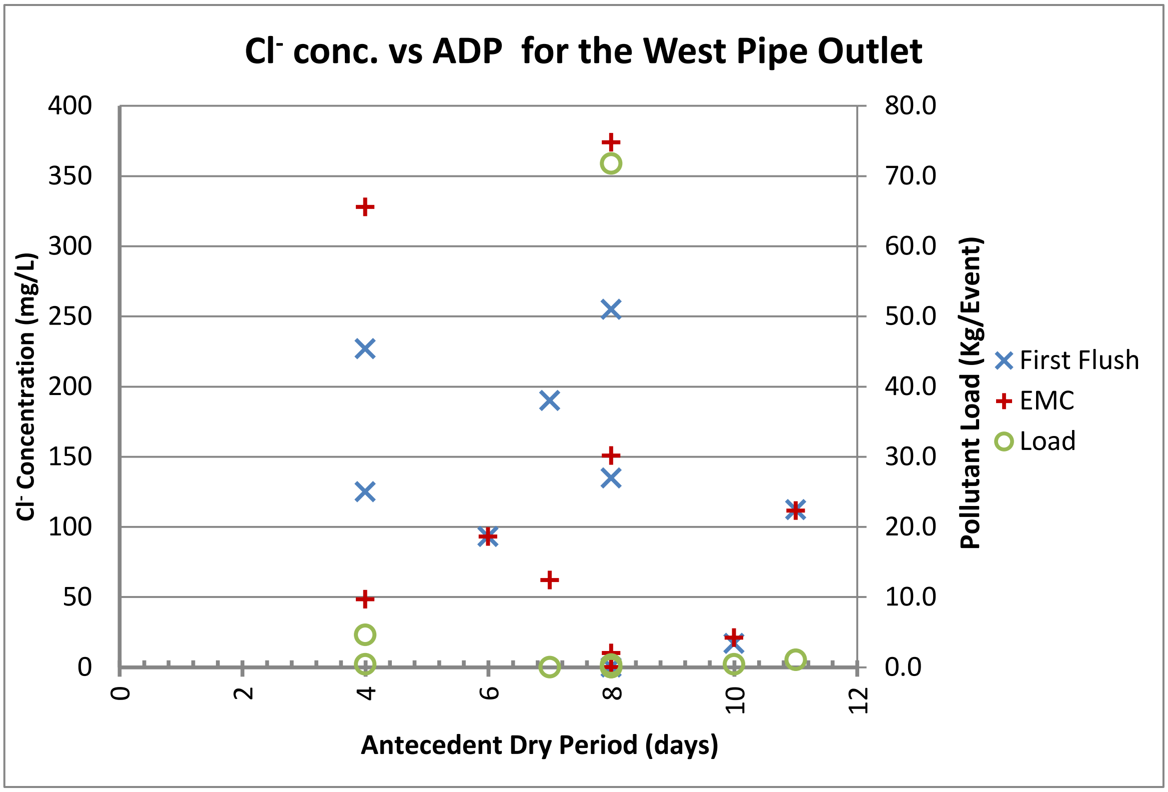

Figure 6.25 Cl^- and ADP for the west pipe outlet

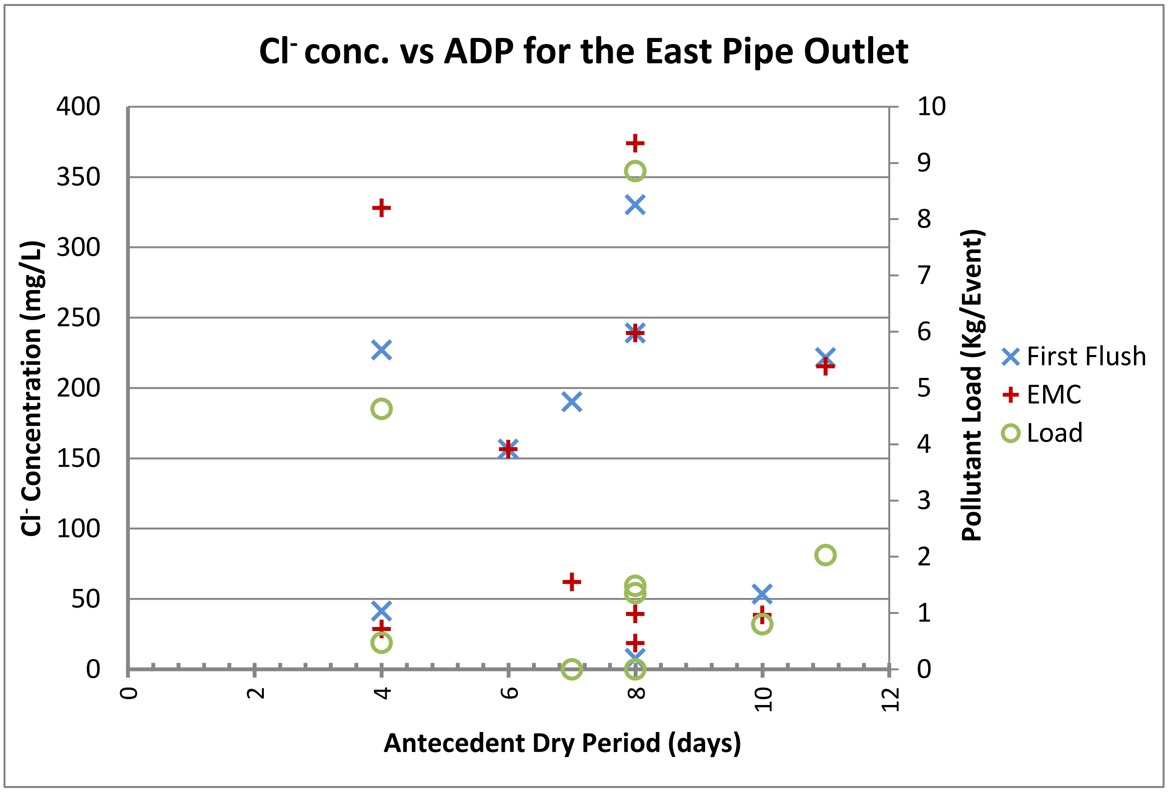

Figure 6.26 Cl^- and ADP for the east pipe outlet

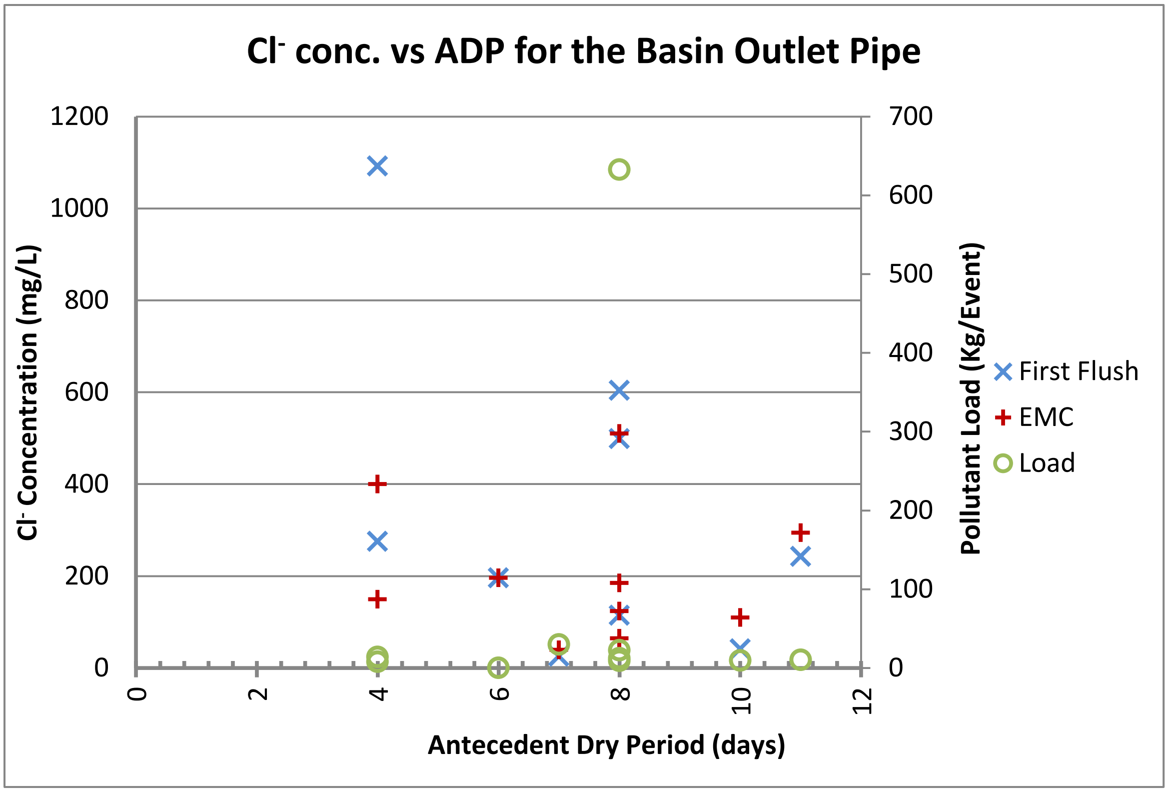

Figure 6.27 Cl$^-$ and ADP for the basin outlet pipe

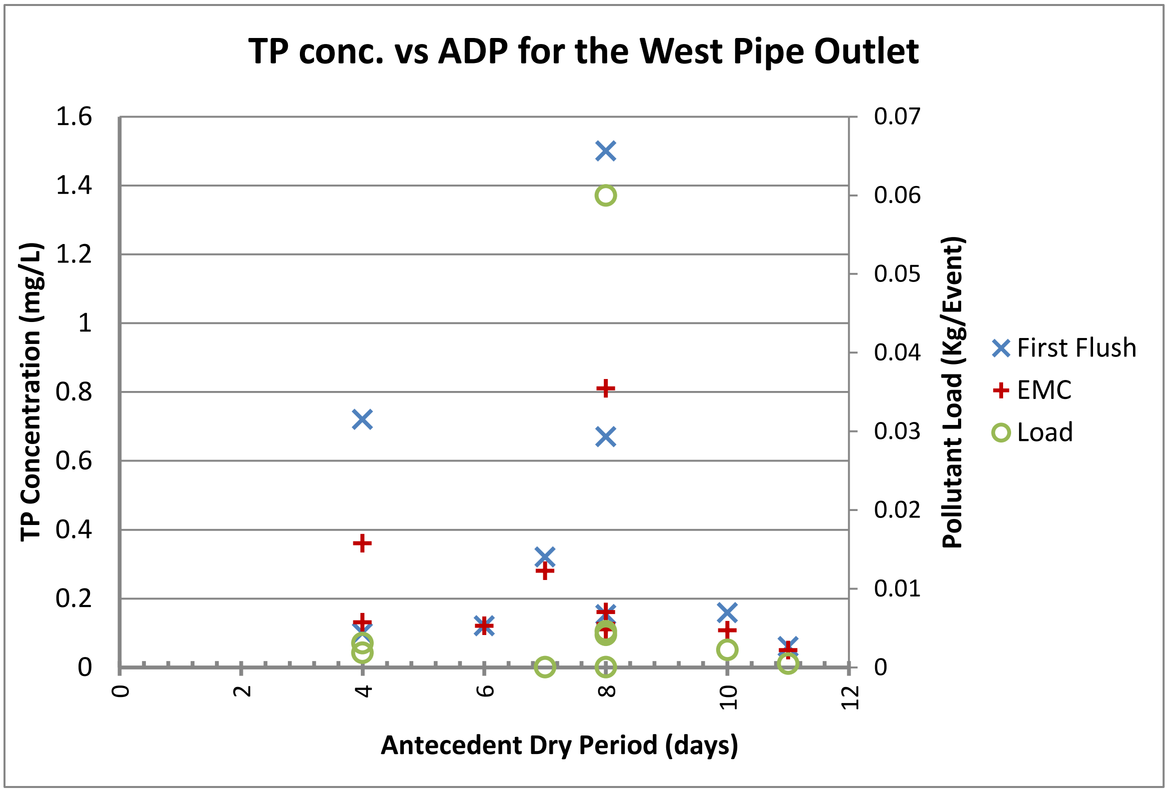

Figure 6.28 TP and ADP for the west pipe outlet

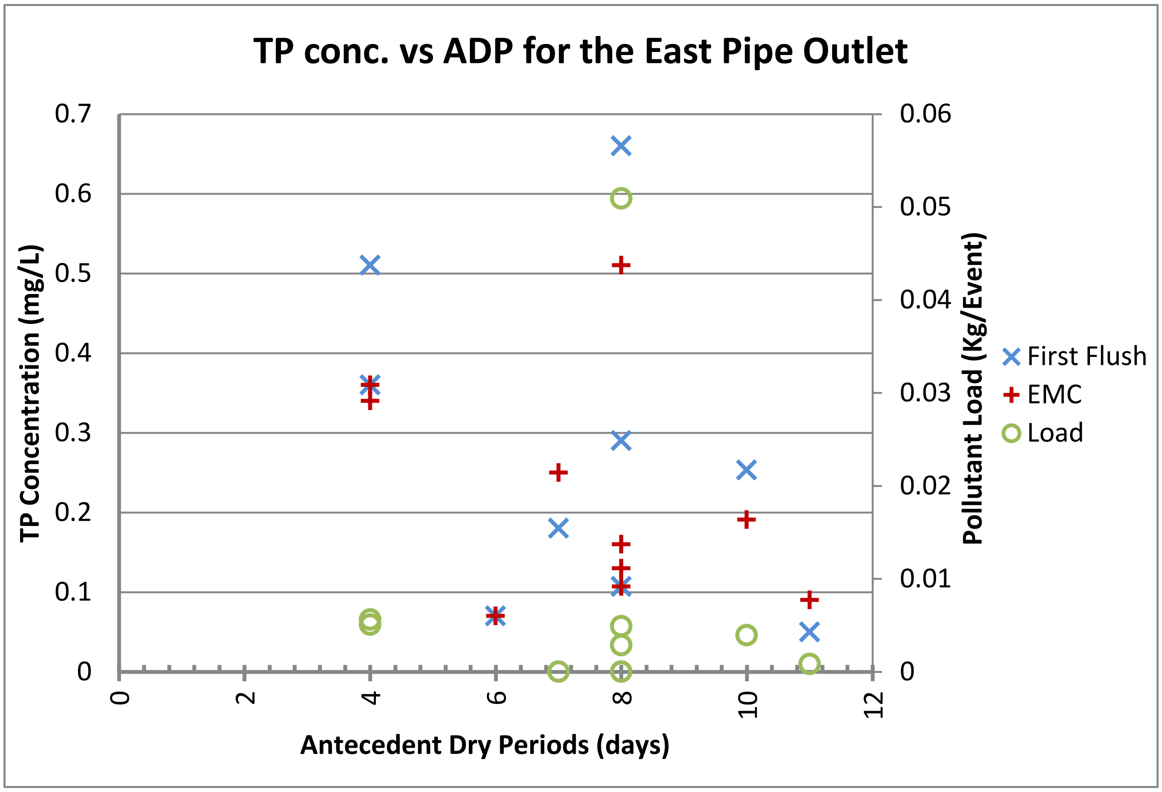

Figure 6.29 TP and ADP for the east pipe outlet

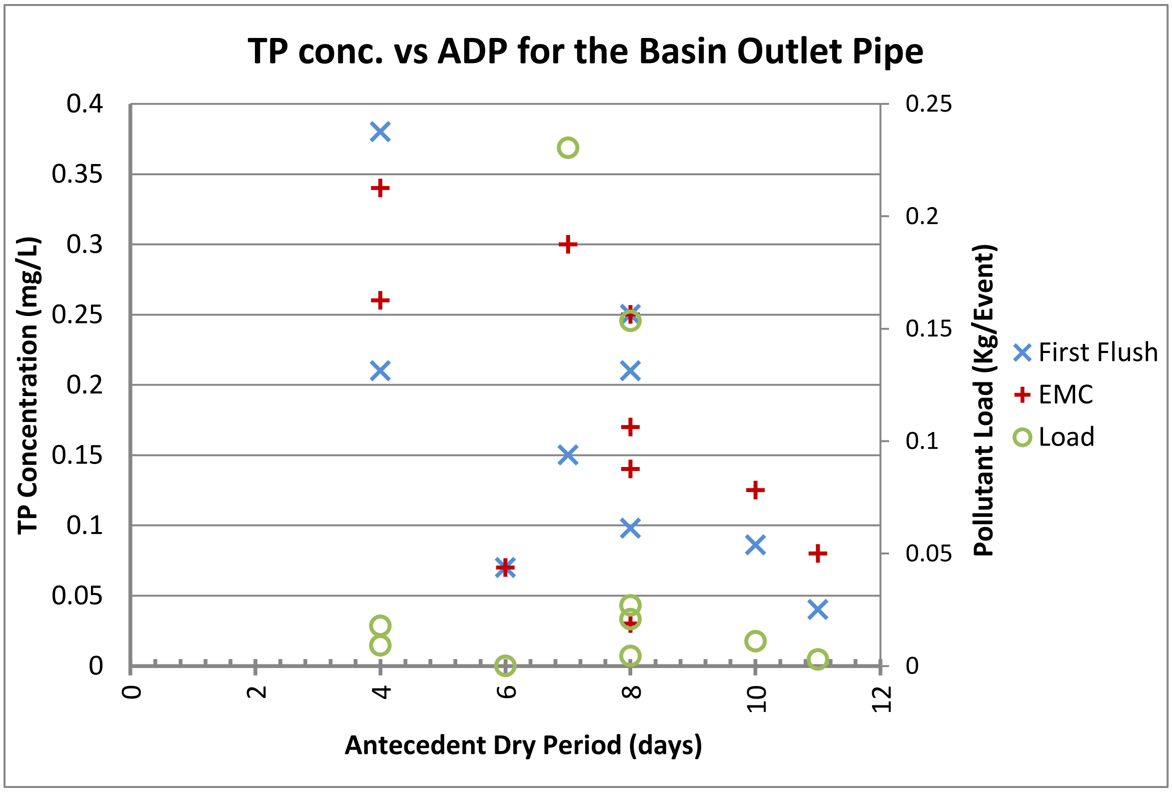

Figure 6.30 TP and ADP for the basin outlet pipe

6.1.3 Pollutant Relationship with Total Rainfall

Concentrations and loads for chromium (Cr), copper (Cu), lead (Pb), chloride (Cl^-) and total phosphorus (TP) were compared to the total rainfall for each of the sampled events for the west, east, and basin outlet pipes. This comparison is shown in figures 6.31 through 6.45. The figures do not show a clear relationship between concentrations and total rainfall for any of the analytes used in the comparison. Concentrations for high intensity events seem to be lower than for lower intensity storms. However, during the study, few high intensity storms were sampled; therefore, it is not possible to be completely sure about this trend.

6.1.4 Pollutant Relationship with Volume of Runoff

Concentrations and loads for chromium (Cr), copper (Cu), lead (Pb), chloride (Cl^-) and total phosphorus (TP) were compared to the volume of runoff for each of the sampled events for the west, east, and basin outlet pipes. This comparison is shown in figures 6.46 through 6.60. Concentrations for chromium, copper, lead, and total phosphorus seem to be unaffected by the volume of runoff for the east and west pipe.

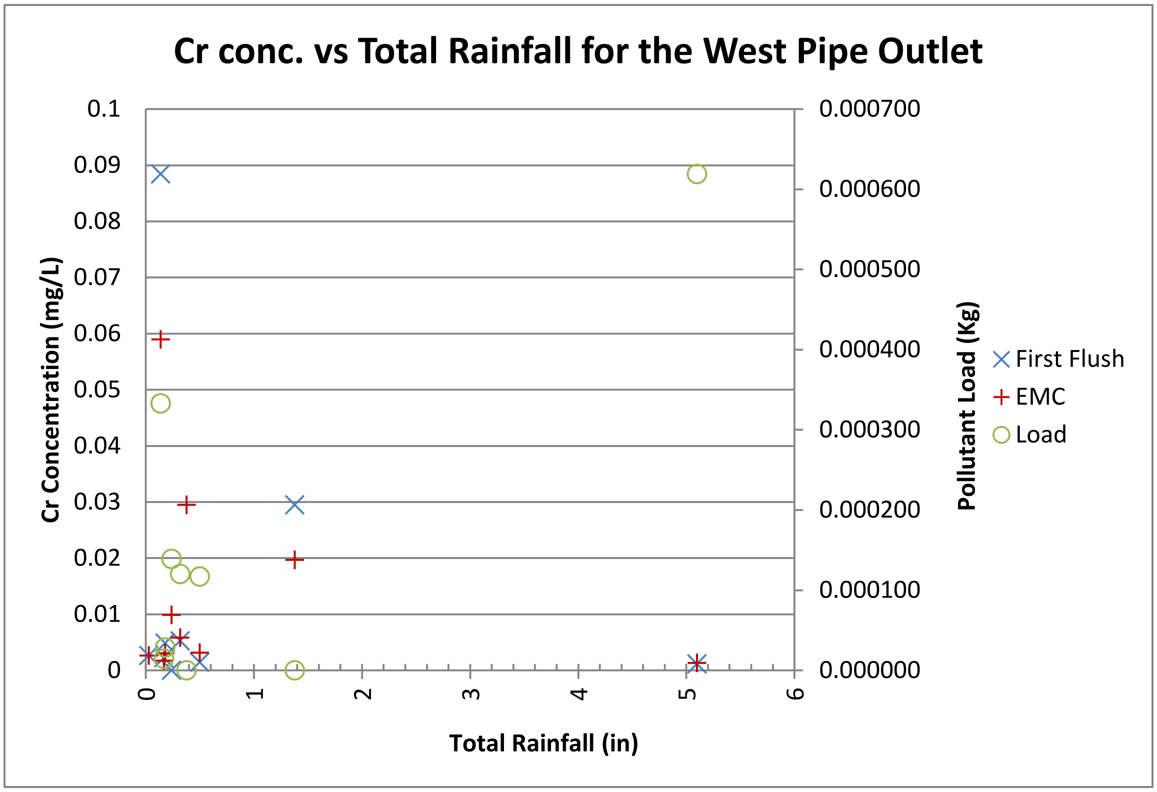

Figure 6.31 Cr concentration and total rainfall for the west pipe outlet

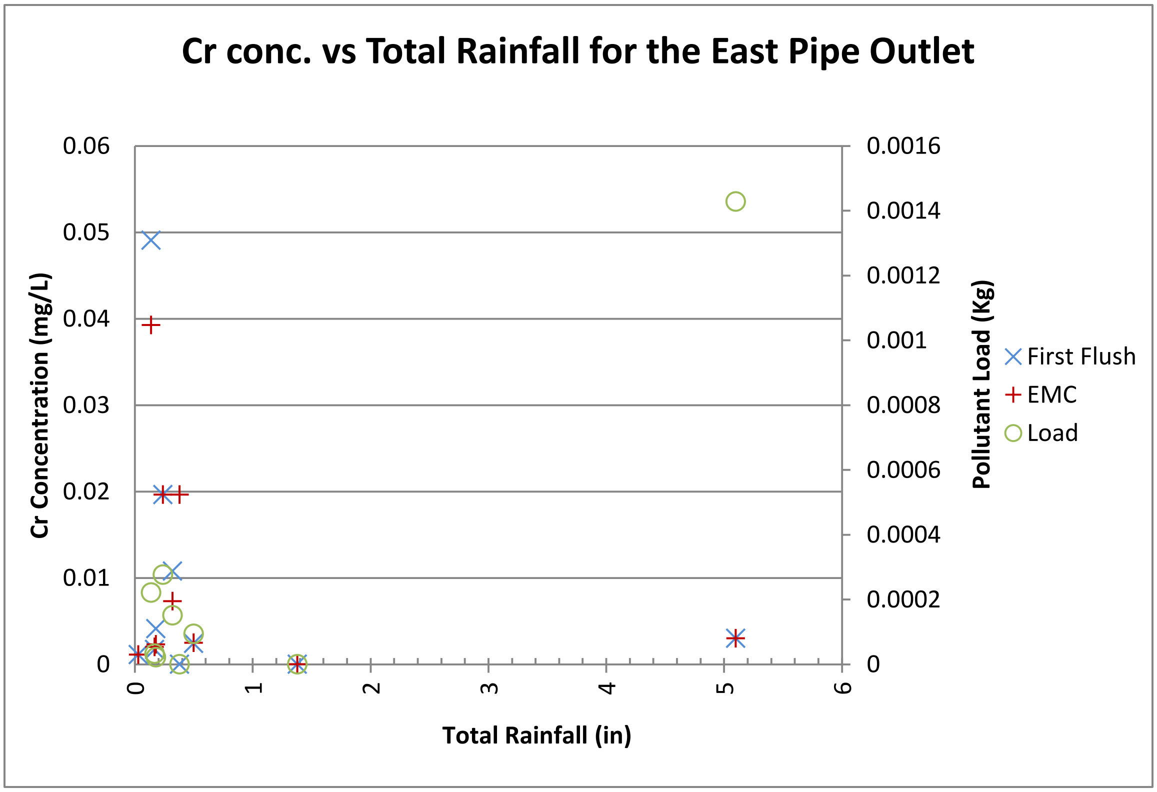

Figure 6.32 Cr concentration and total rainfall for the east pipe outlet

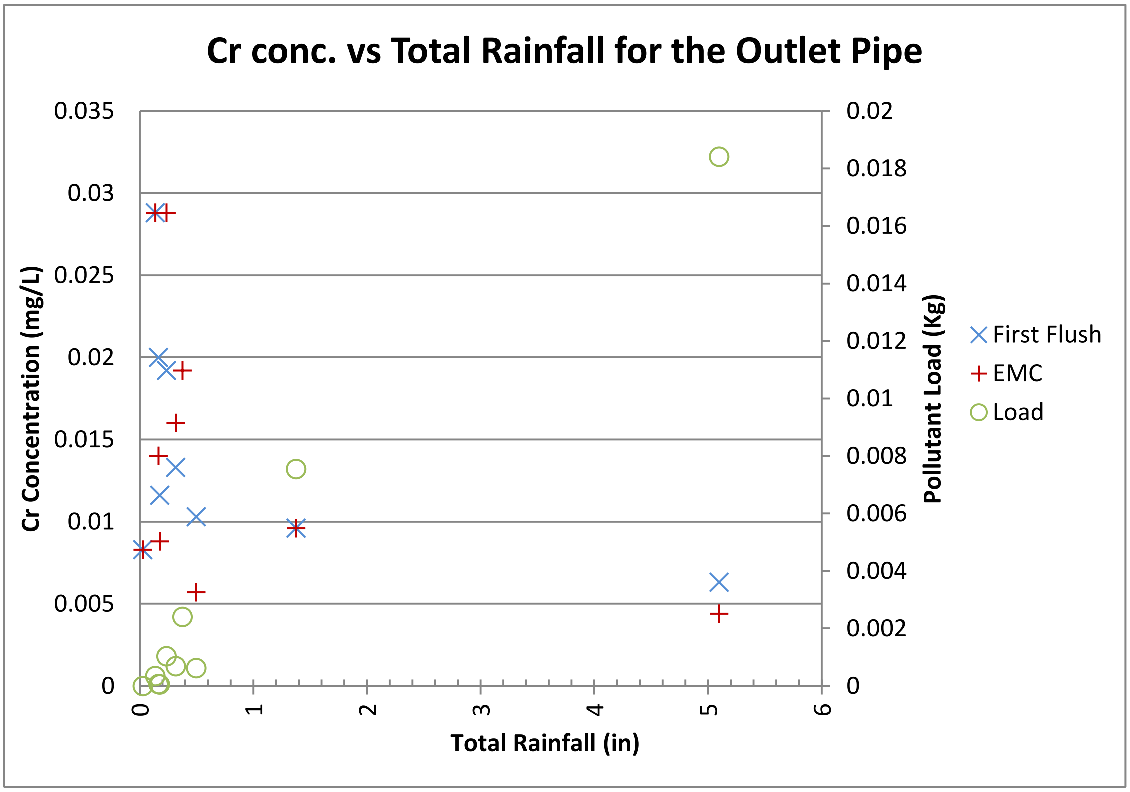

Figure 6.33 Cr concentration and total rainfall for the basin outlet pipe

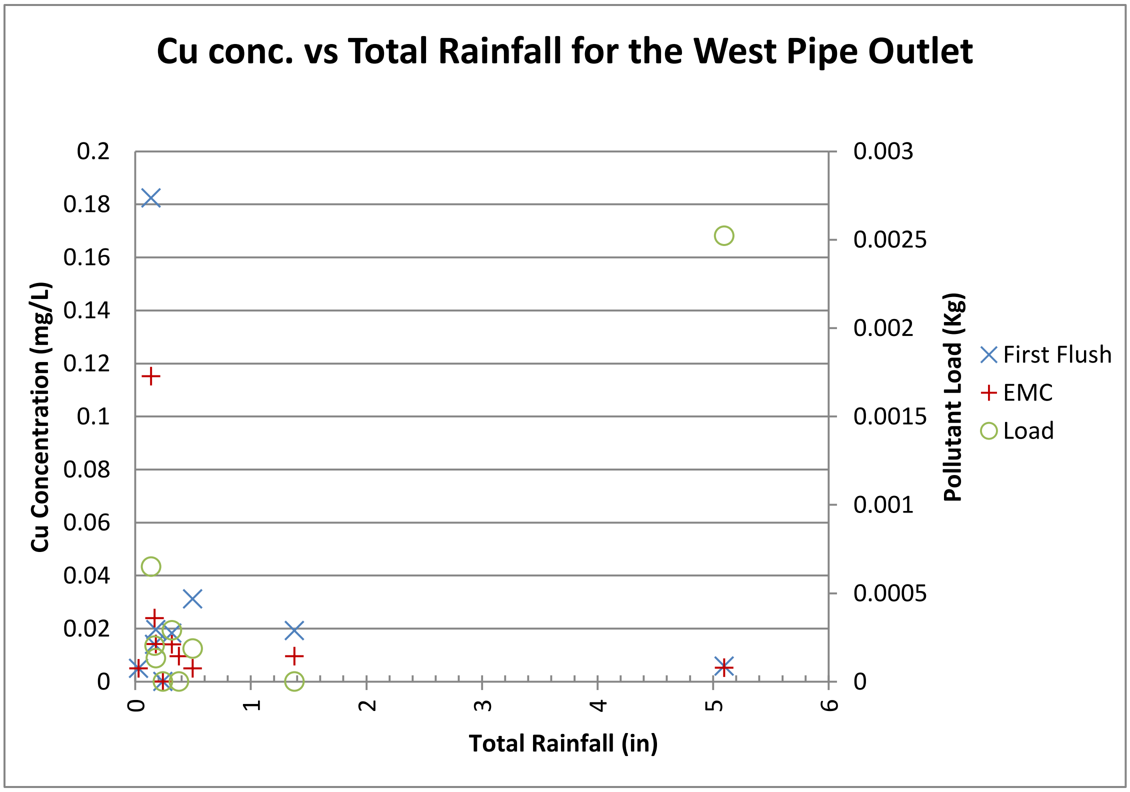

Figure 6.34 Cu concentration and total rainfall for the west pipe outlet

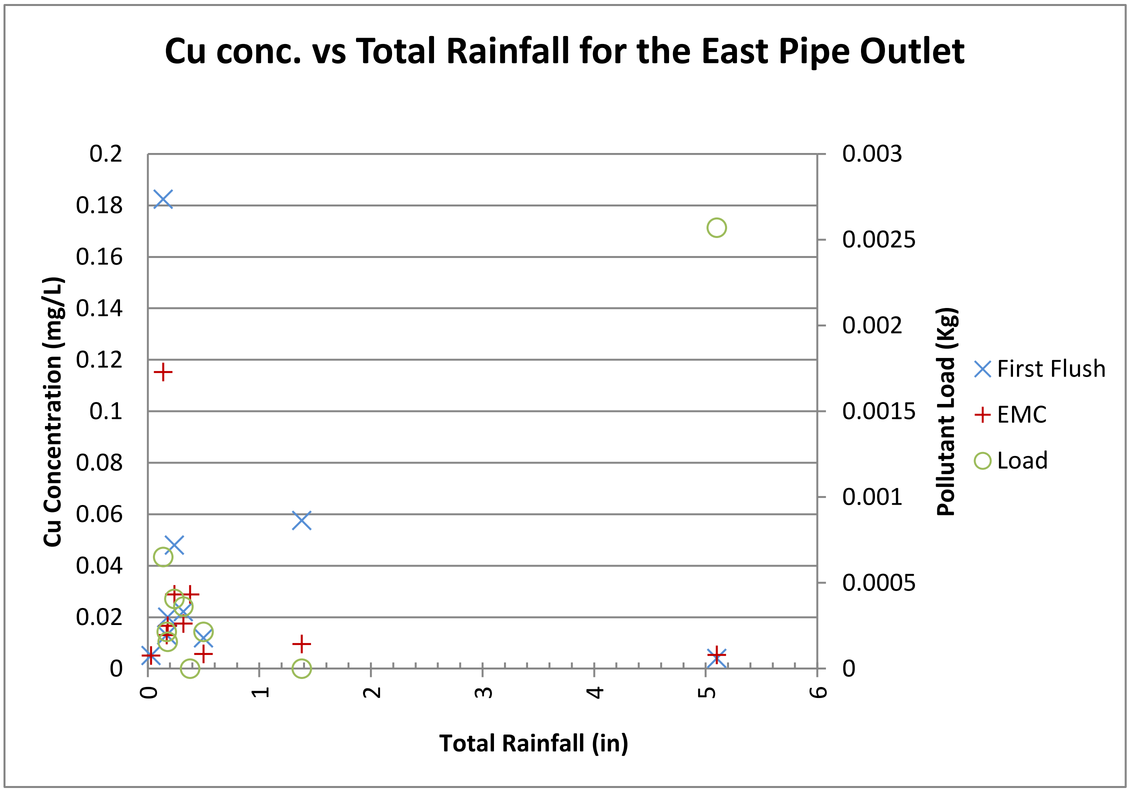

Figure 6.35 Cu concentration and total rainfall for the east pipe outlet

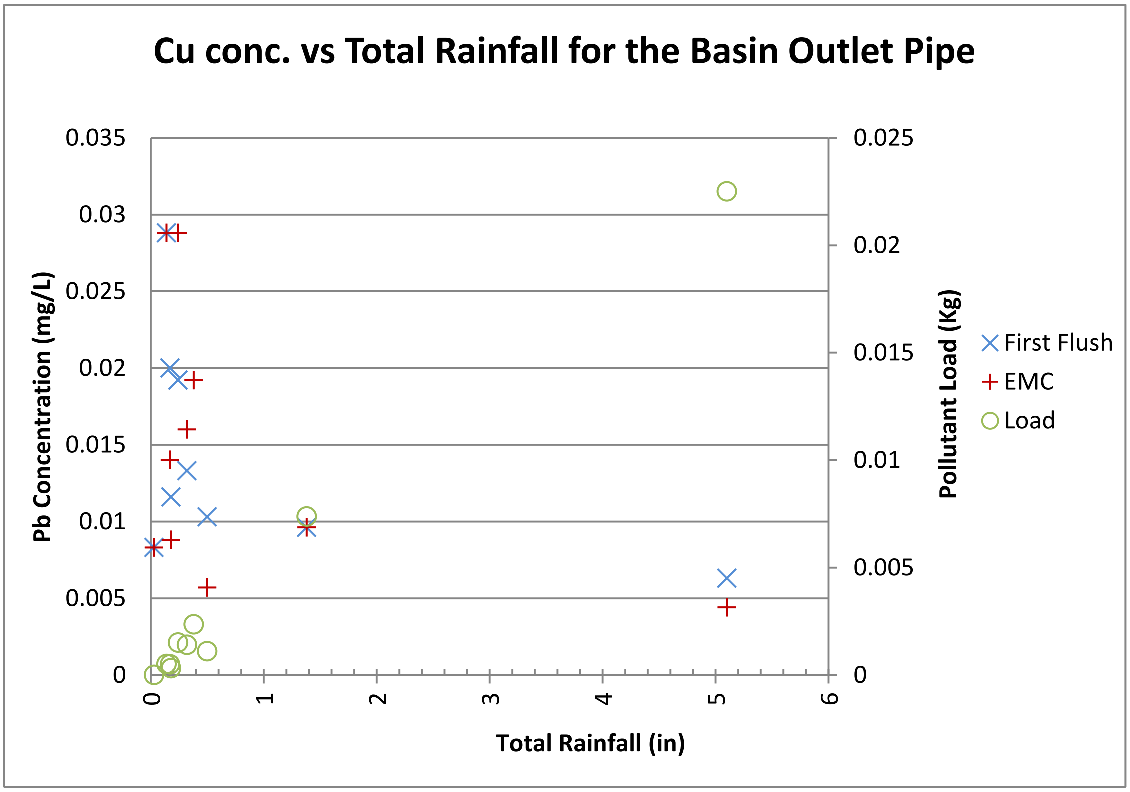

Figure 6.36 Cu concentration and total rainfall for the basin outlet pipe

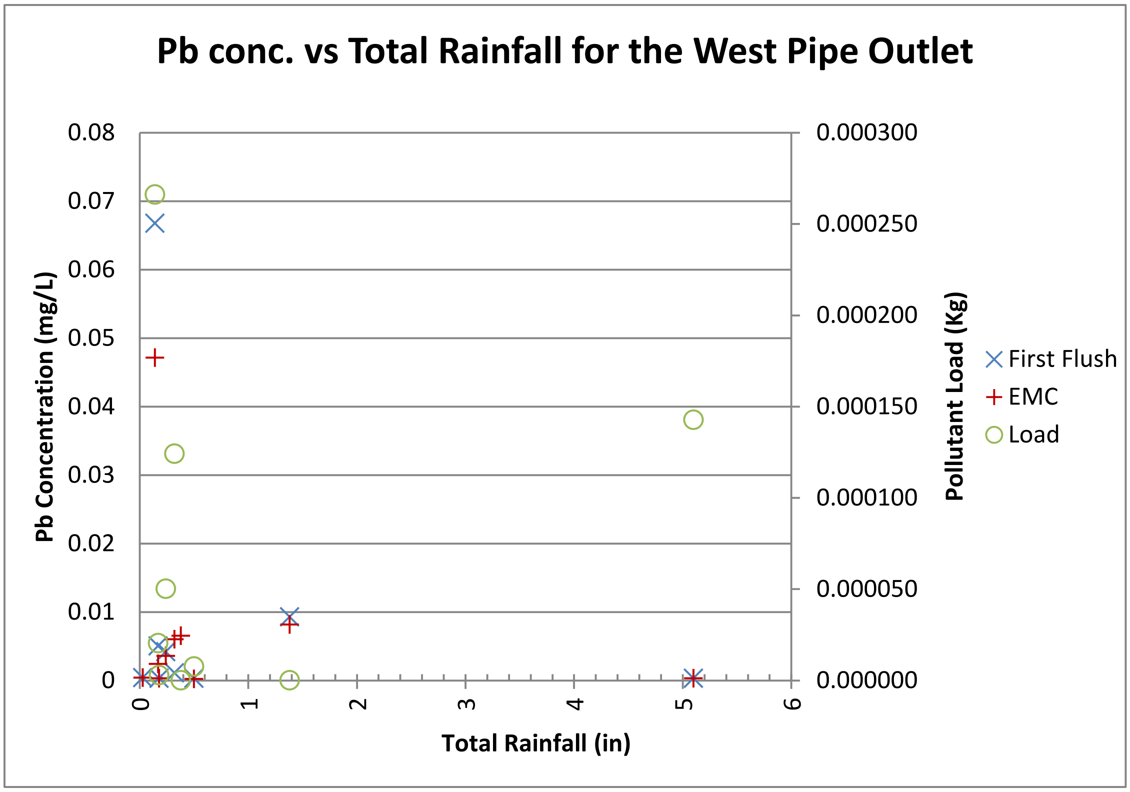

Figure 6.37 Pb concentration and total rainfall for the west pipe outlet

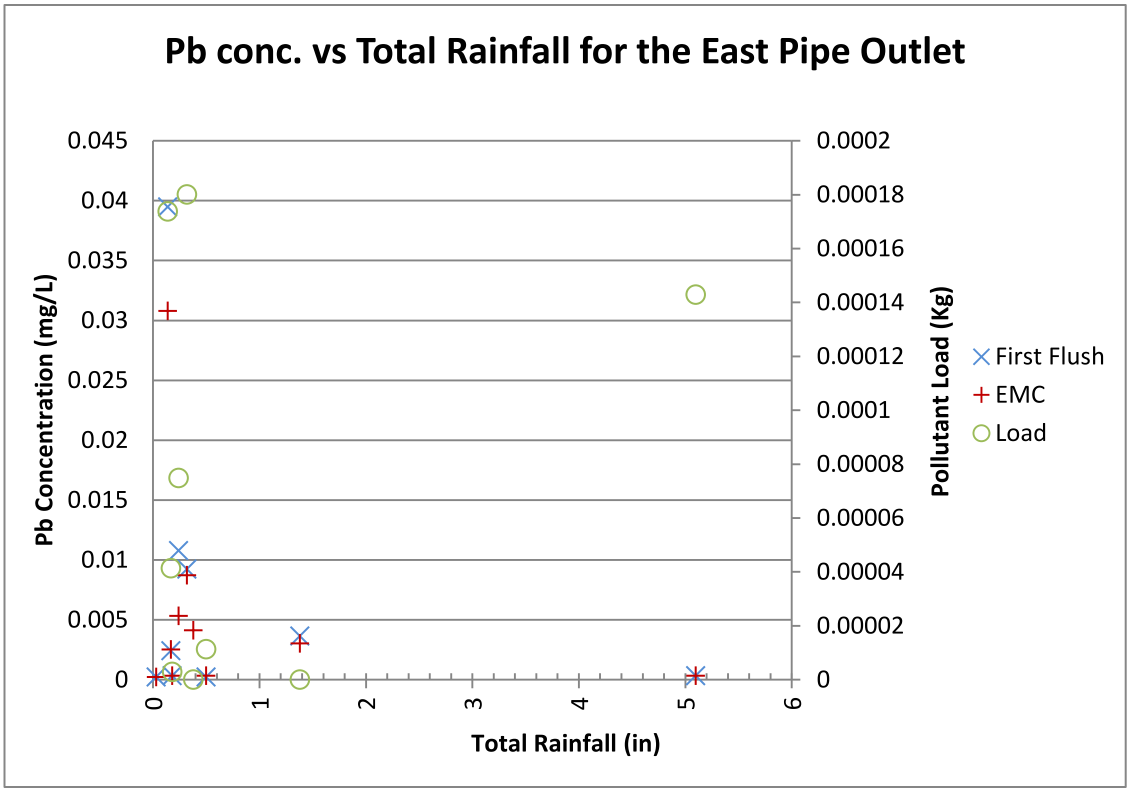

Figure 6.38 Pb concentration and total rainfall for the east pipe outlet

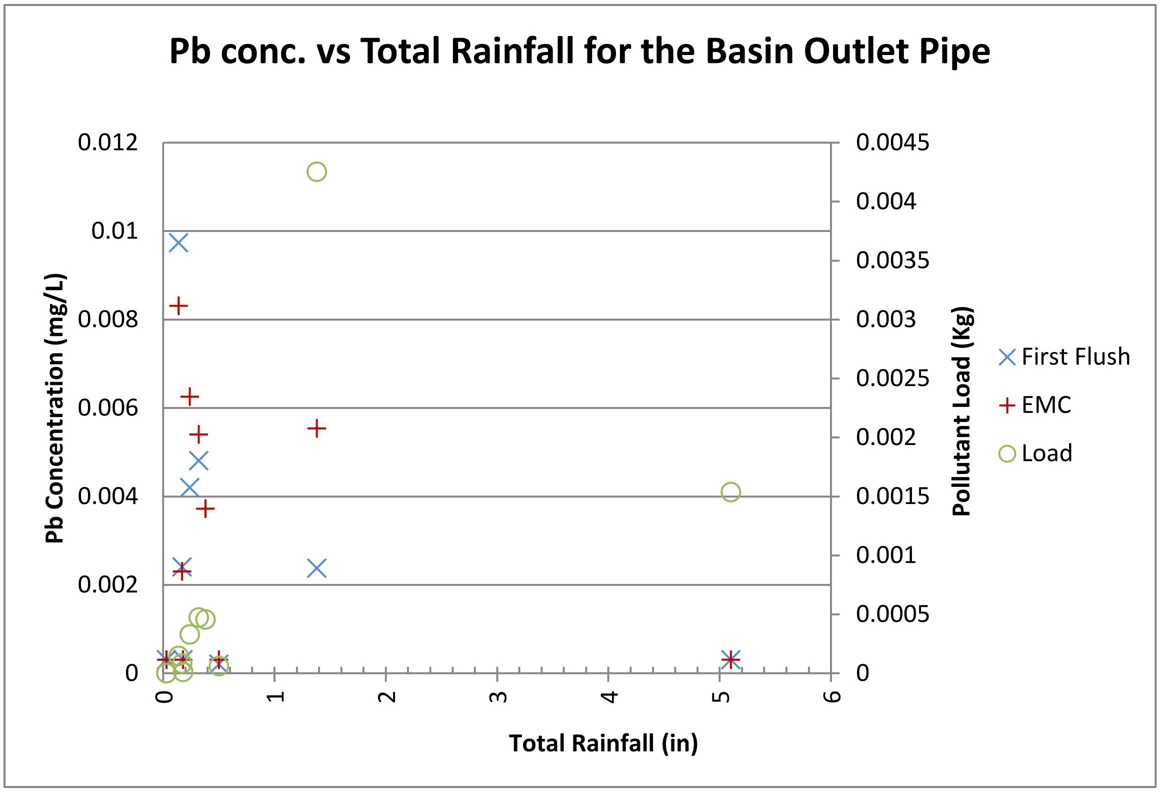

Figure 6.39 Pb concentration and total rainfall for the basin outlet pipe

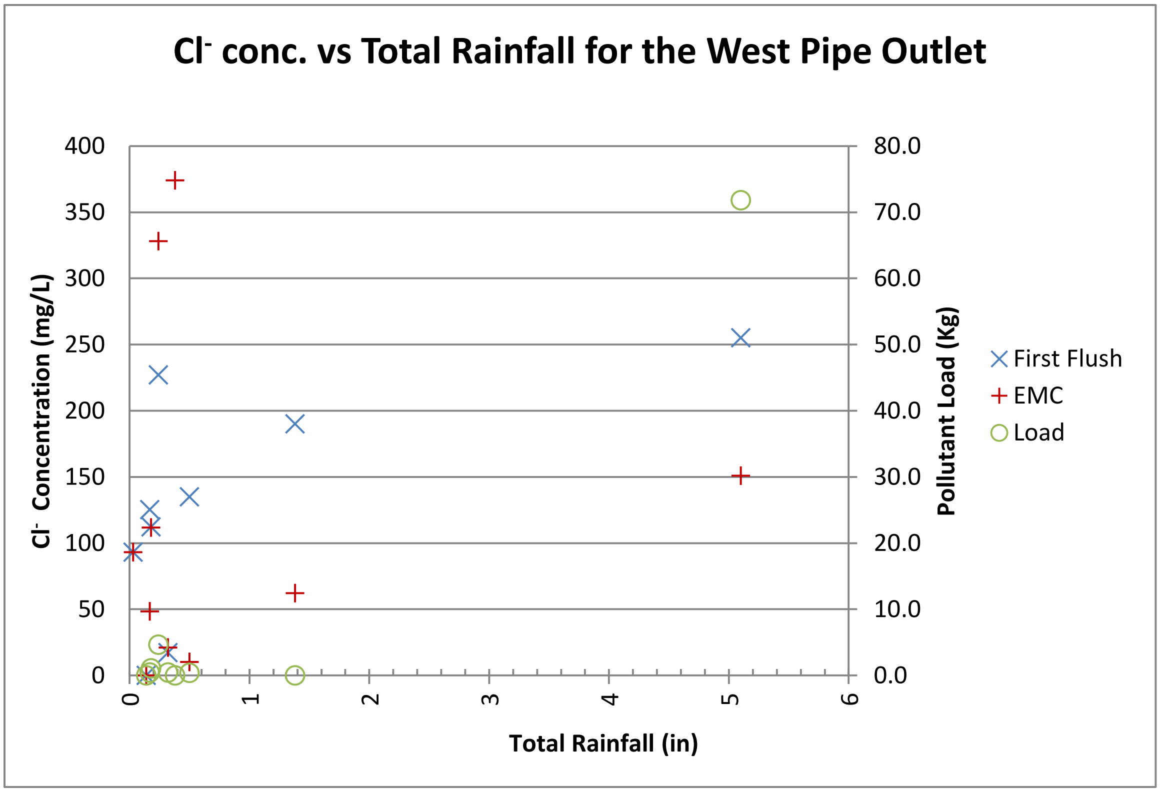

Figure 6.40 Cl^- concentration and total rainfall for the west pipe outlet

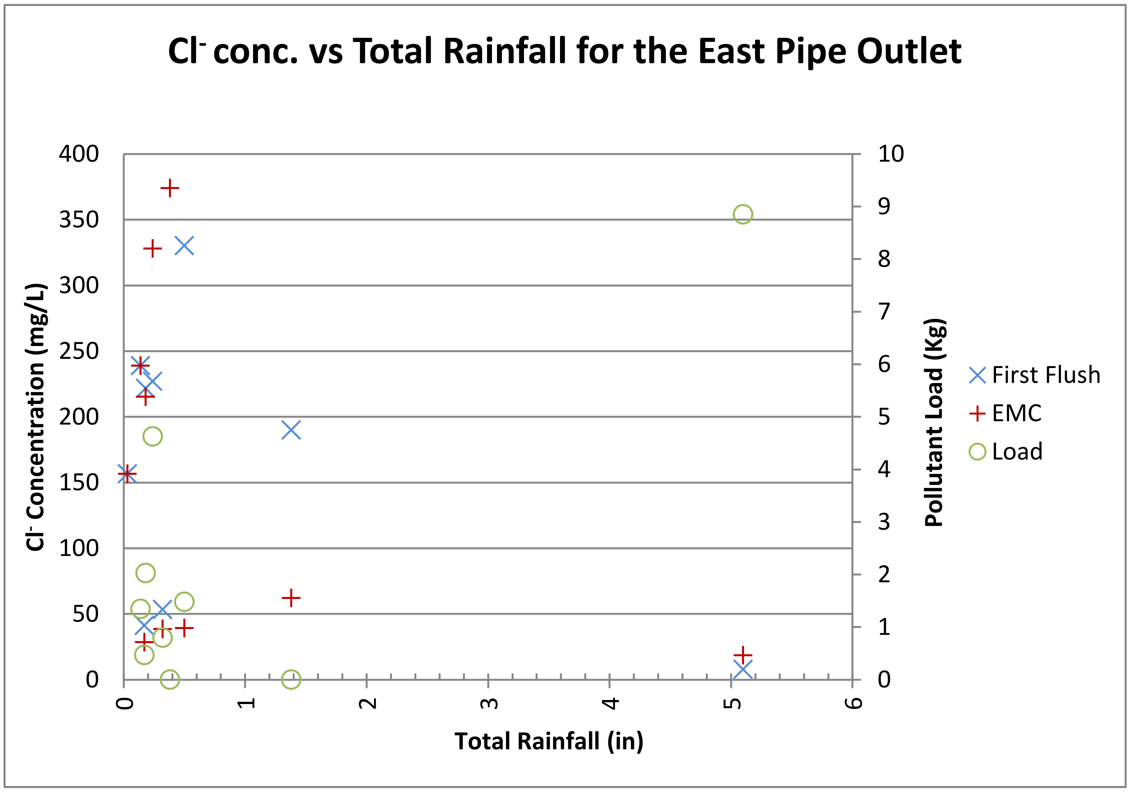

Figure 6.41 Cl$^-$ concentration and total rainfall for the east pipe outlet

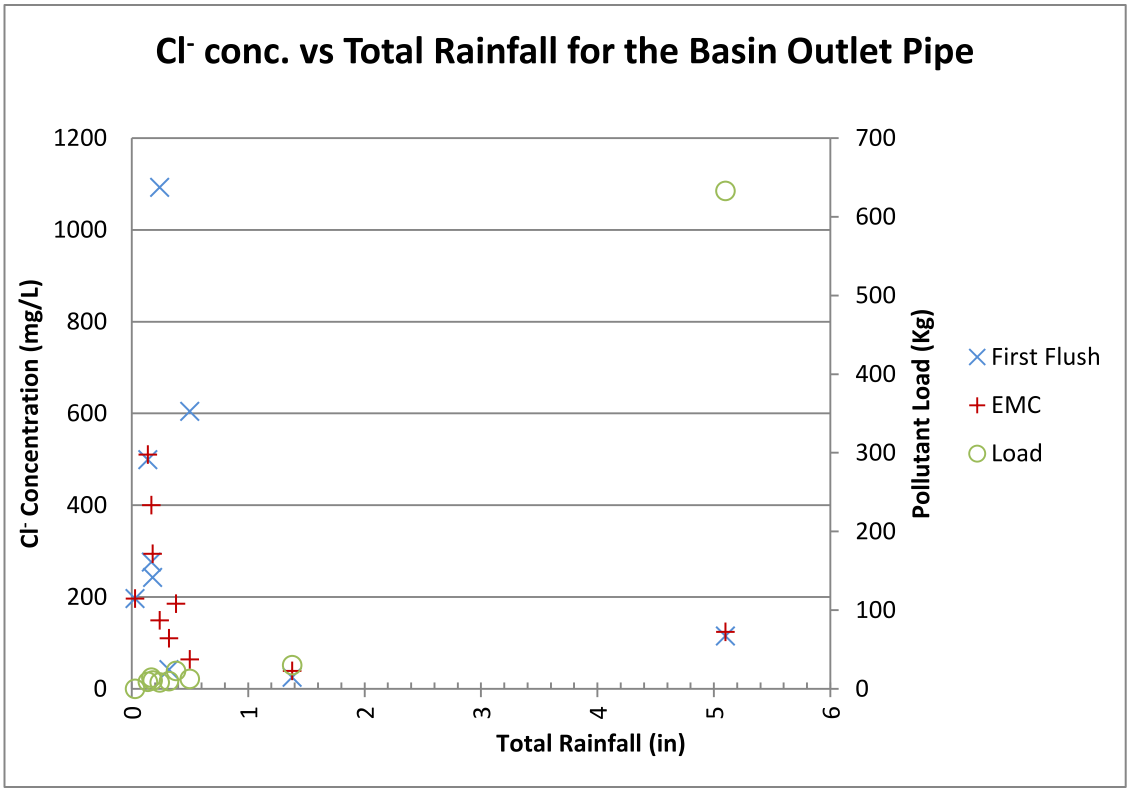

Figure 6.42 Cl$^-$ concentration and total rainfall for the basin outlet pipe

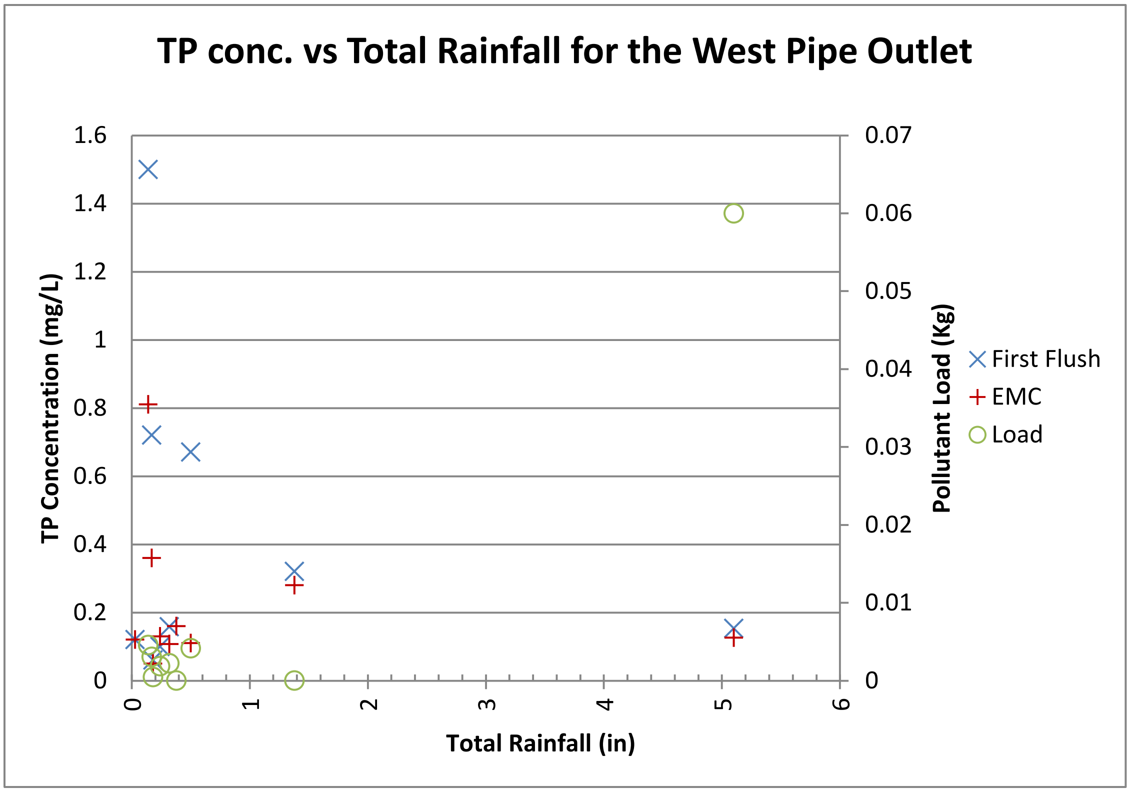

Figure 6.43 TP concentration and total rainfall for the west pipe outlet

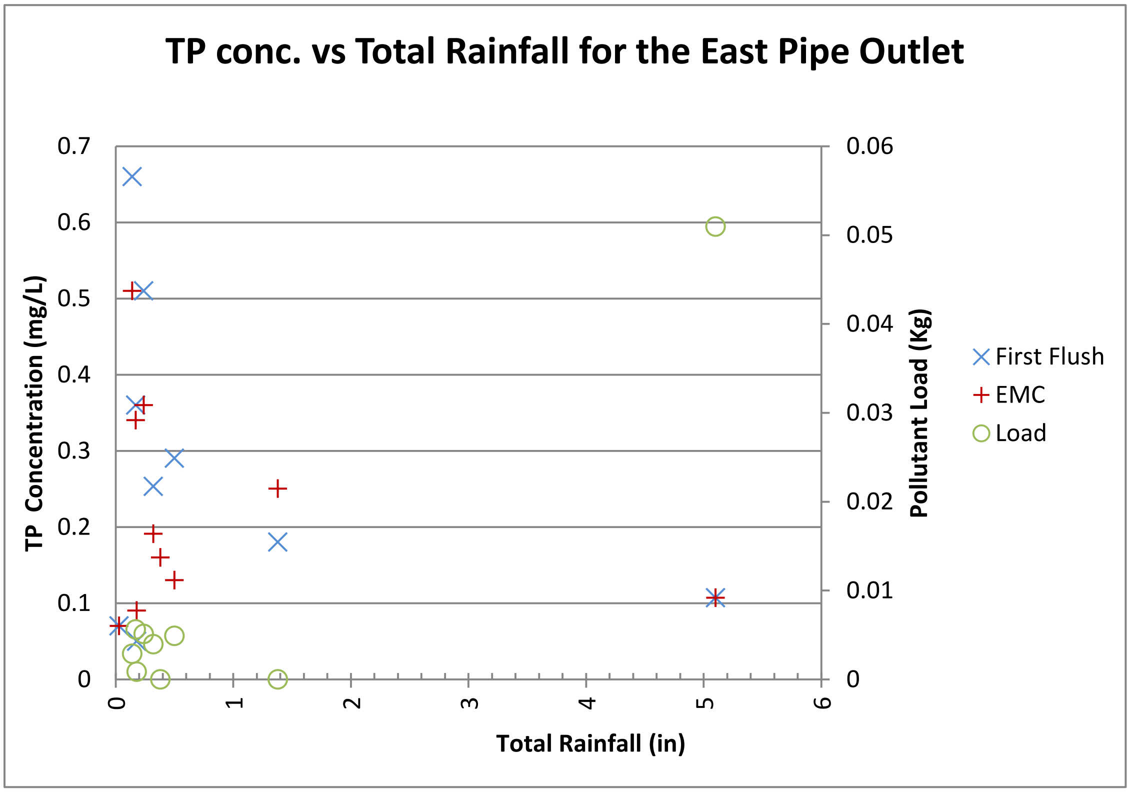

Figure 6.44 TP concentration and total rainfall for the east pipe outlet

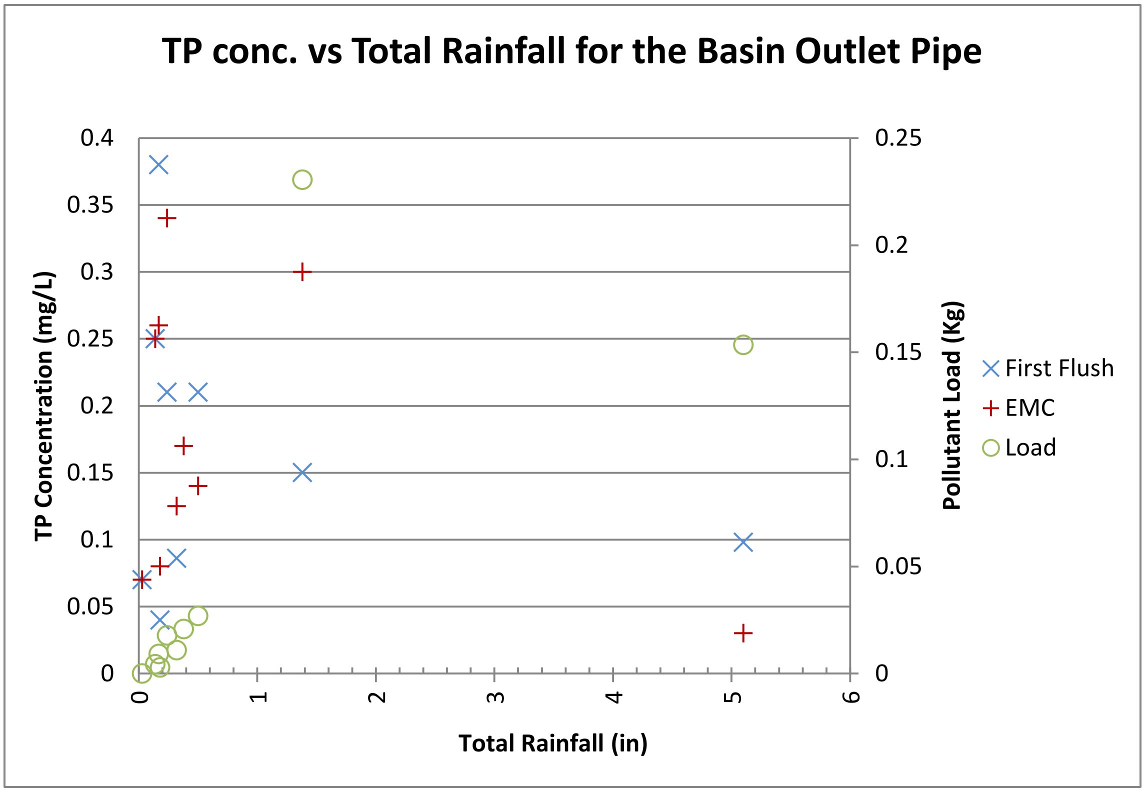

Figure 6.45 TP concentration and total rainfall for the basin outlet pipe

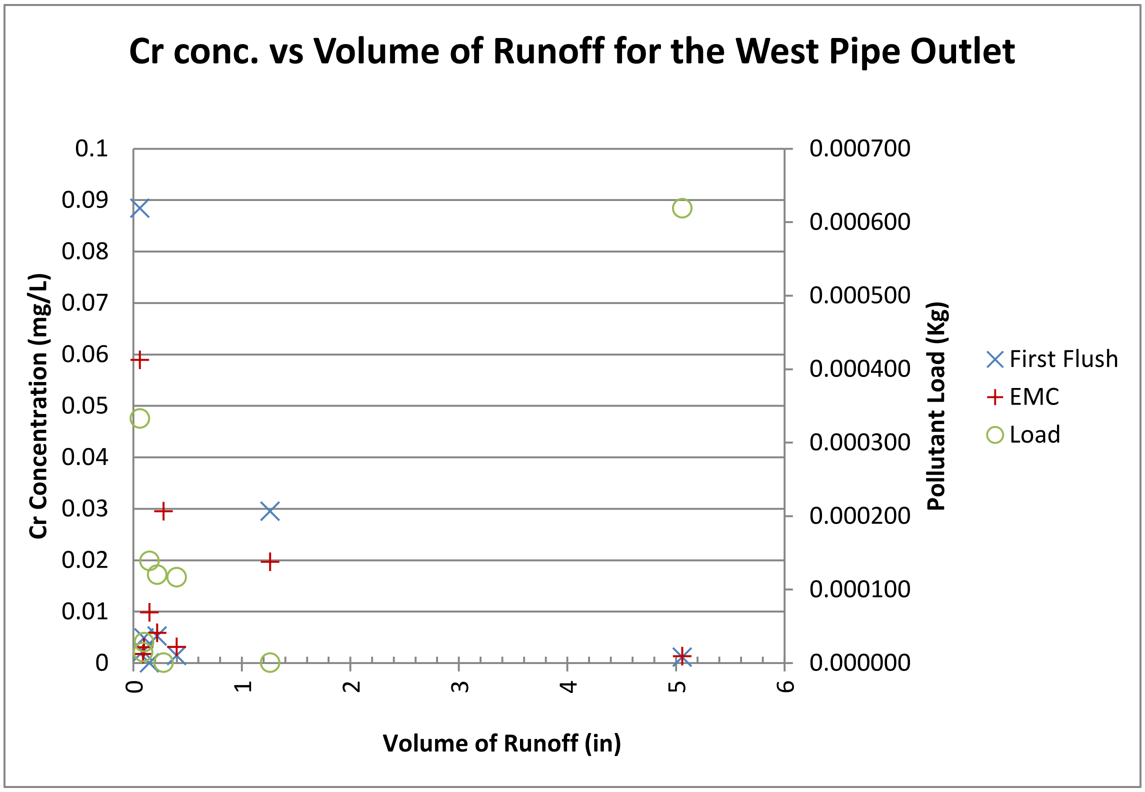

Figure 6.46 Cr concentration and volume of runoff for the west pipe outlet

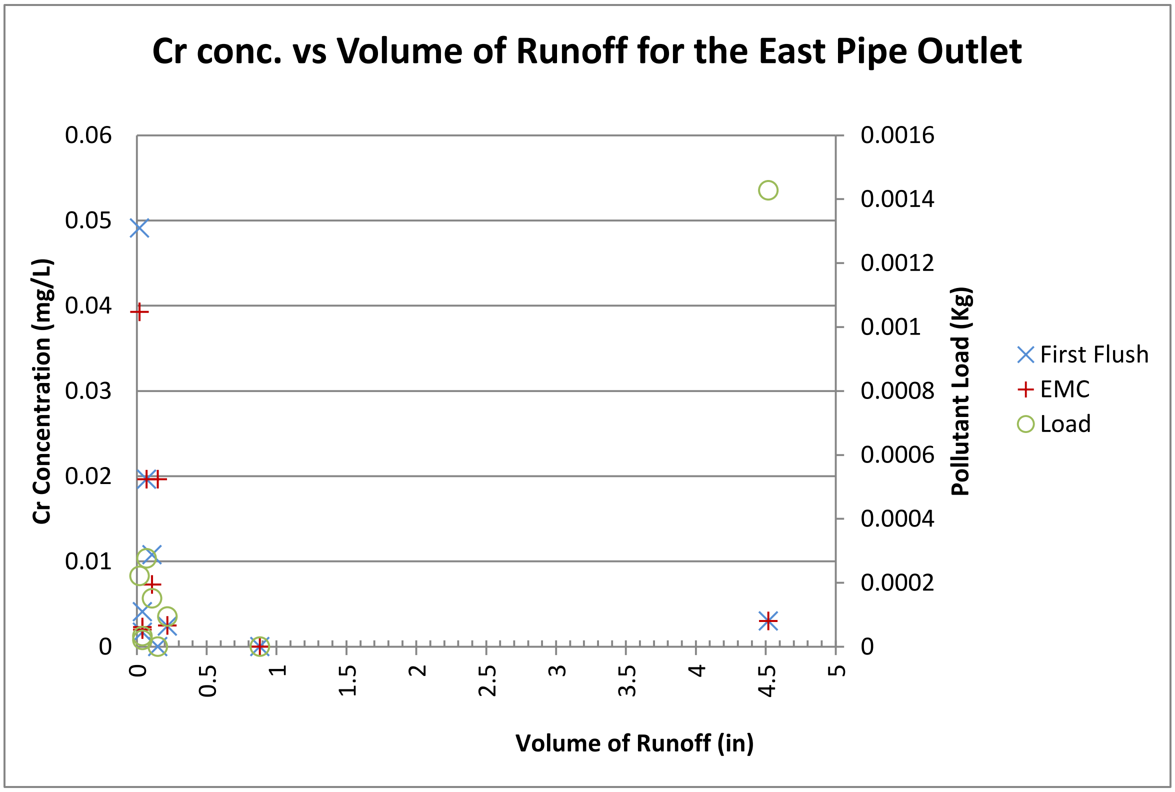

Figure 6.47 Cr concentration and volume of runoff for the east pipe outlet

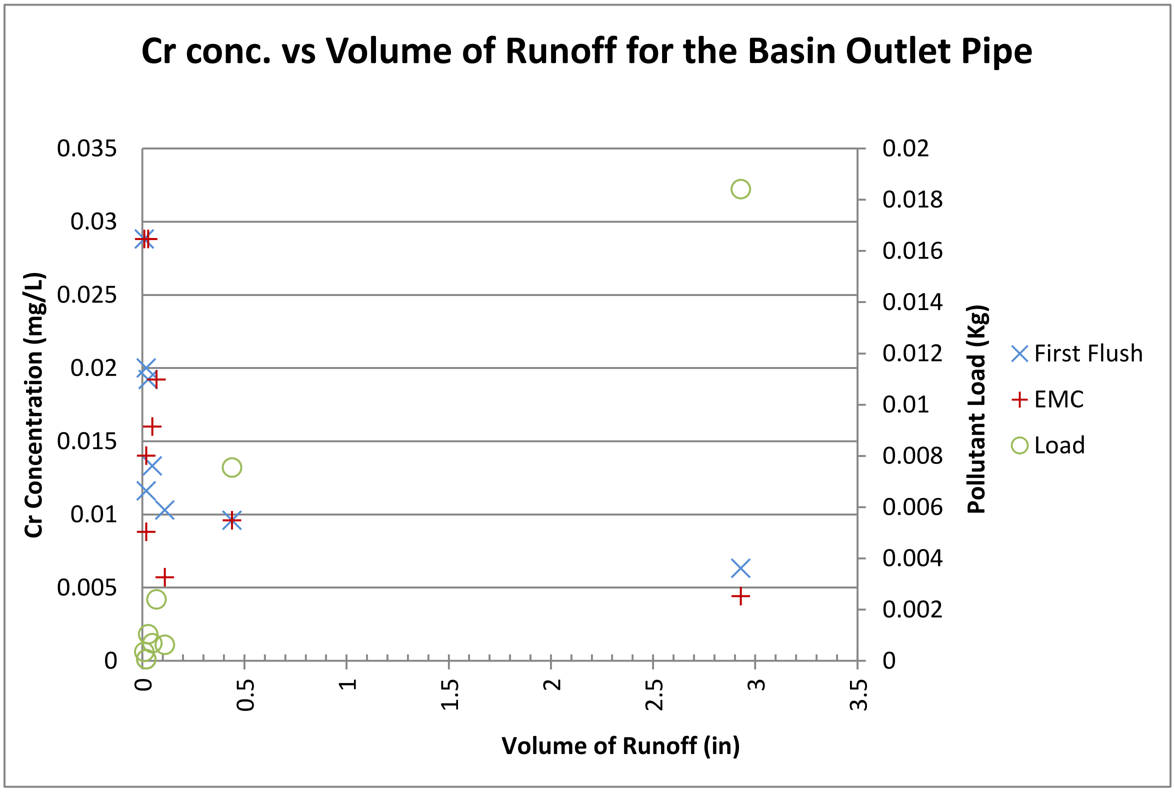

Figure 6.48 Cr concentration and volume of runoff for the basin outlet pipe

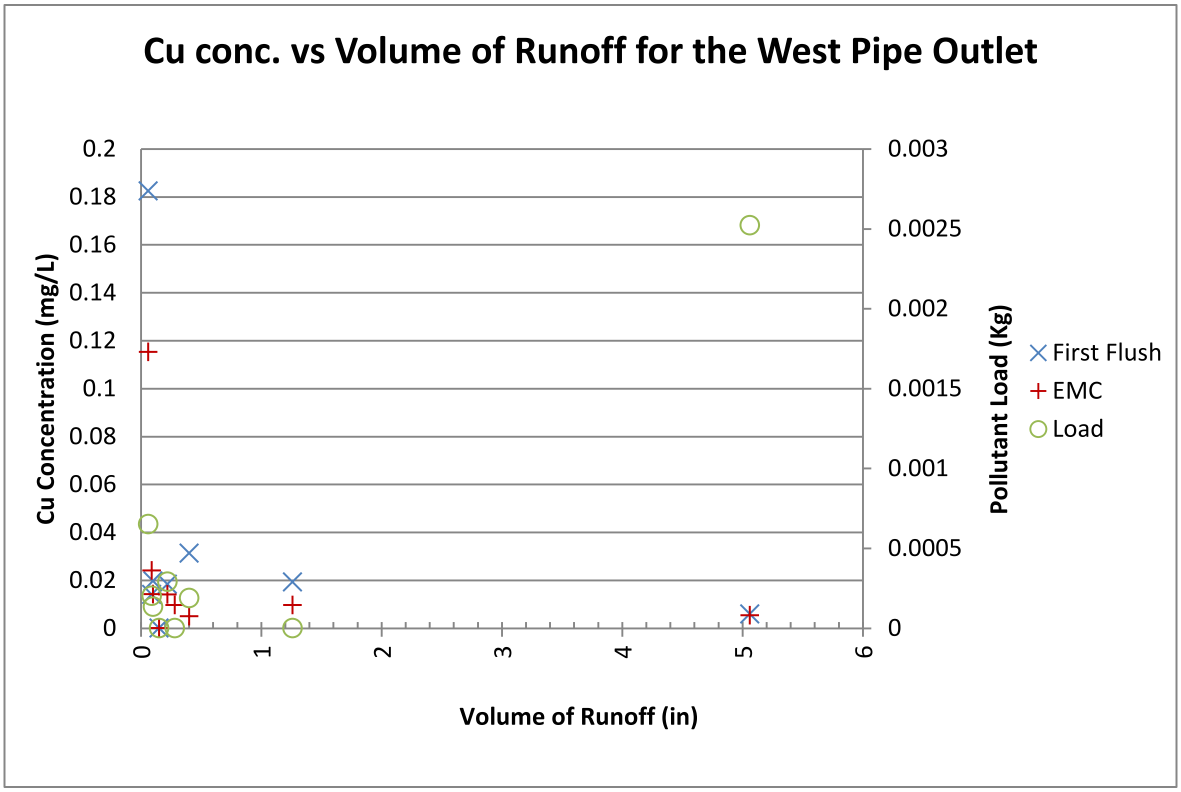

Figure 6.49 Cu concentration and volume of runoff for the west pipe outlet

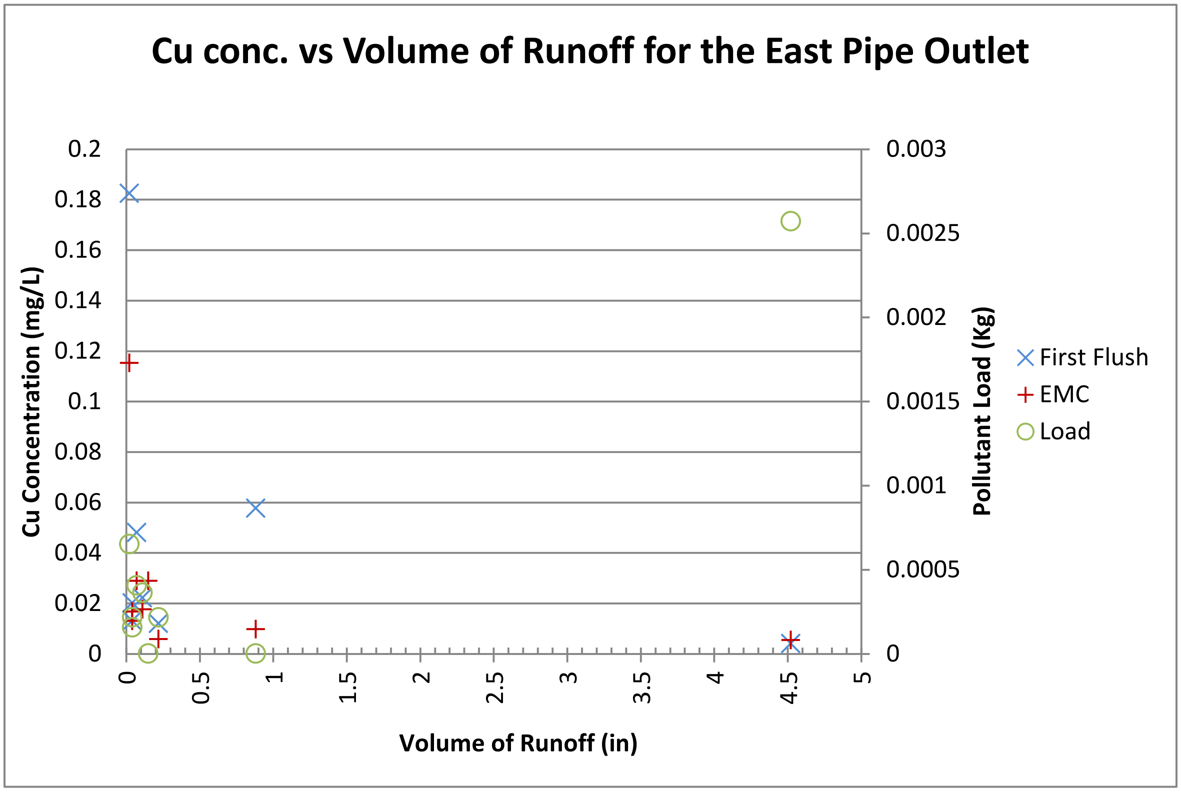

Figure 6.50 Cu concentration and volume of runoff for the east pipe outlet

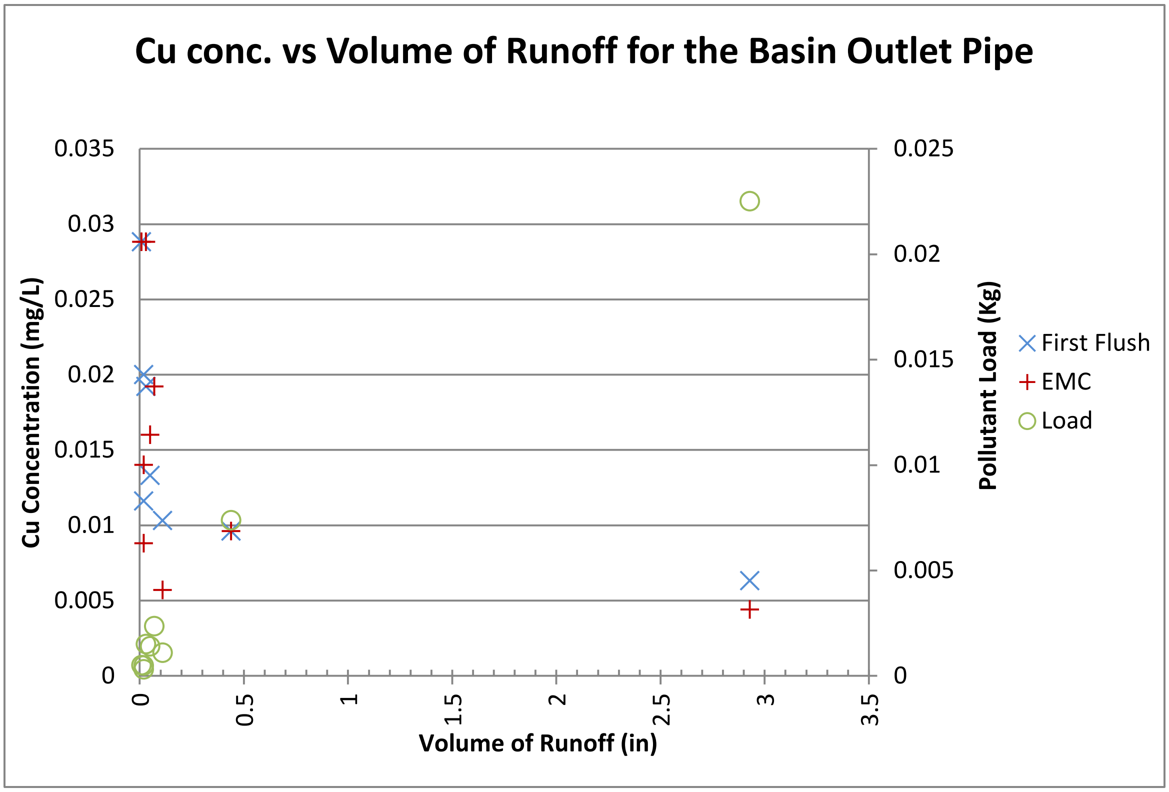

Figure 6.51 Cu concentration and volume of runoff for the basin outlet pipe

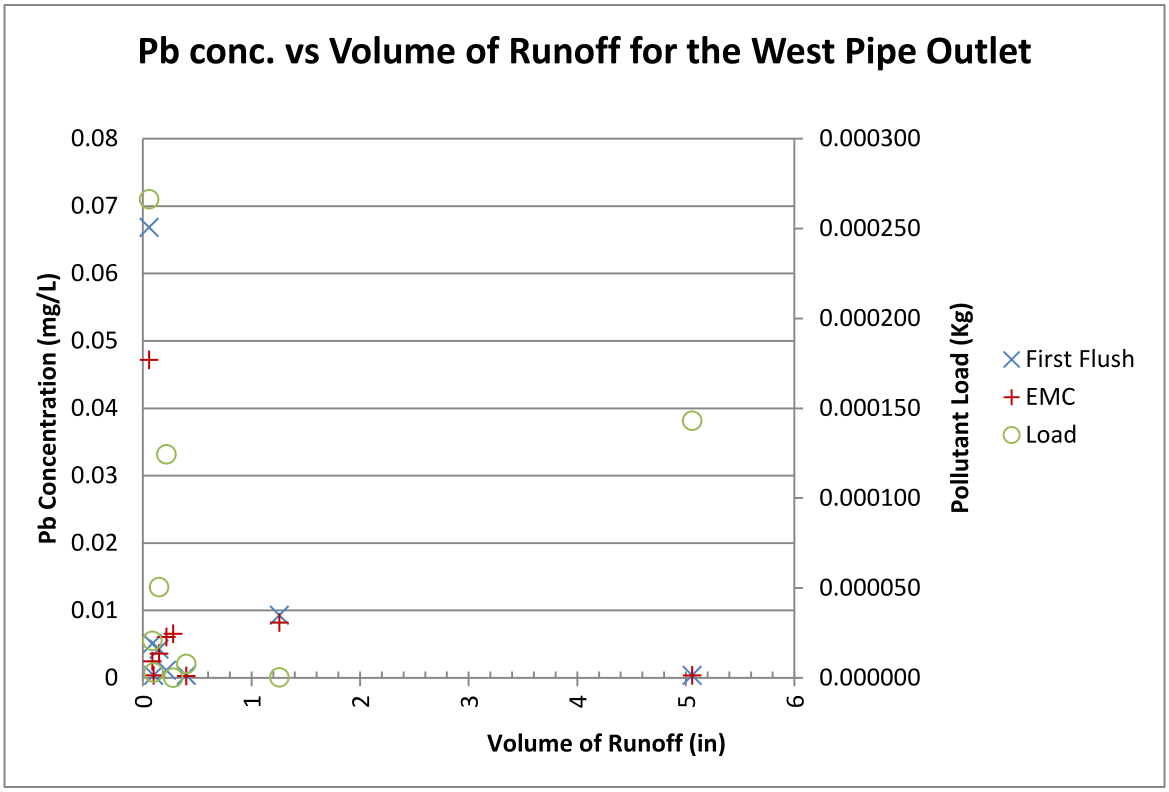

Figure 6.52 Pb concentration and volume of runoff for the west pipe outlet

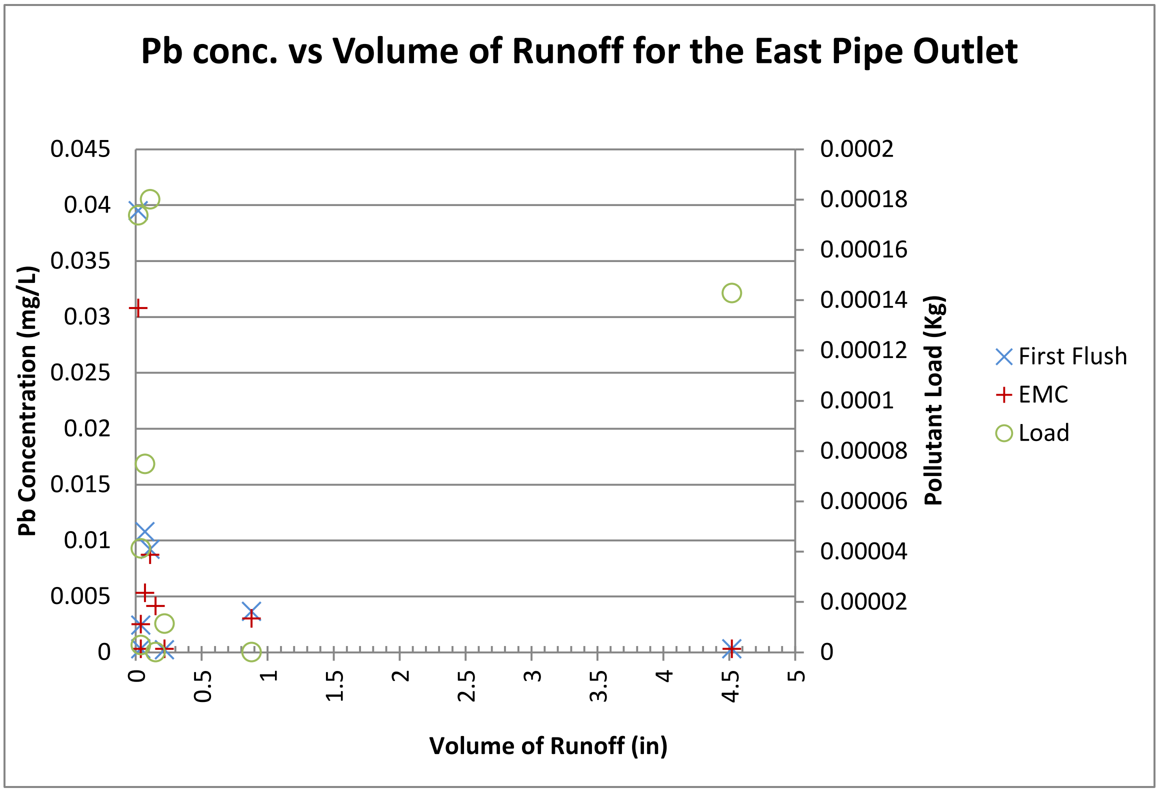

Figure 6.53 Pb concentration and volume of runoff for the east pipe outlet

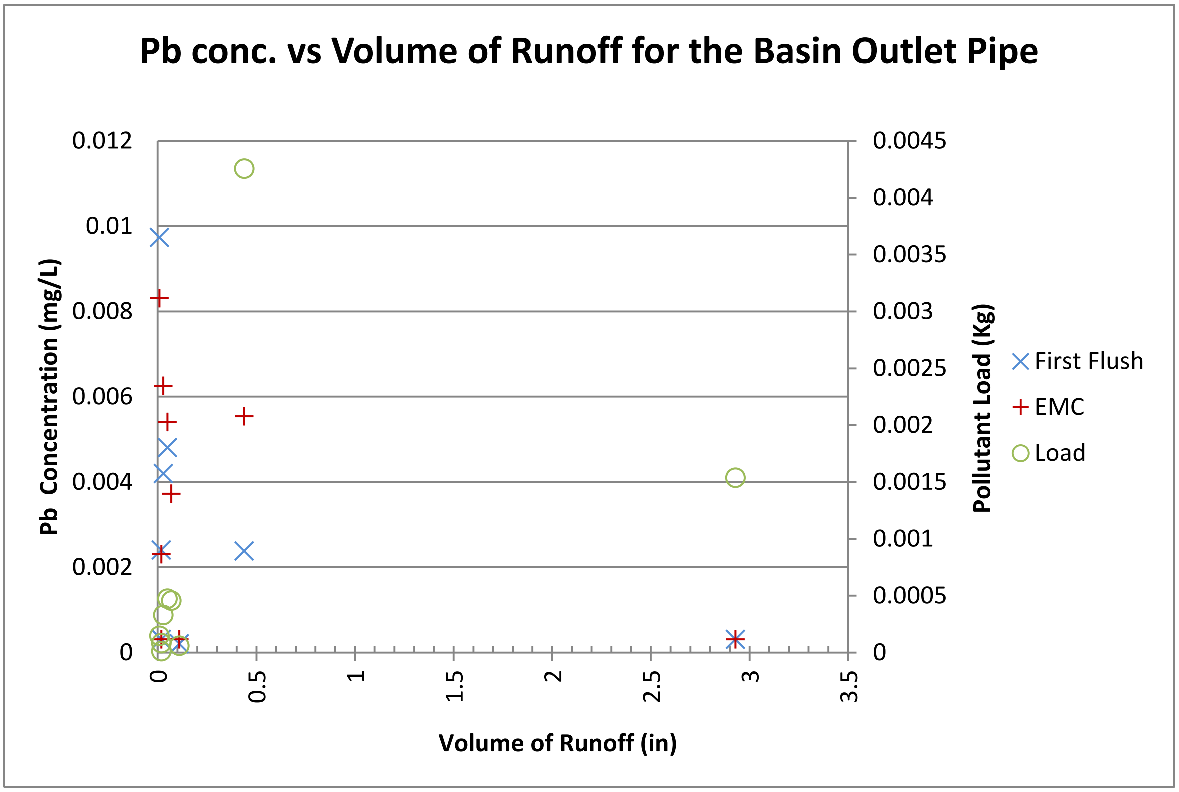

Figure 6.54 Pb concentration and volume of runoff for the basin outlet pipe

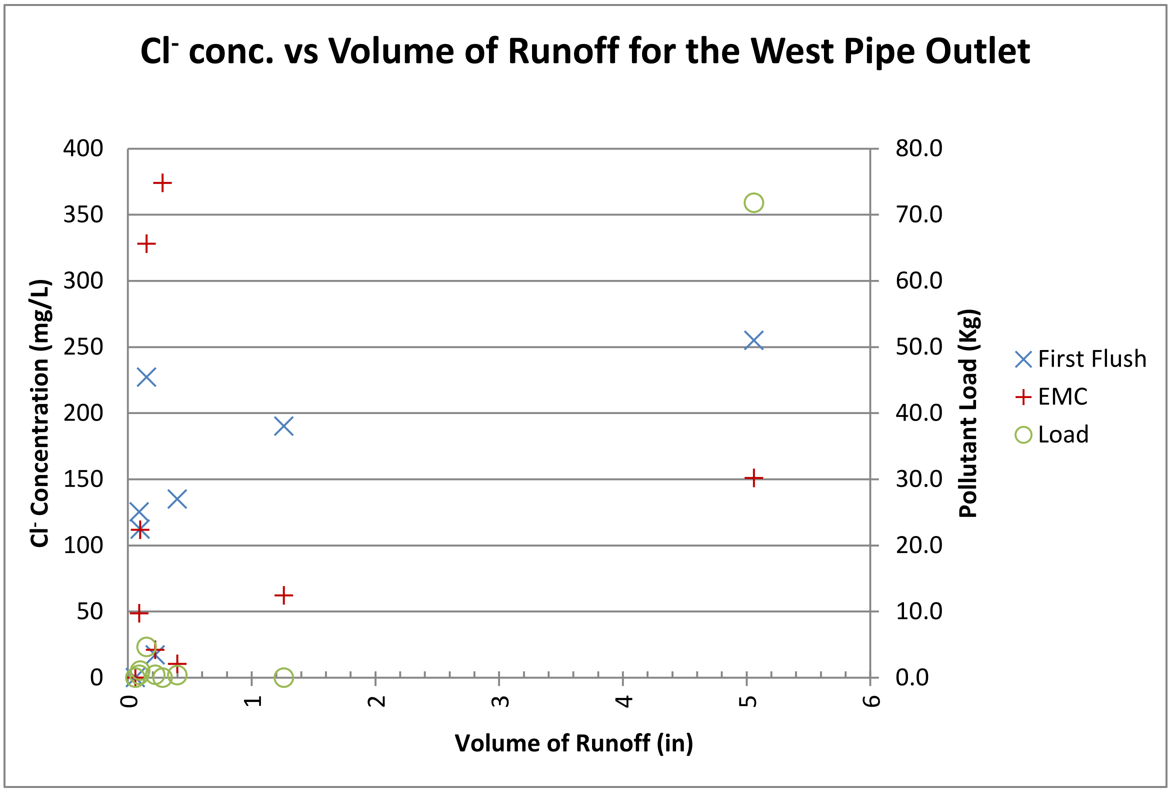

Figure 6.55 Cl$^-$ concentration and volume of runoff for the west pipe outlet

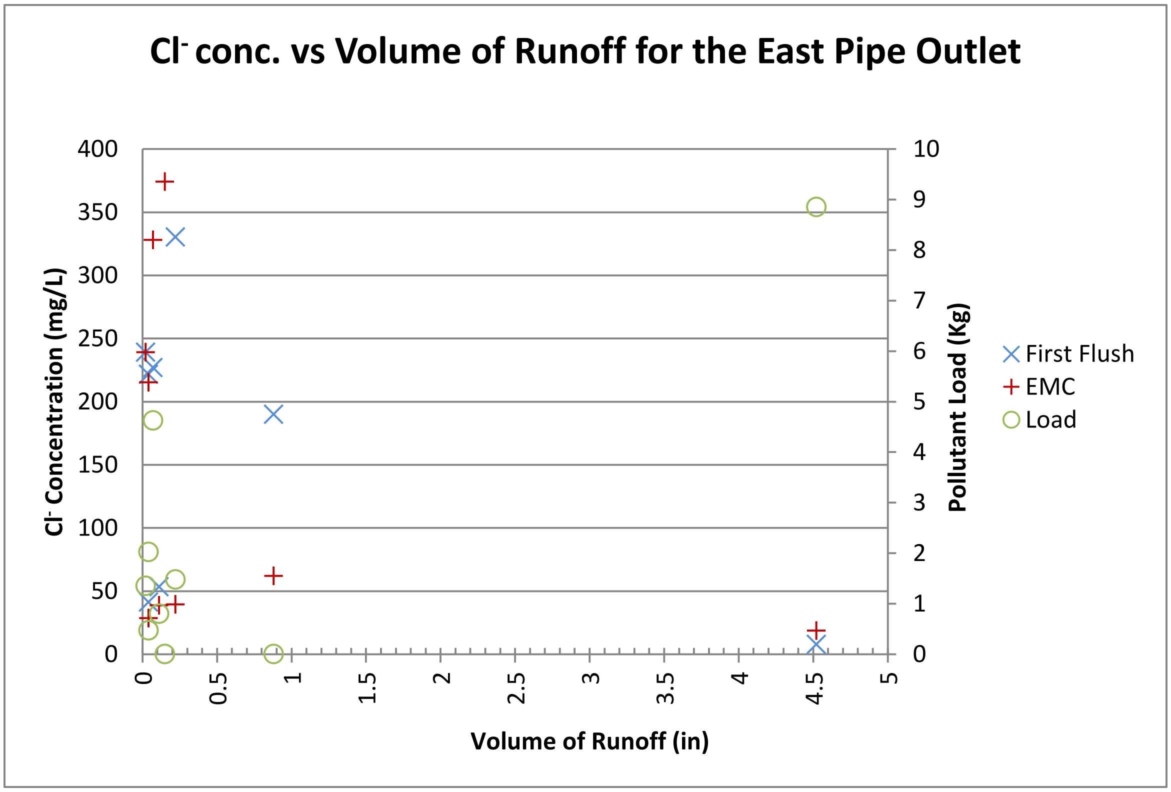

Figure 6.56 Cl$^-$ concentration and volume of runoff for the east pipe outlet

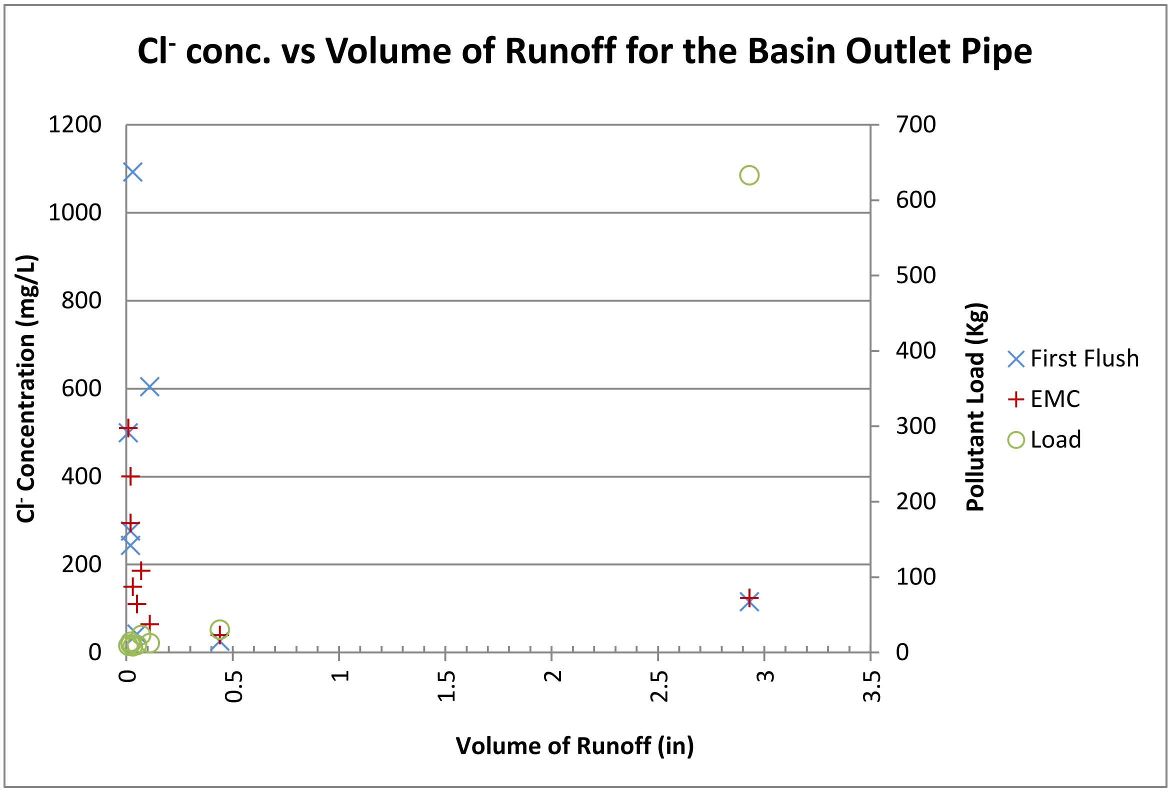

Figure 6.57 Cl$^-$ concentration and volume of runoff for the basin outlet pipe

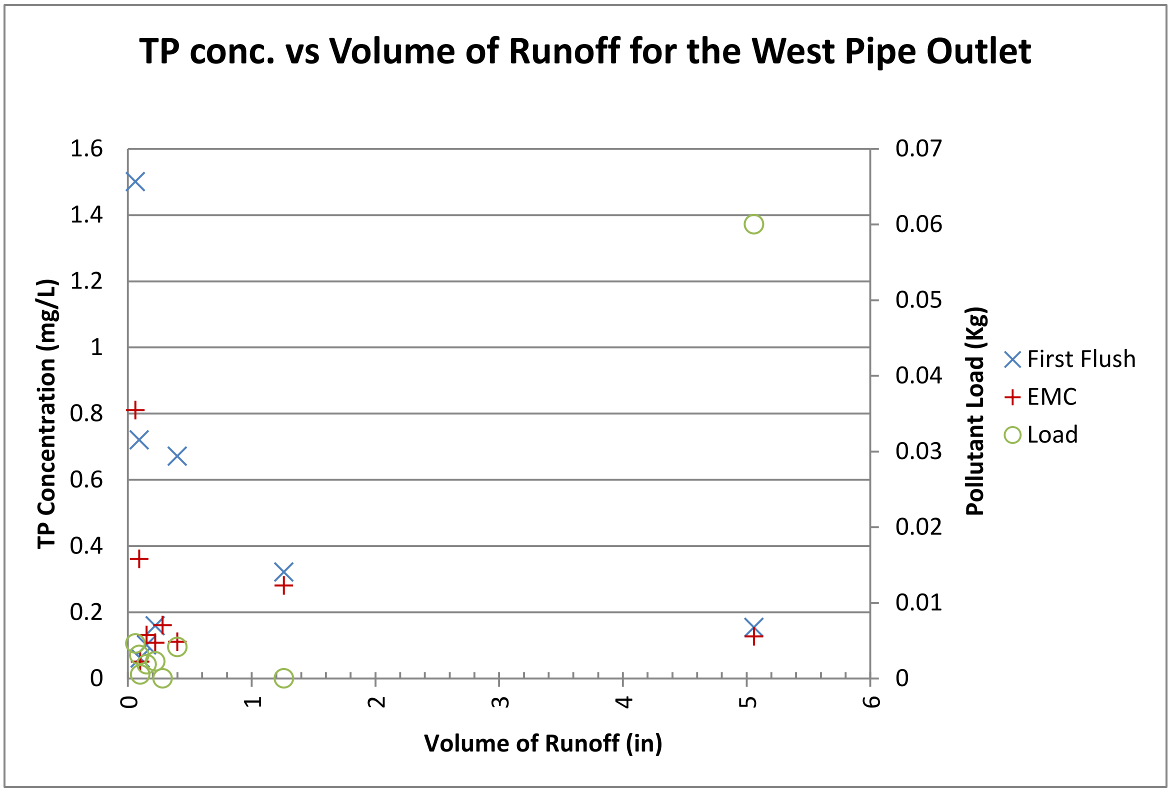

Figure 6.58 TP concentration and volume of runoff for the west pipe outlet

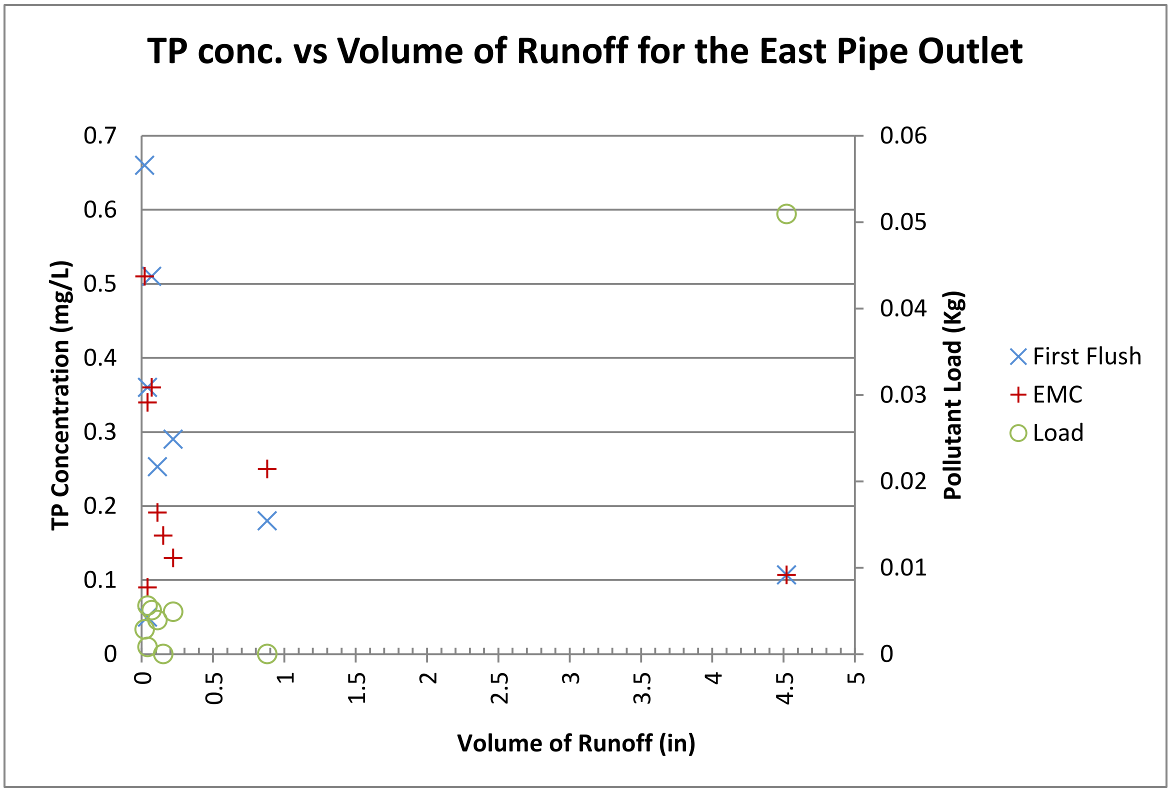

Figure 6.59 TP concentration and volume of runoff for the east pipe outlet

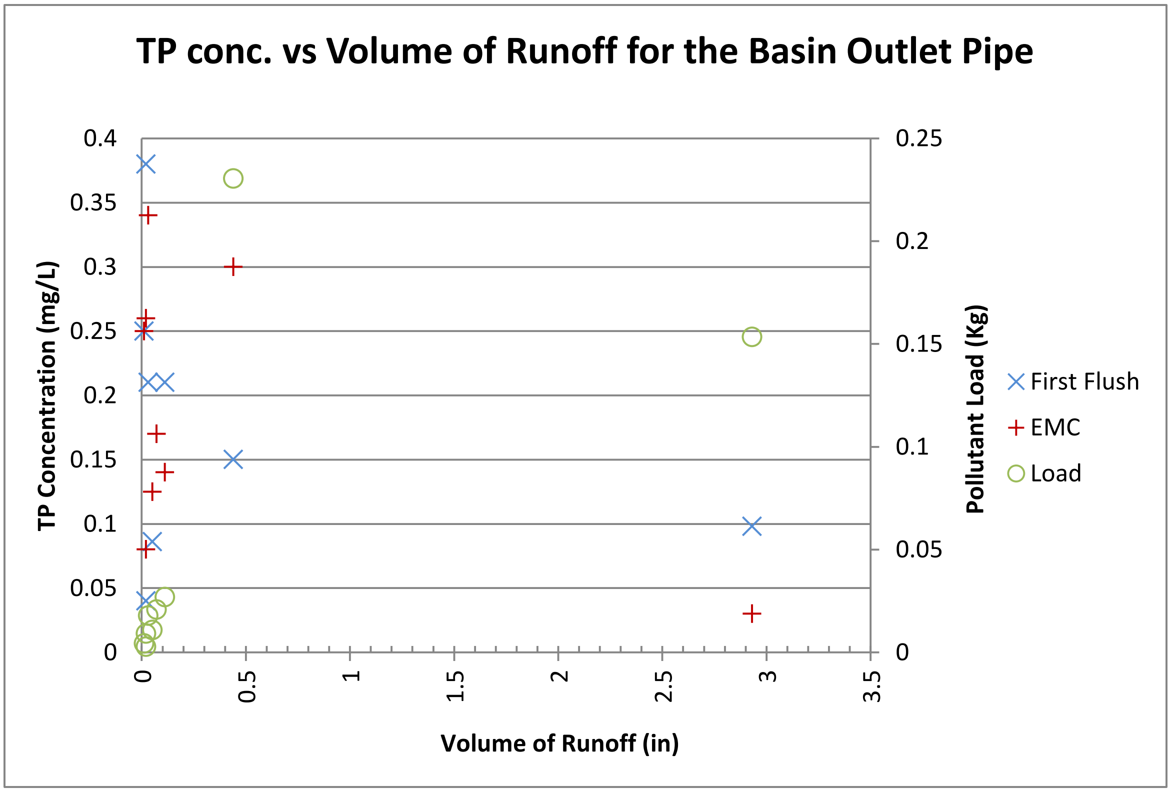

Figure 6.60 TP concentration and volume of runoff for the basin outlet pipe

6.2 Comparison with Similar Studies

As mentioned in the introduction, highway runoff pollution has been a study subject for the past 30 yrs. Table 6.1 shows a comparison between the EMC from the current study and other studies done in California (Kayhanian et al. 2007), North Carolina (Wu et al. 1998), Texas (Barret et al. 1998), and a summary of several studies done by the Federal Highway Administration (FHWA) (Driscoll et al. 1990b).

Kayhanian et al. (2007) obtained samples from 34 highway sites in California, covering different annual average daily traffic levels and environmental conditions. Table 6.2 shows the characteristics of the sampled sites by Kayhanian et al.

Wu et al. (1998) studied three highway sites located inside the city of Charlotte. The site used for comparison purposes is a section of the W.T. Harries Blvd. that carries an average traffic of 25,000 vehicles/day and is a major artery around the north and east sites of the city of Charlotte, extending from I-77 on the north to U.S. 74 on the southeast. The drainage area was 0.37 acres and the surrounding land use was 69% woods, 24% residential, and 6% heavy commercial.

Barret et al. (1998) monitored three sites along the MoPac Expressway in the Austin, Texas area. The site selected for comparison with this study was the MoPac at 35th Street which had an average daily traffic of 60,000 vehicles per day. The land use of the area adjacent to the highway was mixed residential and commercial and the catchment area was 1.32 acres.

Driscoll et al. (1990) compiled results from 31 sites around the United States. Table 6.3 show the location and characteristics of the sites compiled by Driscoll et al.

Table 6.1 Comparison with similar studies

Analyte		NDOR Study						Wu et al.[a]			Kayhanian et al.[b]			Barret et al.[c]			Driscoll et al.[d]		
		West Pipe (EMC)			East pipe (EMC)			Monitoring Site I (EMC)			Range	Median	Mean	35th Street (EMC)			National Highway Runoff Report		
		Range	Median	Mean	Range	Median	Mean	Range	Median	Mean				Range	Median	Mean	Range	Median	Mean
Cu	(µg/L)	5 - 115.2	30	48	5.1 - 76.8	17	26	9.0 - 52	15	24.2	1.1 - 130	14.9	10.2	2.0 – 120	34	38	5 - 155	52	39
Cd	(µg/L)	-	3	3	-	2.8	2.8	<DL	<DL	<DL	0.2 - 8.4	0.24	0.13	-	-	-	-	-	-
Cr	(µg/L)	1.3 - 58.9	30	40	1.1 - 39.3	20	27	5.0 - 20	6.5	8.1	1.0 - 23	3.3	2.2	-	-	-	-	-	-
Pb	(µg/L)	0.2 - 47.1	7	19	0.2 - 30.8	6	14	7.0 - 56	15	21	1.0 - 480	7.6	1.2	7 – 440	50	99	11 - 1457	525	234
Fe	(µg/L)	2.8 - 125	40	50	2.9 – 106	3	4.5	-	-	-	32 - 3310	378	150	300 – 10000	2606	3537	-	-	-
Ni	(µg/L)	0.6 - 29.9	N.D.	10	0.6 – 20.0	N.D.	9	9.0 - 17	9	8.1	1.1 - 40	4.9	3.4	-	-	-	-	-	-
Zn	(µg/L)	4.5 – 82	11.5	27.3	0.7 - 66	12.5	23.5	-	-	-	3 - 1017	68.8	40.4	34 – 590	208	237	40 - 2892	368	217
TDS	(mg/L)	42 – 3226	122	509	10 - 1334	157	332	61 - 577	107	157	3.7 - 1800	87.3	60.3	-	-		-	-	-
TSS	(mg/L)	47 – 1040	116	240	10 - 419	70	120	32 - 771	215	283	1. - 2988	112.7	59.1	33 – 914	131	202	9 - 406	143	93
COD	(mg/L)	14.6 – 276	46	90	20.1 – 200	30.6	52.1	4 - 177	48	70	-	-	-	18 – 464	126	149	41 - 291	103	84
NO_3 + N_2^-	(mg/L)	0.2 - 2.4	0.5	0.8	0.23 - 1.8	0.5	0.63	0.08 - 13.37	0.38	2.25	0.01 - 4.8	1.07	0.6	0.0 – 3.66	1.03	1.25	0.19 - 3.32	0.84	0.66
TKN	(mg/L)	0.64 - 9.04	1.62	2.40	0.49 - 3.86	1.2	1.71	0.76 - 2.45	1	1.42	0.1 - 17.7	2.06	1.4	-	-	-	0.38 - 3.51	1.79	1.48
Ortho P	(mg/L)	-	-	-	-	-	-	0.01 - 0.74	0.08	0.15	0.01 - 2.4	0.11	0.06	-	-	-	-	-	-
Total P	(mg/L)	0.05 - 0.81	0.22	0.248	0.07 - 0.51	0.2	0.215	0.04 - 1.54	0.2	0.43	0.03 - 4.69	0.29	0.18	0.07 - 1.09	0.33	0.42	-	-	-
O&G	(mg/L)	-	< DL	< DL	-	< DL	< DL	1.0 - 11-1	3.3	4.4	1. - 20	6.6	6	0.8 - 35.1	4.1	6.5	-	-	-

<DL: Below detection limit

[a] W.T. Harries Blvd, Charlotte, NC. ADT 25,000 vehicles/day. Drainage area 0.37 acres. Adjacent area: woods, residential, heavy commercial.

[b] 34 highway sites in California. Detailed information about the sites can be found in table 6.2.

[c] MoPac at 35th Street, Austin, TX. ADT 60,000 vehicles/day. Catchment area 1.32 acres. Adjacent area: residential and commercial.

[d] 31 sites along the United States. Detailed information can be found in table 6.3.

Table 6.2 Characteristics of the sampled sites by Kayhanian et al.

Monitoring site identification	Highway	Average annual rainfall (mm)	Catchments area (ha)	Average annual daily traffic	Surrounding land use[a]
1-34	299	1016	0.20	9100	Rural
1-35	36	1016	0.48	3300	Rural
1-36	101	1016	0.87	5800	Rural
1-39	175	914	0.35	1800	Rural
2-01	36	889	0.65	2150	Rural
2-02	5	889	0.65	29,000	Rural
3-05	99	381	0.08	47,500	Commercial
3-06	80	762	0.60	74,000	Commercial
3-07	50	533	0.70	127,000	Commercial
3-224	65	742	1.21	36,000	Commercial
4-35	680	635	0.65	53,000	Rural residential
4-38	680	635	0.14	13,200	Agricultural, rural
4-39	580	635	0.09	134,000	Rural
5-03	25	660	0.04	2250	Rural
5-04	46	558	1.30	23,000	Rural
5-05	227	584	0.01	15,000	Commercial
5-06	1	660	5.95	55,000	Rural
6-05	198	178	0.37	14200	Agricultural
6-06	99	178	0.25	43,000	Agricultural
6-205	180	279	0.75	7200	Residential, Commercial
6-209	41	279	0.18	127,000	Residential
7-201	101	226	1.28	328,000	Residential
7-202	405	209	1.69	260,000	Residential, commercial
7-203	405	228	0.39	322,000	Residential, commercial
8-07	10	260	1.20	18,300	Rural
8-08	10	381	0.48	63,000	Residential
8-10	91	279	1.21	231,000	Rural
9-01	395	152	0.82	5800	Rural
10-02	120	965	0.73	4950	Mixed use
10-03	5	330	0.48	70,000	Rural
10-04	132	787	0.65	2100	Commercial
11-98	805	254	0.77	177,000	Residential, commercial
11-100	805	254	1.13	212,000	Residential, Army
11-101	8	330	0.19	175,000	Rural

[a] Runoff from surrounding land use did not contribute to the monitored drainage area.

Table 6.3 Characteristics of study sites compiled by Driscoll et al.

Site Location	**Avg. Daily Traffic (1000 VDP)**		**Number of Traffic Lanes**		**Section Type**	**Surface Type**	**Curb**	**Land Use**	**Area (Acres)**	**Percentage Impervious**	**Annual Rainfall (in/yr)**
	Total	Monitored Lanes	Total	Monitored Lanes							
Little Rock I-30	42	42	4	4	Bridge, fill	Asphalt	No	Urban – Residential	1.5	90	48.7
Los Angeles I-405	200	200	8	8	Fill	Concrete	Yes	Urban – Commercial/ residential	3.2	100	12.6
Sacramento Hwy 50	86	43	8	4	At grade	Concrete	Yes	Urban – Suburban	2.45	82	16.3
Walnut Creek I-680	70	70	6	6		Concrete	Yes	Urban – Residential	2.1	100	20.3
Denver I-25	149	149	10	10	At grade	Asphalt	Yes	Urban – Suburban	35.3	37	14.8
Broward CO Hwy 834	20	20	6	6	At grade	Asphalt	Both	Urban – Commercial/ residential	58.3	36	62
Miami I-95	140	70	6	3	Bridge	Asphalt	Yes	Urban - Undefined	1.43	100	59.8
Minneapolis I-94	80	80	10	10	Cut	Concrete	Yes	Urban – Commercial/ residential	21	55	24.8
St Paul I-94	65	65	6	6	Cut and fill	Concrete	Yes	Urban – Commercial/ residential	16.3	49	24.8
Efland I-85	26	26	4	3	At grade	Asphalt	No	Non-urban – Undefined	2.49	51	43.6
Harrisburg I-81 (Ph. 1)	24	24	6	6	At grade	Concrete	No	Urban – Suburban	18.5	27	37.7
Harrisburg I-81 (Ph. 2)	56	28	4	2	At grade	Concrete	No	Urban – Suburban	2.81	45	37.7
Nashville I-40	88	88	6	6	Cut – At grade	Concrete	Yes	Urban - Undefined	55.6	37	45.0
Montesano SR-12	7.3	7.3	2	2	At grade	Asphalt	Yes	Non-urban – Agricultural	0.28	100	84.0
Pasco SR-12	4.0	2.0	4	2	Cut	Concrete	Yes	Non-urban - Desert	1.25	100	7.5
Pullman SR-270E	5.0	2.5	2	1	At grade	Asphalt	Yes	Non-urban – Agricultural	0.25	100	18.0
Seattle I-5	106	53	8	4	At grade	Concrete	Yes	Urban – Residential	1.22	100	34.1
Seattle SR-520	84	42	4	2	Bridge	Concrete	Yes	Urban - Undefined	0.099	100	35.0
Snoq. Pass I-90	15	7.7	6	3	At grade	Concrete	Yes	Non-urban – Forest	0.18	100	97.0
Spokane I-90	35	17	6	3	Bridge	Concrete	Yes	Urban – Undefined	0.22	100	17.2
Vancouver I-205	17	8.6	6	3	At grade	Concrete	Yes	Urban – Suburban	0.28	100	39.0
Milwaukee Hwy 45	85	85	6	6	Cut – At grade	Concrete	Yes	Urban – Residential	106	31	27.6
Milwaukee I-794	53	53	8	8	Bridge	Concrete	Yes	Urban - Undefined	2.1	100	27.6
Milwaukee I-94	116	116	8	8		Asphalt	Yes	Urban – Residential	7.6	64	27.6

Comparing the data obtained in this study with similar studies, it is possible to see that the data from this study are comparable to those of other studies. Concentrations of lead, TDS, and TSS differ the most comparing the other studies.

Lead concentrations from other studies (e.g., Barret et al. and Driscoll et al.) are higher than the concentrations found in this study. The reason for this discrepancy is not clear, the values reported in our study are for dissolved lead, and some of the other studies may have included total lead results: however, this alone would not likely account for the large discrepancy.

6.3 Environmental Impact of Highway Pollutants

6.3.1 Surface Water Quality Standards

The Nebraska Department of Environmental Quality (NDEQ, 2009) established general criteria for aquatic life. Table 6.4 and table 6.5 show the acute and chronic concentrations for Metals and Inorganics and Selected Organic Compounds, respectively.

Table 6.4 Comparison of NDEQ criteria for aquatic life for metals and inorganics and mean EMC for the west and east outlet pipes

Analyte	**Acute (mg/L)**	**Chronic (mg/L)**	**Mean EMC for the West Pipe Outlet (mg/L)**	**Max. Concentration for the West Pipe Outlet (mg/L)**	**Mean EMC for the East Pipe Outlet (mg/L)**	**Max. Concentration for the East Pipe Outlet (mg/L)**	**Mean EMC for the Detention Basin Outlet Pipe (mg/L)**	**Max. Concentration for the Detention Basin Outlet Pipe (mg/L)**
Antimony	0.088	0.030	0.0045	0.0098	0.0036	0.0068	0.00255	0.003
Arsenic	0.34	0.0167	0.005	0.008	0.0030	0.006	0.00280	0.003
Beryllium	0.130	0.0053	N.D	0.001	N.D	0.001	N.D	N.D
Cadmium	0.0017	0.0002	0.0015	**0.003**	**N.D**	**0.00283**	**0.0023**	**0.0023**
Chromium (III)	0.489	0.064	0.04	0.059	0.027	0.03928	0.009	0.01964
Copper	0.011	0.008	**0.048**	**0.115**	**0.03**	**0.0768**	**0.014**	**0.0288**
Lead	0.053	0.002	0.019	0.047	0.014	0.0308	0.003	0.0083
Mercury	0.0014	0.00077	**0.0037**	**0.006**	**0.0033**	**0.005**	**0.003**	**0.0044**
Nickel	0.400	0.044	0.01	0.030	0.009	0.01996	0.004	0.014
Selenium	0.02	0.005	N.D	0.001	N.D	N.D	N.D	0.003
Silver	0.003	(Reserved)	N.D	N.D	N.D	N.D	N.D	N.D
Thallium	1.4	0.0063	N.D	N.D	N.D	N.D	N.D	N.D
Zinc	0.100	0.101	**0.0273**	**0.0802**	**0.0235**	**0.066**	0.077	**0.166**
Chloride	Not to exceed 860 mg/l at any time or a four-day average concentration of 230 mg/l.		207.3	**1709**	139	690	207.3	510

Table 6.5 NDEQ criteria for aquatic life for selected organic compounds

Analyte	Acute (μg/L)	Chronic (μg/L)
Monocyclic Aromatics except Phenols, Cresols, and Phthalates		
Benzene	5300	712.8
Chlorinated benzenes	250	50
Dichlorobenzenes	1120	763
Ethylbenzene	32,000	29,000
Hexachlorobenzene	6.0	.0077
Nitrobenzene	27,000	1,900
Pentachlorobenzene	(Reserved)	41
1,2,4,5-tetrachlorobenzene	(Reserved)	29
1,2,4-trichlorobenzene	(Reserved)	940
Toluene	17,500	200,000
2,4-dinitrotoluene	330	91
Polycyclic Aromatic Hydrocarbons (PAHs)		
Acenaphthene	1,700	520
Anthracene	(Reserved)	110,000
Benzo(a)anthracene	(Reserved)	0.49
Benzo(a)pyrene	(Reserved)	0.49
Benzo(b)fluoranthene	(Reserved)	0.49
Benzo(k)fluoranthene	(Reserved)	0.49
Chrysene	(Reserved)	0.49
Dibenzo(a,h)anthracene	(Reserved)	0.49
Fluoranthene	3,980	370
Fluorene	(Reserved)	14,000
Ideno(1,2,3-cd)pyrene	(Reserved)	0.49
Naphthalene	2,300	620
2-chloronaphthalene	1,600	4,300
Phenanthrene	30	6.3
Pyrene	(Reserved)	11,000

Comparing the results obtained from the analyzed samples with the Nebraska surface water quality standards, copper and zinc have been found to be above the acute toxicity levels. Cadmium was found to be above the acute toxicity levels on two rainfall events; however, it was below the detection limits for the other storm events. Sodium was found to have high concentrations comparing to other metals. Sodium, calcium, and other salts are included in the NDEQ criteria for TDS and it should be less than 500 mg/L. For most of the rainfall events, the TDS criterion was exceeded.

The NDEQ criteria for aquatic life apply to streams rather than at a discharge point; therefore, pollutant concentrations may not exceed the NDEQ standards once the discharged water is diluted into the stream. Sampling the stream would be required to verify if the discharge from the study site would cause the stream to exceed the NDEQ criteria. However, dilution would reduce nearly all of the concentrations to below the acute criteria.

Another major pollutant found in this study was Total Extractable Hydrocarbons (TEH) as diesel. Diesel fuels are similar to fuel oils used for heating (fuel oils no.1, no.2, and no. 4). Fuel oils consist of complex mixtures of aliphatic and aromatic hydrocarbons. The aliphatic alkanes (paraffins) and cycloalkanes (napthenes) are hydrogen saturated and make approximately 80% to 90% of the fuel oils. Aromatics (e.g., benzene) and olefins (e.g., styrene and indene) make 10% to 20% and 1% respectively, of the fuel oils. Diesel fuels predominately contain a mixture of C_{10} through C_{19} hydrocarbons, which include approximately 64% aliphatic hydrocarbons, 1% to 2% olefinic hydrocarbons, and 35% aromatic hydrocarbons (USDHHS, 1995). The compounds that were detected and reported as "diesel" are not generally included in the list of chemicals of environmental concern and are, therefore, not included in the list of VOCs and VOCs and SVOCs that are typically monitored in environmental studies. VOCs and SVOCs analyzed in this study were mostly below the detection limit or were at very low concentrations because most of these analytes are not a significant diesel fuel constituent.

6.3.2 Impact of Highway Pollutants on Aquatic Life

Cadmium is a very toxic element and it can easily accumulate in the body. The salmonid species of fish are more sensitive to cadmium than the cyprinid varieties. An increase in water hardness reduces its toxicity (Dojlido and Best, 1993). Although concentrations for cadmium were found to be below the water quality criteria for the mean EMC at the east and detention basin outlet pipes, there were two events in which cadmium was observed to slightly exceed the NDEQ criteria. However, it does not appear that cadmium will pose a significant ecological problem in the receiving water body because expected dilution will reduce concentrations to the water quality criteria.

Small amounts of copper are essential to life; however, large doses are toxic, especially to plants (Dojlido and Best, 1993). According to Moore and Ramamoorthy (1984), copper restricts the growth of aquatic plants at concentrations greater than 100 µg/L (as cited in Dojlido and Best, 1993). Copper mean EMC concentrations were found to be slightly above the water quality criteria for the east, west, and detention basin outlet pipes; however, the maximum EMC were found to be 7, 10, and 2 times higher than the acute water quality criteria, respectively. These high concentrations may be of concern for potential ecological effects.

Mercury compounds are very toxic to aquatic organisms. At a concentration of a few µg/L, the growth of plants may be inhibited. The lethal levels for fish vary from 2 mg/L for shrimp (daphnia), to 2 mg/L for insects, and from 30 mg/L for guppies to 1 mg/L for tilapia (Dojlido and Best, 1993). Mercury mean EMC concentrations were found to be slightly above the water quality criteria for the east, west, and detention basin; however, EMC concentrations were found to be 5, 6, and 4 times higher than the acute water quality criteria, respectively. These high concentrations may be of concern for potential ecological effects.

6.3.3 Comparison Between BOD and COD

A comparison between BOD and COD gives information about the sources of the oxygen demanding constituents and their ability to biodegrade. Table 6.6 shows the BOD/COD ratio for the west and east outlet pipes for the sampled rainfall events where both indicators were analyzed.

Table 6.6 Comparison between BOD and COD for selected sampled events

Rainfall Event	**BOD/COD ratio**			
	West Pipe Outlet		**East Pipe Outlet**	
	First Flush	**EMC**	**First Flush**	**EMC**
11/10/2008	0.051	0.150	0.188	0.134
3/27/2010	0.071	0.052	0.105	0.070
5/7/2010	0.224	0.091	0.116	0.300
5/20/2010	-	0.234	-	0.196
7/4/2010	0.400	0.292	0.304	0.207
9/13/2010	0.205	0.205	0.225	0.225

The BOD/COD ratio ranges from 0.051 to 0.400. These values indicate that most of the oxygen demand comes from inorganic chemicals.

6.4 Effectiveness of the Detention Basin

The effectiveness of the current detention basin as a water quality BMP at the study site can be determined by comparing the calculate loads from the basin outlet with the loads from the west and east outlet pipes. Loads at the basin outlet correspond to the total load for the study site; therefore, in order to estimate the BMP effectiveness, it is necessary to subtract the loads from non-highway runoff sources. For this study, the only non-highway source monitored was the construction debris lot. Once the loads from this source were subtracted, the results were compared to the loads from the east and the west pipe outlets. Table 6.7 shows the loads entering and exiting the detention basin.

Table 6.7 Comparison between pollutant loads entering and exiting the detention basin

Analyte	Rain Storm Event									
	11/10/2008		5/12/2009		7/3/2009		7/31/2009		9/3/2009	
	Loads Into the Basin (Kg)	Loads Out of the Basin (Kg)	Loads Into the Basin (Kg)	Loads Out of the Basin (Kg)	Loads Into the Basin (Kg)	Loads Out of the Basin (Kg)	Loads Into the Basin (Kg)	Loads Out of the Basin (Kg)	Loads Into the Basin (Kg)	Loads Out of the Basin (Kg)
Cadmium (dissolved)	4.17E-05	3.19E-05	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001
Calcium (dissolved)	0.49	1.12	0.32	0.58	0.61	1.37	0.58	-4.54[a]	11.8	89.2
Chromium (dissolved)	4.74E-05	1.86E-05	4.99E-05	-9.84E-05[a]	2.11E-04	4.32E-04	2.71E-04	1.55E-04	0.0020	0.0113
Copper (dissolved)	4.18E-04	3.23E-04	2.91E-04	1.99E-04	4.03E-04	-0.00110[a]	6.52E-04	1.22E-03	0.0051	0.0180
Iron (dissolved)	0.0028	0.0002	1.12E-04	1.82E-04	0.00052	0.00052	0.027	0.0117	0.0055	0.0176
Lead (dissolved)	6.16E-05	1.27E-05	5.64E-06	5.37E-06	1.881E-05	4.36E-05	3.04E-04	-1.76E-04[a]	2.86E-04	1.20E-03
Magnesium (dissolved)	0.048	0.496	0.020	0.063	0.023	0.023	0.019	-0.415[a]	0.67	19.35
Mercury (dissolved)	N/A	N/A	4.52E-05	4.94E-05	3.95E-04	1.30E-04	1.70E-04	1.75E-04	0.0021	0.0070
Nickel (dissolved)	N/A	N/A	3.29E-05	2.07E-05	5.27E-05	1.39E-04	4.74E-04	8.49E-04	6.19E-04	0.00491
Potassium (dissolved)	0.20	0.36	0.43	-0.262[a]	0.50	0.02	1.78	-1.95[a]	10.8	50.6
Sodium (dissolved)	0.63	5.77	1.89	0.74	1.95	4.97	1.24	-8.14[a]	14.5	296.3
Zinc (dissolved)	0.41	0.16	<0.10	<0.10	0.028	<0.10	7.20	14.31	<0.1	<0.1
Silica	0.18	0.26	0.060	0.045	0.15	0.40	0.097	-0.498[a]	<0.1	<0.1
Chloride	0.88	8.15	3.08	1.33	1.86	3.84	1.23	-10.7[a]	80.6	379
Fluoride	0.014	0.012	0.0092	0.014	0.017	0.022	0.012	0.019	0.10	0.45
Nitrate	<0.10	-0.00242[a]	0.013	-0.00292[a]	0.022	0.025	0.036	-0.00917[a]	0.77	1.09
Phosphate	<0.10	<0.10	0.0048	0.0099	0.021	0.037	0.018	0.033	0.25	1.01
Sulfate	0.32	1.47	0.75	-0.638[a]	0.62	1.20	0.57	-3.50[a]	6.61	146.9
Soluble Phosphate	0.0028	0.0010	0.0017	0.0025	0.0060	0.013	0.027	-0.00153[a]	0.080	0.140
Total Phosphorus	0.0087	0.0074	0.0013	0.0028	0.0090	0.0227	0.0062	0.0088	0.11	0.15
Total Kjeldahl Nitrogen	N/A	N/A	0.025	0.030	0.045	0.010	0.079	-0.117[a]	1.44	4.60
Total Dissolved Solids	1.71	0.00	7.38	-1.94[a]	8.43	90.50	4.43	-27.9[a]	78.1	1616.3
Total Suspended Solids	N/A	N/A	1.19	-1.15[a]	6.40	118.9	4.55	-26.5[a]	94.3	1235.2
Total Solids	15.7	15.6	8.47	-3.09[a]	14.8	209.4	8.98	-54.4[a]	172	2852
Volatile Dissolved Solids	1.71	<10	<10.0	<10.0	6.77	37.28	0.79	-8.78[a]	195	195
Volatile Suspended Solids	2.08	5.25	1.19	1.01	2.78	0.12	1.37	-0.674[a]	82.8	55.0
Total Volatile Solids	3.79	2.57	0.96	-0.459[a]	9.56	37.40	2.15	-9.46[a]	92.3	250.3
Alkalinity as $CaCO_3$	1.21	3.25	1.11	1.22	2.00	9.45	1.60	0.27	27.3	232.6
COD	0.73	2.24	0.48	-0.635[a]	4.08	7.34	0.718	-0.00595[a]	234	234

N/A: Not data available.

[a] Negative values indicates that loads at the construction debris lot are higher than loads at the detention basin outlet

Table 6.7 Comparison between pollutant loads entering and exiting the detention basin (cont.)

Analyte	Rain Storm Event							
	3/27/2010		5/7/2010		5/20/2010		7/4/2010	
	Loads Into the Basin (Kg)	Loads Out of the Basin (Kg)	Loads Into the Basin (Kg)	Loads Out of the Basin (Kg)	Loads Into the Basin (Kg)	Loads Out of the Basin (Kg)	Loads Into the Basin (Kg)	Loads Out of the Basin (Kg)
Antimony (total)	9.37E-05	4.73E-05	3.528E-05	1.10E-04	1.176E-09	3.30E-04	<0.0010	<0.0010
Arsenic (total)	7.902E-05	2.05E-05	9.878E-05	1E-05	0.0003666	3.67E-04	8.82E-09	0.0023
Calcium (total)	1.04	1.00	2.00	0.49	3.95	3.95	5.58E-05	16.4
Chromium (dissolved)	5.54E-04	3.43E-04	4.16E-04	8.41E-04	1.37E-08	0.0024	0.0075	0.0075
Chromium (total)	5.64E-04	3.49E-04	4.23E-04	8.56E-04	1.4E-08	0.0024	0.0077	0.0077
Copper (dissolved)	0.0013	0.0005	0.0015	0.0015	1.08E-08	0.0023	2.42E-08	0.0074
Copper (total)	0.0014	0.0005	0.0016	0.0016	1.12E-08	0.0024	2.52E-08	0.0077
Lead (dissolved)	4.40E-04	1.37E-04	1.25E-04	2.59E-04	2.97E-09	4.54E-04	1.41E-08	0.0043
Lead (total)	5.56E-04	1.74E-04	1.58E-04	3.28E-04	3.75E-09	5.74E-04	1.78E-08	0.0054
Magnesium (total)	0.12	0.03	0.45	-0.43	4.52E-06	6.46E-01	8.59E-06	2.50
Nickel (dissolved)	2.82E-04	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Nickel (total)	2.82E-04	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Sodium (total)	2.14	4.93	10.4	-1.1[a]	9.67E-05	18.3	1.23E-04	31.0
Thallium (total)	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005
Zinc (dissolved)	0.00817	0.00290	0.0022	0.0042	3.83E-08	8.37E-03	7.39E-08	0.023
Zinc (total)	0.00835	0.00296	0.00226	0.0043	3.92E-08	0.0086	7.56E-08	0.023
Chloride	1.56	7.38	9.26	1.53	2.09E-04	22.6	1.56E-04	30.0
Nitrate/Nitrite Nitrogen	0.0068	0.0070	0.040	0.075	4.48E-07	0.073	0.23	0.23
Phosphorus (dissolved ortho)	0.0010	<0.05	0.0042	0.0042	<0.05	<0.05	2.14E-07	0.100
Total Phosphorus	0.0075	0.0037	0.0069	0.0153	8.96E-08	0.021	6.68E-07	0.23
Total Kjeldahl Nitrogen	0.028	0.012	0.073	0.139	7.196E-07	0.12	3E-06	0.91
Total Dissolved Solids	6.08	15.7	38.6	-7.3	3.96E-04	63.5	3.30E-04	98.3
Total Suspended Solids	5.73	2.12	3.73	4.35	5.29E-05	8.31	3.91E-04	83.0
TEH as Diesel	0.0062	-2.16[a]	0.012	-2.24[a]	0.032	0.032	1.31E-07	0.055
BOD	0.16	0.20	0.47	1.19	4.76E-06	1.10	1.76E-05	5.38
COD	2.66	2.18	4.57	2.56	4.28E-05	7.33	9.45E-05	33.03

N/A: No data available.

[a] Negative values indicates that loads at the construction debris lot are higher than loads at the detention basin outlet

A reduction of some contaminants can be observed in low intensity rain events (11/10/2008, 5/12/2009, 7/31/2009, 3/27/2010, and 5/07/2010). The total rainfall in these events ranges from 0.14 to 0.32 in. The reduction was observed in heavy metals, chloride, COD, and TSS. This reduction may be due to the low flow generated during low intensity storms; therefore, more particulates are absorbed by the soil and grass cover in the detention basin. Conversely, during high-intensity events the higher flow conditions in the basin may not have allowed particulate settling, and indeed, some of the previously particulates may have been picked up, increasing the pollutant loads at the detention basin outlet.

Since the detention basin was not designed to reduce pollutant loads from the highway runoff, it is not surprising that little load reduction is observed. To improve pollutant removal, the basin should be re-designed to increase detention time, sedimentation rates, and biological activity.

6.5 Possible Measures to Reduce Pollutants

Heavy metals, COD, BOD, suspended solids, dissolved solids, and diesel fuel are the main concerns from the highway runoff at the study site. An option that can be considered to reduce the concentration and loads from these pollutants is the construction of an extended detention basin. This type of structure typically provides 68% to 90% removal of sediments, 42% to 90% removal of heavy metals, and 42% to 50% removal of COD (FHWA, 1996). The existent detention basin could be adapted by making changes in the outlet design. For an extended basin, the outlet would be much smaller, extending the detention time in the basin.

Chapter 7 Conclusions

Twelve highway runoff samples were collected from November 2008 through November 2010. For each sampled year, two rain events for each weather season (spring, summer, and fall) were sampled. Winter snowfalls were not sampled in this study. A first flush and a composite sample for each sampling location were analyzed for every rainfall event. Heavy metals, anions, nutrients, BOD, COD, SVOCs, and VOCs concentrations were obtained from the Water Science Laboratory at UNL and Midwest Laboratories for 2009 and 2010 respectively. The results of the samples show:

- Metals (especially copper, chromium, and sodium), BOD, COD, suspended solids, and dissolved solids are the primary contaminants found in the highway runoff.
- The metals of most concern to the environment are copper, mercury, and cadmium.
- TEH as diesel was found in all 2010 samples (it was not analyzed for the 2009 samples). According to the laboratory, this result does not correspond specifically to diesel. Instead, the results correspond to a mixture of compounds with diesel-range molecular weights that cannot be classified as gasoline, diesel, oil, fuel additives, coolant fluid, or brake fluid.
- Chloride, total phosphorus, and TKN were found in all of the sampled events. However, the concentrations were low, having little or no apparent impact to the receiving stream.
- SVOCs and VOCs were found to be below the detection limits in most of the sampled events.
- Total suspended solids (TSS), antecedent dry period (ADP), total rainfall, and volume of runoff are typically believed to influence highway runoff pollutant concentrations. However, no strong correlations between pollutant concentrations and these variables

were found in this study.

- The existing detention basin has minimal effectiveness in reducing the pollutants from the highway.

Chapter 8 Volume II Introduction

8.1 History of Stormwater Regulation

As water quality regulations have developed, a greater focus has been put on remediating pollutants associated with stormwater runoff. The main piece of surface water quality legislation is the Clean Water Act (CWA). The CWA was originally passed in 1972 as the Federal Water Pollution Control Amendments of 1972, and became known as the CWA after 1977 amendments were made. The goal of the CWA was to “Restore and maintain the chemical, physical, and biological integrity of the nation’s waters” (CWA 1977a). The original CWA was implemented to regulate discharges into navigable waters from discrete point sources. Point sources were considered to be any “pipe, ditch, channel, tunnel, conduit, well, discrete fissure, container, rolling stock, concentrated animal feeding operation, or vessel or float craft” (Clean Water Act 1977a). Regulations were based on effluent limitations which were enforced through permitting in the National Pollutant Discharge Elimination System (NPDES). Permitting for a facility included conditions for effluents limitation, monitoring, operation and maintenance, upset and bypass provisions, record keeping, and inspections.

Although stormwater runoff, known as non-point source pollution, is conveyed through measures which are considered point sources (i.e., ditches, pipes, or channels), they were not regulated until the 1987 amendments to the CWA which included them into the NPDES (CWA 1977b). Runoff under NPDES is regulated for construction and post-construction considerations. Highway construction and operation permits are regulated under NPDES due to the build-up of pollutants associated with automobiles and wear of the driving surface. However, not all highway systems currently require permitting. Permits are only required where the roadway discharges into a Municipal Separate Storm Sewer System (MS4), which have discrete outfalls to receiving waters and are located in urban areas (NDOR 2010). Storm Water Management Plans are developed for MS4 permits which feature the following 6 minimum Best Management Practice (BMP) programs: public education and outreach, public participation and involvement, illicit discharge detection and elimination, construction site runoff control, post-construction site runoff control, and pollution prevention and good house-keeping (CWA 1977a). The permitting does not include effluent limitations, but does stipulate that the above six minimum BMP programs should be instituted to remediate runoff to the maximum extent practicable (CWA 1977a). BMPs can be broadly categorized as structural or non-structural. Structural BMPs actively remove pollutants from runoff while non-structural are generally related to source control.

8.2 Pollutants Discharging from Roadways

Knowing which pollutants are present in runoff from the roadway is essential to remedial efforts.

Table 8.1 shows the primary constituents of runoff from Interstate 80 near the 108th street crossing in Omaha, Nebraska which had been sampled between 2009 and 2011 for the Nebraska Department of Roads (NDOR) (Torres 2010). Many of the contaminants are innocuous in themselves or in such low concentrations that they will not impact the ecosystem.

Table 8.1 Roadway pollutants (Torres 2010)

Calcium	Total Kjeldahl Nitrogen	Total Phosphorus	Lead
Magnesium	Total Dissolved Solids	Nitrate	Mercury
Potassium	Total Suspended Solids	Nitrite	Nickel
Sodium	Total Solids	Phosphate	Oil and Grease
Cadmium	Volatile Dissolved Solids	Sulfate	COD
Chromium	Volatile Suspended Solids	Zinc	Soluble Phosphate
Copper	Total Volatile Solids	Silica	Chloride
Iron	Alkalinity as CaCO3	Bromide	Fluoride

Pollutants of concern found in the Nebraska study are metals (primarily copper, mercury, and cadmium) total solids, dissolved solids (in the form of sodium), and diesel and gasoline constituents (Torres 2010). Although nutrients such as nitrogen and phosphorous were found they were not at high enough concentrations to adversely impact receiving waters (Torres 2010). Sampling found high concentrations of total extractable hydrocarbons (TEH). The TEH are generally compounds with molar weights consistent with gasoline or diesel. There is no toxicity data available for the TEH, so it is not generally classified as a chemical of environmental concern (Torres 2010).

It would be ideal to establish primary pollutants for each site where BMPs are being considered. However, this is not always feasible. The data collected during the NDOR study will, therefore, be considered to be characteristic of runoff contamination across the state. Metals and solids will be the pollutants the BMPs will be designed to remediate.

Chapter 9 Volume II Objectives

The objective of volume II was to assemble a set of design guides of Best Management Practices (BMPs) tailored to treating runoff from roadways. This will be accomplished by identifying which BMPs are applicable to roadside scenarios, compiling fact sheets on the applicable BMPs, and establishing the design processes for the selected BMPs. The fact sheets are to be consulted in order to determine which BMP is best for site-specific conditions. Then the design guide for that BMP will be used to ensure the selected BMP will function properly. This work has been developed to comply with requirements for a NPDES permit for the MS4 servicing highways in Nebraska.

Chapter 10 Volume II Literature Review

10.1 Historical Perspective

The forerunner of BMPs came with the Soil Conservation Act of 1935. This Act was enacted to counter the soil erosion of the dust bowl era and spawned a Soil Conservation District movement (Ice 2004). Although this Act did not directly regulate discharges to water, it did begin legislation directed towards protecting environmental resources.

In 1949 the *Yearbook in Agriculture* published the article "Watersheds and How to Care for Them." This article stressed the importance of maintaining the land and streams, which would allow them to continue to be usable. It called for the implementation of better land practices to protect receiving waters and prevent erosion, much like BMPs are used today (Ice 2004).

The watershed approach from 1949 can be seen mirrored in modern Total Maximum Daily Load (TMDL) requirements. TMDLs are the acceptable loading of a given compound in a water-body which is considered safe for the intend use of the water-body. Non-point source pollution has been ruled in the case of *Pronsolino v Nastri* to be considered in the TMDL (Ice 2004). This ruling furthered the need for BMPs on a watershed scale.

One of the first BMPs to be developed and rigorously studied was the surface sand filter by the city of Austin, Texas (Landphair et al. 2000).

10.2 Expected Quality of Runoff From Roadways

Pollutant concentrations can vary widely. Table shows a comparison of observed pollutant concentrations coming from roadways. These results show a significant variation on a site-to-site basis. Runoff from roadways is generally low in nitrogen and phosphorous concentrations, but may contain excessive amounts of solids, metals, or oil and grease.

Table 10.1 Comparison of contaminant concentrations in roadway runoff (Torres 2010)

Analyte		NDOR Study						Wu et al.[a]			Kayhanian et al.[b]			Barret et al.[c]			Driscoll et al.[d]		
		West Pipe (EMC)			East pipe (EMC)			Monitoring Site I (EMC)			Range	Median	Mean	35th Street (EMC)			National Highway Runoff Report		
		Range	Median	Mean	Range	Median	Mean	Range	Median	Mean				Range	Median	Mean	Range	Median	Mean
Cu	(µg/L)	5 - 115.2	30	48	5.1 - 76.8	17	26	9.0 - 52	15	24.2	1.1 - 130	14.9	10.2	2.0 – 120	34	38	5 - 155	52	39
Cd	(µg/L)	-	3	3	-	2.8	2.8	<DL	<DL	<DL	0.2 - 8.4	0.24	0.13	-	-	-	-	-	-
Cr	(µg/L)	1.3 - 58.9	30	40	1.1 - 39.3	20	27	5.0 - 20	6.5	8.1	1.0 - 23	3.3	2.2	-	-	-	-	-	-
Pb	(µg/L)	0.2 - 47.1	7	19	0.2 - 30.8	6	14	7.0 - 56	15	21	1.0 - 480	7.6	1.2	7 – 440	50	99	11 - 1457	525	234
Fe	(µg/L)	2.8 - 125	40	50	2.9 – 106	3	4.5	-	-	-	32 - 3310	378	150	300 – 10000	2606	3537	-	-	-
Ni	(µg/L)	0.6 - 29.9	N.D.	8	0.6 – 20.0	N.D.	9	9.0 - 17	9	8.1	1.1 - 40	4.9	3.4	-	-	-	-	-	-
Zn	(µg/L)	4.5 – 82	11.5	27.3	0.7 - 66	0.2	0.4	-	-	-	3 - 1017	68.8	40.4	34 – 590	208	237	40 - 2892	368	217
TDS	(mg/L)	42 – 3226	122	509	10 - 1334	157	332	61 - 577	107	157	3.7 - 1800	87.3	60.3	-	-		-	-	-
TSS	(mg/L)	47 – 1040	116	240	10 - 419	70	120	32 - 771	215	283	1. - 2988	112.7	59.1	33 – 914	131	202	9 - 406	143	93
COD	(mg/L)	14.6 – 276	46	90	20.1 – 200	30.6	52.1	4 - 177	48	70	-	-	-	18 – 464	126	149	41 - 291	103	84
NO$_3$ + N$_2^-$	(mg/L)	0.2 - 2.4	0.5	0.8	0.23 - 1.8	0.5	0.63	0.08 - 13.37	0.38	2.25	0.01 - 4.8	1.07	0.6	0.0 – 3.66	1.03	1.25	0.19 - 3.32	0.84	0.66
TKN	(mg/L)	0.64 - 9.04	1.62	2.40	0.49 - 3.86	1.2	1.71	0.76 - 2.45	1	1.42	0.1 - 17.7	2.06	1.4	-	-	-	0.38 - 3.51	1.79	1.48
Ortho P	(mg/L)	-	-	-	-	-	-	0.01 - 0.74	0.08	0.15	0.01 - 2.4	0.11	0.06	-	-	-	-	-	-
Total P	(mg/L)	0.05 - 0.81	0.22	0.248	0.07 - 0.51	0.2	0.215	0.04 - 1.54	0.2	0.43	0.03 - 4.69	0.29	0.18	0.07 - 1.09	0.33	0.42	-	-	-
O&G	(mg/L)	-	< DL	< DL	-	< DL	< DL	1.0 - 11-1	3.3	4.4	1. - 20	6.6	6	0.8 - 35.1	4.1	6.5	-	-	-

10.2.1 Sources of Pollutants

Roadway pollutants are associated with wear and maintenance of the roadway surface, vehicle operation, and atmospheric loading. Wear of the surface creates particulates which are then washed off the road by rainfall. Vehicles can deposit heavy metals, oil/grease, poly-aromatic hydrocarbons (PAHs), petroleum hydrocarbons, benzene, toluene, ethyl benzene, xylene (BTEX), as well as debris from careless drivers throwing trash out as they drive (MSSC 2008; Nixon & Saphores 2007).

Vehicles also discharge contaminants into the atmosphere which then settle back onto roads. These pollutants include heavy metals, dust, and PAHs (Barrett et al. 1995). Atmospheric loading is also shown to deposit nutrients, accounting for as much as 90% of the nitrogen loading (Wu et al. 1998).

Deicing considerations must be taken in winter months in order to keep roadways running efficiently (MSSC 2005). Deicing salts add high levels of sodium and chloride to runoff, as well as adding suspended solids. These loads can be reduced by employing more benign salts, such as Calcium Magnesium Acetate and Potassium Acetate, which will have less negative environmental impacts (FHA 1997a). Vegetated systems adjacent to roadways may also be negatively affected by road salt (Barrett et al. 1995). When the roadside vegetative cover decreases it promotes channelization of runoff causing erosion, which adds to particulate loading (FHA 2002a). Sand added for traction during the winter also contributes to particulate loading (MSSC 2005).

10.2.2 Factors Affecting Pollutant Loads

Pollutant concentrations and constituents vary with season, time between runoff events, road usage, and within individual events. Average daily traffic has been found to result in higher concentrations of some pollutants in urban area and higher concentrations of other pollutants in rural (Kayhanian et al. 2003). Urban areas have been shown to have high metals and solids but low nutrient loads (Flint and Davis 2007) while rural areas may have higher nutrient loads from agricultural practices. This finding implies there are factors besides traffic volume affecting pollutant concentration.

Required roadway maintenance can cause pollutant fluctuation with the seasons (Barrett et al. 1995). Table lists necessary maintenance practices which may impact receiving waters (Kramme et al. 1985). The potential for these activities to adversely affect water quality increases with proximity to the receiving water (Barrett et al. 1995).

Table 10.2 Maintenance activity which may contribute to highway contamination (Kramme et al. 1985)

Activities with Probable Impact	Activities with Possible Impact
Repairing slopes, slips, and slides	Full depth repairs
Cleaning ditches, channels and drainage structures	Surface treatments
Repairing drainage structures	Blading and repairing unpaved berms and /or ditches
Bridge painting	Bridge surface cleaning
Subsurface repair	Bridge deck repairs
Chemical vegetation control	Mowing
	Planting or care of shrubs, plants, and trees
	Seeding, sodding, and fertilizing
	Application of abrasives
	Care of rest areas
	Washing and cleaning maintenance equipment
	Bulk storage of motor fuels
	Disposal of used lubricating oil

There also may be concentration differences between wet and dry periods due to time between runoff events (Lee et al. 2004). Periods with little rain allow pollutants to build-up on roadways creating higher loads when the accumulated pollutants are subsequently washed away.

Although traffic volume, antecedent dry period, rainfall intensity, and rainfall depth have been demonstrated to affect pollutant loading and concentration, that is not always the case. Multiple studies have shown weak correlations to these factors (Desta et al. 2007; Torres 2010). Site-specific sampling is required to get an accurate prediction for contamination loads. However, this may not be cost effective, and the wide variation within an individual site, as shown in Table , often still leaves significant uncertainty.

10.2.3 First Flush

Pollutants tend to be washed from the surface of the roadway by the initial runoff in a phenomenon known as the first flush. If the majority of pollutants are contained within the first small portion of rainfall, BMPs only need to be sized to accommodate that volume. The first flush can be described by the first percentage of a storm which runs off or as the first depth of runoff, regardless of total event precipitation. Using the first percentage from roadways method has yielded inconclusive or unsatisfactory results for pollutant loading (Hallberg & Renman 2008; Flint & Davis 2007). Therefore, basing the first flush will be based on an initial runoff depth. The first flush has been observed to remove 81% to 86% of contaminants in the first 0.5 in. and 89% to 96% of pollutants in the first 0.75 in. (Flint & Davis 2007).

Early spring rain events and snow melt may also cause a seasonal first flush phenomenon (Sansalone et al. 1995; Stenstrom & Kayhanian 2005). For example, a spike in pollutants during spring may be due to the washing away of pollutants which have built up on roadways during the winter such as deicing agents, and sand and gravel applied to the roadway (MSSC 2005).

Chapter 11 Volume II Methods

11.1 BMP Selection Criteria

The following criteria, which are based on guidance from NDOR, were considered when determining which BMPs were most applicable for roadside applications:

- Pollutants to be remediated: 80% removal TSS (MSSC 2008; KCDENR 2009), heavy metal (Torres 2010), total extractable hydrocarbons (gasoline and diesel) (Torres 2010)
- Low maintenance
- Cost Effective
- No permanent pools
- Implement BMP within existing right of way
- Infiltration should not be primary removal mechanism near roadway
- Peak flow reduction
- Aesthetics
  - Green infrastructure

11.2 BMP Selection Process

Many BMPs were considered for this manual based on the selection criteria. The BMPs which were selected for this manual are vegetated filter strips, vegetated swales, bioretention, sand filters, and horizontal filter trenches.

- Vegetated filter strips and vegetated swales have shown adequate pollutant removal while providing low construction and maintenance costs. They also have high retrofit potential within the right-of-way and provide pleasing aesthetics of vegetation near roadways (UDFC 2010; CEI & NHDES 2008).
- Bioretention is also an effective pollutant removal BMP while providing positive

aesthetics (UDFCD 2010; CEI & NHDES 2008) and is flexible enough to be located within the right-of-way or in urban areas (SEMCOG 2008).

- Sand filters were selected based on their track record of successful application in storm water management (Landphair et al. 2000) and their ability to be used in urban areas where land availability limits other BMPs.
- Horizontal filter trenches are a BMP which is being developed. They were selected for this design guide because they are a relatively simple BMP which will fit within the right-of-way and will not require a significant amount of maintenance.

After evaluating the criteria, some common BMPs which were not deemed suitable for roadside applications were detention facilities, retention ponds, permanent wetlands and infiltration facilities. These were not further evaluated in this work.

- Detention ponds were not included due to limited solids removal compared to other BMPS (CEI & NHDES 2008; EPA 2006a) and space constraints within the right-of-way.
- Retention ponds and permanent wetlands were not considered due to the inherent danger of locating standing water near roadways.
- Inclusion of infiltration facilities would have been redundant because design variations to the horizontal filter trench enable it to act as an infiltration trench, and variations to bioretention allow it to perform as an infiltration basin.
- Ultra-urban BMPs, such as inlet inserts and hydrodynamic separators, were not considered in this report due to their general ineffectiveness as stand-alone BMPs in regard to removal of dissolved solids and metals (EPA 2006e; FHA 2002). These products also tend to be expensive compared to the selected BMPs, particularly in regards to treatment attained (UNH 2005). There are also generally high maintenance burdens to

avoid a drop off in performance (EPA 2006e; UNH 2005; FHA 2002c).

11.3 Hydrology

11.3.1 Water Quality Flow and Volume

11.3.1.1 First Flush

The first flush is the initial runoff which comes off the roadway. This value can be defined by percentage of pollutant load, percentage of total runoff, or as a static value of runoff depth (e.g., 0.5 in. or 0.75 in.). The latter definition is a simple yet effective means to quantify the first flush. The first 0.5 in. has been shown to contain 81% to 86% of pollutants and is commonly used to define the water quality volume (WQV) that requires treatment (Flint and Allen 2010). Therefore, a depth of 0.5 in. was used to represent the first flush and to determine the WQV runoff throughout this document.

11.3.1.2 Calculating the Design Precipitation

To determine the rainfall that produces the first flush, or WQV, back calculations using the National Resource Conservation Service (NRCS) method and 0.5 in. of runoff were performed using equation 11.1 (NRCS 1986):

$$Q = \frac{(P-0.2S)^2}{(P+0.8S)} \tag{11.1}$$

where:

Q: Depth of runoff over the watershed (in or cm)

P: Precipitation (in or cm)

S: Potential maximum retention of water by the soil (in or cm)

Potential maximum retention is a function of the Curve Number (CN) calculated with equation 11.2:

$$\mathrm{CN} = \frac{1000}{10+\mathrm{S}} \qquad (11.2)$$

Table shows the curve numbers for various land uses and hydrologic soil groups. Table defines these soil groups.

Table 11.1 Curve numbers for various land uses and conditions

Description of Land Use	**Hydrologic Soil Group**			
	A	**B**	**C**	**D**
Paved parking lots, roofs, driveways	98	98	98	98
Streets and Roads:				
Paved with curbs and storm sewers	98	98	98	98
Gravel	76	85	89	91
Dirt	72	82	87	89
Cultivated (Agricultural Crop) Land:				
Without conservation treatment (no terraces)	72	81	88	91
With conservation treatment (terraces, contours)	62	71	78	81
Pasture or Range Land:				
Poor (<50% ground cover or heavily grazed)	68	79	86	89
Good (50–75% ground cover; not heavily grazed)	39	61	74	80
Meadow (grass, no grazing, mowed for hay)	30	58	71	78
Brush (good, >75% ground cover)	30	48	65	73
Woods and Forests:				
Poor (small trees/brush destroyed by over-grazing or burning)	45	66	77	83
Fair (grazing but not burned; some brush)	36	60	73	79
Good (no grazing; brush covers ground)	30	55	70	77
Open Spaces (lawns, parks, golf courses, cemeteries, etc.):				
Fair (grass covers 50–75% of area)	49	69	79	84
Good (grass covers >75% of area)	39	61	74	80
Commercial and Business Districts (85% impervious)	89	92	94	95
Industrial Districts (72% impervious)	81	88	91	93
Residential Areas:				
1/8 Acre lots, about 65% impervious	77	85	90	92
1/4 Acre lots, about 38% impervious	61	75	83	87
1/2 Acre lots, about 25% impervious	54	70	80	85
1 Acre lots, about 20% impervious	51	68	79	84

(NRCS 1986)

Table 11.2 Hydrologic soil groups

Group	Minimum Infiltration Rate (in/hr)	Texture
A	0.3–0.45	Sand, loamy sand, or sandy loam
B	0.15–0.3	Silt loam or loam
C	0.05–0.15	Sandy clay loam
D	0–0.05	Clay loam, silty clay loam, sandy clay, silty clay, or clay

(Gupta 2008)

For impervious surfaces, such as pavement, a CN of 98 was assigned resulting in S = 0.2 from equation 11.2.

$$98 = \frac{1000}{10 + S}$$

Equation 11.1 was then used, in accordance with Nebraska Department of Roads (NDOR) guidance, with the calculated S value and a known Q of 0.5 in. (WQV) in the NRCS equation, the design precipitation was determined to be approximately 0.75 in. The 0.75 in. event will produce 0.55 in. of runoff. This 0.75 in. storm was then used for BMP designs.

$$0.5 = \frac{(P - 0.2 * 0.2)^2}{(P + 0.8 * 0.2)}$$

11.3.1.3 Peak Flow-Rate Calculations

Separate peak flow calculations were performed for the WQV peak flow rate, which was used to size the BMP treatment processes, and peak flow for the 10-yr storm, which was used as a design to check for potential scouring.

Peak Water Quality Flow Rates. Peak flows have been calculated and displayed in Table for impervious surfaces, such as pavement, up to 5 acres. For pervious areas or areas larger than 5 acres, peak flow rates were determined by using the 0.75 in. design storm with a type II NRCS 24 hr distribution and equation 11.3 (NRCS 1986).

$$q_p = q_u A_m Q F_p \tag{11.3}$$

Where:

q_p: Peak discharge (cfs)

q_u: Unit peak discharge (cfs/mi^2/in) (Figure or Table)

A_m: Drainage area (mi^2)

Q: Runoff depth corresponding to 24-hr rainfall (in) (Table for WQV)

F_p: Pond or swamp adjustment factor (1.0 for Nebraska)

Table 11.3 Peak water quality flows and water quality volumes for impervious watersheds up to 5 acres

Drainage Area (ac)	Peak Discharge ^{a} (cfs)	WQV^b (ft^3)	Drainage Area (ac)	Peak Discharge ^{a} (cfs)	WQV^b (ft^3)
0.1	0.095	181.5	1.25	1.184	2268.75
0.2	0.189	363	1.5	1.421	2722.5
0.3	0.284	544.5	1.75	1.657	3176.25
0.4	0.379	726	2	1.894	3630
0.5	0.474	907.5	2.5	2.368	4537.5
0.6	0.568	1089	3	2.841	5445
0.7	0.663	1270.5	3.5	3.315	6352.5
0.8	0.758	1452	4	3.788	7260
0.9	0.852	1633.5	4.5	4.262	8167.5
1	0.947	1815	5	4.735	9075

a) Calculated with equation 11.3 and the 0.75 in. design storm
b) Calculated with equation 11.4 and 0.5 in. of runoff

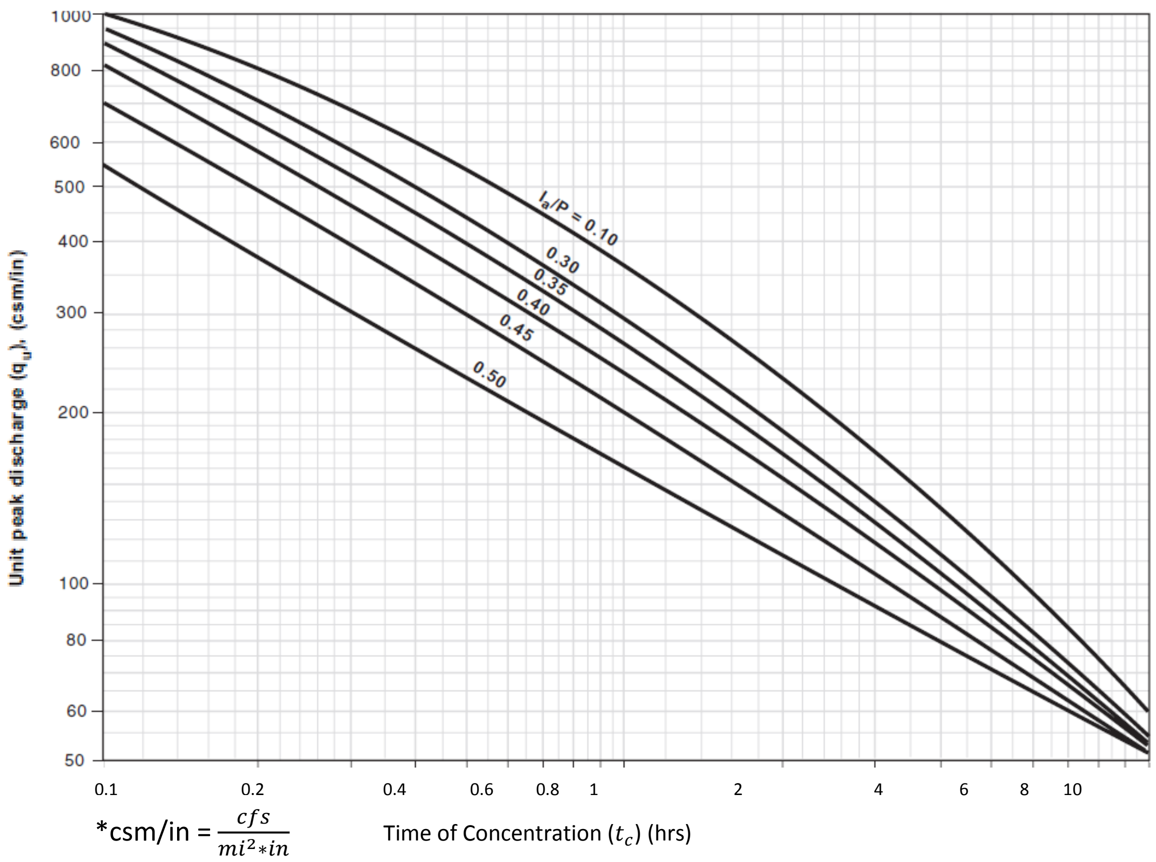

Figure 11.1 Unit peak discharge for type II distribution (NRCS 1986)

Table 11.4 Unit peak discharge (qu) and initial abstraction (Ia) for various curve numbers (CN), for the water quality volume (WQV), and scour check (10-yr) storms (NRCS 1986)

CN	Ia^{a} (in)	$\frac{I_a}{0.75\,in}^{b}$ (WQV)	$\frac{I_a}{5\,in}^{d}$ (10 yr)	qu ^{c} (10 yr) $\left(\frac{cfs}{mi^2 * in}\right.$	CN	Ia^{a} (in)	$\frac{I_a}{0.75\,in}^{b}$ (WQV)	qu ^{c} (WQV) $\left(\frac{cfs}{mi^2 * in}\right.$	$\frac{I_a}{5\,in}^{d}$ (10 yr)	qu ^{c} (10 yr) $\left(\frac{cfs}{mi^2 *}\right.$	CN	Ia^{a} (in)	$\frac{I_a}{0.75\,in}^{b}$ (WQV)	qu ^{c} (WQV) $\left(\frac{cfs}{mi^2 * in}\right.$	$\frac{I_a}{5\,in}^{d}$ (10 yr)	qu ^{c} (10 yr) $\left(\frac{cfs}{mi^2 * in}\right.$
40	3.00	4.00	0.60	550	60	1.33	1.78	-	0.27	965	80	0.50	0.67	550	0.10	1000
41	2.88	3.84	0.58	550	61	1.28	1.71	-	0.26	965	81	0.47	0.63	550	0.09	1000
42	2.76	3.68	0.55	550	62	1.23	1.63	-	0.25	965	82	0.44	0.59	550	0.09	1000
43	2.65	3.53	0.53	550	63	1.18	1.57	-	0.24	970	83	0.41	0.55	550	0.08	1000
44	2.55	3.39	0.51	550	64	1.13	1.50	-	0.23	970	84	0.38	0.51	550	0.08	1000
45	2.44	3.26	0.49	580	65	1.08	1.44	-	0.22	970	85	0.35	0.47	600	0.07	1000
46	2.35	3.13	0.47	610	66	1.03	1.37	-	0.21	975	86	0.33	0.43	720	0.07	1000
47	2.26	3.01	0.45	700	67	0.99	1.31	-	0.20	975	87	0.30	0.40	795	0.06	1000
48	2.17	2.89	0.43	720	68	0.94	1.25	-	0.19	975	88	0.27	0.36	880	0.05	1000
49	2.08	2.78	0.42	800	69	0.90	1.20	-	0.18	975	89	0.25	0.33	910	0.05	1000
50	2.00	2.67	0.40	815	70	0.86	1.14	-	0.17	980	90	0.22	0.30	955	0.04	1000
51	1.92	2.56	0.38	840	71	0.08	0.11	-	0.02	980	91	0.20	0.26	965	0.04	1000
52	1.85	2.46	0.37	880	72	0.78	1.04	-	0.16	980	92	0.17	0.23	970	0.03	1000
53	1.77	2.37	0.35	900	73	0.74	0.99	-	0.15	985	93	0.15	0.20	975	0.03	1000
54	1.70	2.27	0.34	910	74	0.70	0.94	550.00	0.14	985	94	0.13	0.17	980	0.03	1000
55	1.64	2.18	0.33	925	75	0.67	0.89	550.00	0.13	990	95	0.11	0.14	985	0.02	1000
56	1.57	2.09	0.31	935	76	0.63	0.84	550.00	0.13	990	96	0.08	0.11	990	0.02	1000
57	1.51	2.01	0.30	950	77	0.60	0.80	550.00	0.12	995	97	0.06	0.08	1000	0.01	1000
58	1.45	1.93	0.29	950	78	0.56	0.75	550.00	0.11	995	98	0.04	0.05	1100	0.01	1100
59	1.39	1.85	0.28	960	79	0.53	0.71	550.00	0.11	995						

a) Initial abstraction is a function of the CN and was found in TR-55 (NRCS 1986)

b) Initial abstraction to precipitation ratio for WQV (0.75 in.) rainfall

c) Determined from Figure with a t_c= 5 min and the corresponding $\frac{I_a}{P}$ value

d) Initial abstraction to precipitation ratio for 10-yr (5 in) rainfall

Table 11.5 Curve numbers with their associated runoff depths for 0.75 in. rainfall (WQV)

CN	Q^a (in)	CN	Q^a (in)
≤73	0	86	0.088
74	0.001	87	0.105
75	0.002	88	0.124
76	0.004	89	0.145
77	0.007	90	0.170
78	0.011	91	0.198
79	0.017	92	0.230
80	0.023	93	0.266
81	0.030	94	0.307
82	0.039	95	0.355
83	0.049	96	0.410
84	0.060	97	0.475
85	0.073	98	0.551
a) Calculated using equation 11.1			

The unit peak discharge (q_u) is a function of the time of concentration, the $\frac{I_a}{P}$ ratio, and the rainfall distribution type. The time of concentration is dependent on watershed characteristics and is defined as the time it takes for water to move from the hydraulically most distant point in the watershed to the outlet. The $\frac{I_a}{P}$ ratio was determined by dividing the initial abstraction (I_a), which can be found in Table , by the total precipitation (0.75 in. for the design storm). The entire state of Nebraska falls within the type II rainfall distribution.

For runoff from impervious areas and rainfall depth of 0.75 in. the $\frac{I_a}{P}$ ratio is ~0.055. This value, along with time of concentration, was then used to determine q_u from Figure . Using a conservative 5 min time of concentration, q_u was found to be approximately 1100 $\frac{cfs}{mi^2*in}$. This value was extrapolated from Figure .

The accuracy of this method will be reduced for values of $\frac{I_a}{P}$ outside of the range shown on Figure . If the values fall outside of this range use the tabular hydrograph method as stated in the TR-55 manual (NRCS 1986). There are also several software packages which are equipped to perform these calculations for complicated basins.

When considering a watershed with both impervious and pervious ground cover, the area can either be considered completely impervious, or a weighted flow may be calculated. Assuming total imperviousness would result in larger than actual flows and, therefore, oversized BMPs. For this reason the weighted flow method is recommended.

When using the weighted flow method, consider the impervious and pervious sections of the watershed individually and sum the resulting peak flows from each section. This method differs from the weighted curve number method by taking into account the runoff which flows directly from the impervious area to the BMP without first encountering the pervious area. The weighted flow method results in larger flows which are more realistic in many roadway scenarios.

Peak ten year flow rates. The peak ten-year (scouring) flow rate will be used for scour checks in coordination with storm sewer sizing for expressways (NDOR 1996) and is calculated using equation 11.3. The equation 11.3 variables associated with the 10-yr storm can be found in these locations:

- q_u can be found on Table or Figure
- Q can be found on Table
- F_P is 1.0 for Nebraska
- A_m is site-specific

When calculating the 10-yr scour flow a 5-in. rainfall will be used. The 5 in. rainfall represents the highest peak precipitation in the state of Nebraska for the 10-yr storm. Using the largest rainfall event will result in adequate or conservative sizing across the state. Similarly to the WQV calculations, the weighted flow method should be used.

Table 11.6 Curve numbers with their associated runoff depths for 5 in. rainfall (10-yr storm)

CN	Q^a (in)	CN	Q^a (in)	CN	Q^a (in)	CN	Q^a (in)
31	0.01	48	0.59	65	1.65	82	3.08
32	0.03	49	0.64	66	1.73	83	3.17
33	0.04	50	0.69	67	1.80	84	3.27
34	0.06	51	0.75	68	1.88	85	3.37
35	0.08	52	0.80	69	1.96	86	3.47
36	0.11	53	0.86	70	2.04	87	3.57
37	0.14	54	0.92	71	2.12	88	3.67
38	0.17	55	0.98	72	2.20	89	3.77
39	0.20	56	1.04	73	2.28	90	3.88
40	0.24	57	1.10	74	2.36	91	3.98
41	0.27	58	1.17	75	2.45	92	4.09
42	0.31	59	1.23	76	2.54	93	4.20
43	0.35	60	1.30	77	2.62	94	4.31
44	0.40	61	1.37	78	2.71	95	4.42
45	0.44	62	1.44	79	2.80	96	4.53
46	0.49	63	1.51	80	2.89	97	4.65
47	0.54	64	1.58	81	2.99	98	4.76

a) Calculated using equation 11.1

11.3.1.4 Calculating the Water Quality Volume

Water quality volumes for impervious surfaces, such as pavement, up to 5 acres have been calculated and displayed in

Table . For pervious areas or areas larger than 5 acres, use the following methodology.

The water quality volume is found by multiplying the new development area (e.g., newly constructed roadway) by 0.5 in. (equation 11.4). This volume will then be incorporated into the BMP design.

$$WQV_{New\ Dev} = 0.5in * \frac{Area\ of\ New\ Roadway\ (ft^2)}{12\frac{in}{ft}} \tag{11.4}$$

11.3.1.5 Calculating Run-On Volume

Run-on (WQV_{Run-On}) is water from surfaces (impervious or pervious), other than the new development area, that is co-mingled with water from the new development area. Because run-on co-mingles with the $WQV_{New\ Dev}$, it must be treated in the BMP.

Run-on volume from pervious surfaces during the 0.75 in. rainfall event will result in less than 0.5 in. of runoff. Table shows the runoff depth from a 0.75 in. rainfall for areas with various curve numbers.

For areas that contribute run-on that will co-mingle with the $WQV_{New\ Dev}$, the run-on volume (WQV_{Run-On}) can be calculated by using equation 11.5:

$$\mathrm{WQV_{Run-On}} = \mathrm{Q\ from\ Table\ 8} * \frac{\mathrm{Contributing\ area(ft^2)}}{12\frac{\mathrm{in}}{\mathrm{ft}}} \tag{11.5}$$

The total water quality volume (WQV_{Total}) is the sum of the runoff from new impervious areas ($WQV_{New\ Dev}$) and run-on (WQV_{Run-on}) as shown in equation 11.6.

$$\mathrm{WQV_{Total}} = \mathrm{WQV_{New\ Dev}} + \mathrm{WQV_{Run-on}} \tag{11.6}$$

This volume is the minimum amount of water to be treated.

In-line BMPs need to be designed to either handle the flow of larger storms, or they need to be able to bypass larger flows. For offline BMPs the WQV from the new roadway to be treated must be routed through the BMP.

11.3.2 Design Example

The urban highway in Figure is being redeveloped. The redeveloped highway contributes 3.3 acres which will contribute to the water quality volume and peak flows. There are vegetated areas north and south of the highway that account for 4 acres of extra drainage (run-on) to the system.

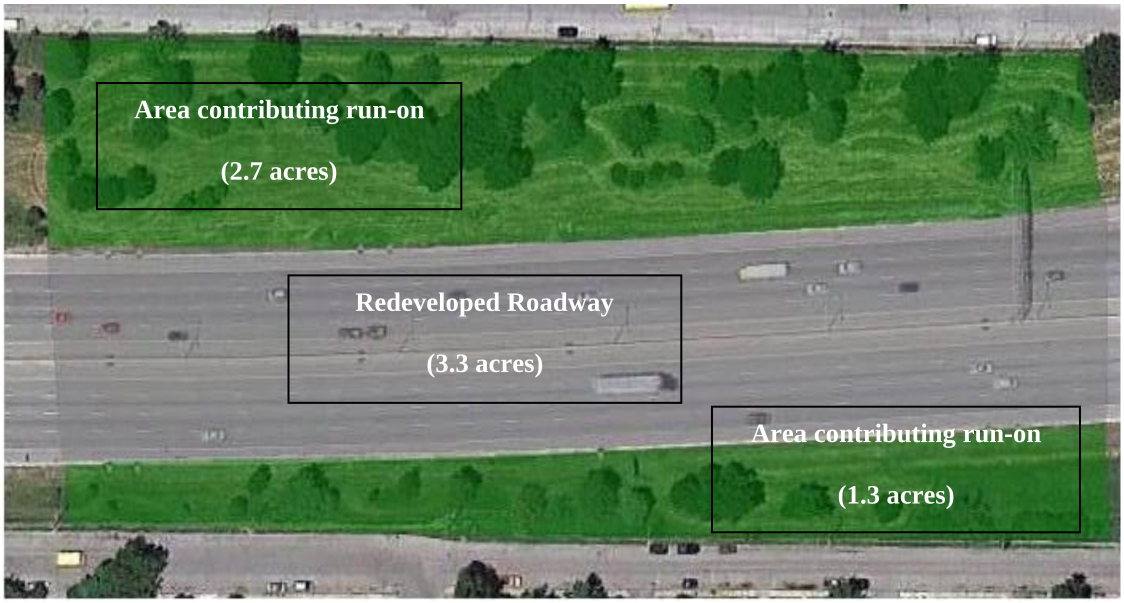

Figure 11.2 Plan view of redeveloped highway

11.3.2.1 Calculating Peak Water Quality Flow Rates

The WQV and peak WQV flow rate were calculated using precipitation of 0.75 in. which corresponds to 0.5 in. of runoff from impervious surfaces. The peak water quality flow rate was found by summing the peak flow coming off the 3 subbasins within the system. The individual peak flow rates were found using equation 11.3.

The curve number for each section is given in Table . The CN was used to determine the runoff depth (Q) from Table . The redeveloped roadway is paved, so it has a curve number of 98 according to Table . The vegetated sections were considered open space with grass cover of greater than 75% in soil type C, as described by Table , so they each have a curve number of 74. Table shows that a curve number of 98 produces a runoff, Q, of 0.551 in. for the design 0.75 in. rainfall, and a curve number of 74 produces a runoff depth, Q, of 0.001 in. for the design 0.75 in. rainfall.

The unit peak discharge (q_u) can be found on Table , or it can be determined by using the initial abstraction (I_a) in the ratio of initial abstraction (I_A) to precipitation (P) $\left(\frac{I_a}{P}\right)$, found on Table , along with Figure and an assumed time of concentration (t_c). A curve number of 98 results in a q_u of 1100 $\frac{cfs}{mi^2*in}$, and a curve number of 74 results in a q_u of 550 $\frac{cfs}{mi^2*in}$ for the WQV rainfall.

The swamp adjustment factor $F_p$is assumed to be 1 for the state of Nebraska. Equation 11.3 ($q_p = q_u A_m Q F_p$) is then solved for each area, these values are given for impervious areas in

Table but must be calculated for the pervious areas.

Redeveloped roadway:

$$q_p = 1100\ \frac{cfs}{mi^2 * in} * 0.005\, mi^2 * 0.551\ in * 1 = 3.03 \text{ cfs}$$

2.7 acre vegetated area:

$$q_p = 550\ \frac{cfs}{mi^2 * in} * 0.004 mi^2 * 0.001\ in * 1 = 0.0022 \text{ cfs}$$

1.3 acre vegetated area:

$$q_p = 550\ \frac{cfs}{mi^2 * in} * 0.002\ mi^2 * 0.001\ in * 1 = 0\ .0011 \text{ cfs}$$

The flows from each area were then summed to find the peak WQV flow of the drainage area.

$$3.03\ cfs + 0.0022\ cfs + 0.0011\ cfs = 3.03\ cfs$$

The peak WQV flow was found to be 3.03 cfs.

Table summarizes calculations for peak water quality flow.

Table 11.7 Calculations for peak WQV flow

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
A (acres)	P (in)	CN	${t_c}^a$ (min)	I_a (in)	$\frac{I_a}{P}^a$	${q_u}^b$ $\left(\frac{cfs}{mi^2 * in}\right)$	Q^c (in)	Area (A) (mi^2)	F_p	${q_p}^d$ (cfs)
3.3	0.75	98	5	0.04	0.053	1100	0.551	0.005	1	3.031
2.7	0.75	74	5	0.7	0.933	550	0.001	0.004	1	0.002
1.3	0.75	74	5	0.7	0.933	550	0.001	0.002	1	0.001
a) Use with Figure to find q_u b) Found with Figure or Table										q_p Total (cfs)
c) Found in Table d) (7)*(8)*(9)*(10)										3.03

11.3.2.2 Calculating Peak Flow Rates for Scour Evaluation

The peak flow rate was used to evaluate the need for scour protection in flow-through BMPs. It was found by summing the peak flow coming from the 3 subbasins within the system from the 10-yr (5-in.) storm. The individual peak flow rates were found using equation 11.3 ($q_p = q_u A_m Q F_p$).

The curve number (Table) for each section was used to find the runoff depth (Q) from Table . The redeveloped roadway is paved, so it has a curve number of 98 and a Q of 4.76 in. The vegetated sections are considered open space with grass cover of greater than 75% in soil type C, as described by Table , so they each have a curve number of 74 and a Q of 2.36 in..

The unit peak discharge (q_u) can be found on Table or can be determined by using the initial abstraction (I_a) in the ratio of initial abstraction (I_a) to precipitation (P) $\left(\frac{I_a}{5\ in}\right)$, found on Table , along with Figure and an assumed time of concentration (t_c). A curve number of 98 resulted in a q_u of 1100 $\frac{cfs}{mi^2 * in}$, and a curve number of 74 resulted in a q_u of 985 $\frac{cfs}{mi^2 * in}$ for the 10-yr rainfall.

The swamp adjustment factor $F_p$was assumed to be 1 for the state of Nebraska.

Equation 11.3 ($q_p = q_u A_m Q F_p$) was then solved for each area.

Redeveloped roadway:

$$q_p = 1100 \ \frac{cfs}{mi^2 * in} * 0.005 \ mi^2 * 4.76 \ in * 1 = 26.2 \text{ cfs}$$

2.7 acre vegetated area:

$$q_p = 985 \ \frac{cfs}{mi^2 * in} * 0.004 mi^2 * 2.36 \ in * 1 = 9.3 \text{ cfs}$$

1.3 acre vegetated area:

$$q_p = 985 \ \frac{cfs}{mi^2 * in} * 0.002 \ mi^2 * 2.36 \ in * 1 = 4.6 \text{ cfs}$$

The flows from each area are then summed to find the peak flow of the drainage area.

$$26.2 \ cfs + 9.3 \ cfs + 4.6 \ cfs = 40.1 \ cfs$$

The peak flow was found to be 40.1 cfs.

Table **2** summarizes the calculations for peak scour flow.

Table 2.8 Calculations for peak scour flow

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
A (acres)	P (in)	CN	$t_c{}^a$ (min)	I_a (in)	$\frac{I_a}{P}{}^a$	$q_u{}^b$ $\left(\frac{cfs}{mi^2 * in}\right)$	Q^c (in)	Area (A) (mi^2)	F_p	$q_p{}^d$ (cfs)
3.3	5	98	5	0.04	0.008	1100	4.76	0.005	1	26.180
2.7	5	74	5	0.7	0.140	985	2.36	0.004	1	9.298
1.3	5	74	5	0.7	0.140	985	2.36	0.002	1	4.649
a) Use with to find q_u b) Find with or Table c) Found in Table										q_p Total (cfs)
d) (7)*(8)*(9)*(10)										40.1

11.3.2.3 Calculating Water Quality Volume

The total water quality volume (WQV) is the sum of the WQV from new development (e.g. pavement) ($WQV_{New\ Dev}$) and the volume of run-on which co-mingles with the $WQV_{New\ Dev}$ (WQV_{Run-On}). $WQV_{New\ Dev}$ was found by multiplying the newly constructed or redeveloped area by 0.5 in. using equation 11.4.

$$WQV_{New\ Dev} = 0.5in * \frac{Area\ of\ New\ Roadway\ (ft^2)}{12\frac{in}{ft}}$$

For this example, the area of newly developed pavement was 3.3 acres (143,747 ft^2). Thus the $WQV_{New\ Dev}$ was:

$$WQV_{New\ Dev} = 0.5in * \frac{143{,}747\ ft^2}{12\frac{in}{ft}} = 5{,}989ft^3$$

In order to calculate the run-on volume (WQV_{Run-On}) the depth of runoff (Q) from the 0.75 in. storm must be found for the associated curve numbers of the contributing areas determined by Table . This value was incorporated into equation 11.5 to find the WQV_{Run-On}.

$$WQV_{Run-On} = Q\ from\ \mathbf{Table}\ * \frac{Contributing\ area(ft^2)}{12\frac{in}{ft}}$$

The WQV_{Run-On} for the 2.7 acre vegetated area was:

$$WQV_{Run-On} = 0.001\ inches * \frac{117{,}611ft^2}{12\frac{in}{ft}} = 9.8\ \ ft^3$$

The WQV_{Run-On} for the 1.3 acre vegetated area was:

$$WQV_{Run-On} = 0.001\ inches * \frac{56{,}628ft^2}{12\frac{in}{ft}} = \ 4.7\ ft^3$$

Equation 11.6 was then used to find the total volume requiring treatment.

$$5{,}989ft^3 + 9.8\ \ ft^3 + 4.7\ ft^3 = 6{,}004\ ft^3$$

Chapter 12 Volume II Results and Discussion

12.1 Fact Sheets

The fact sheets provide the design engineer the background on each BMP which will be used to determine the applicability of a specific BMP or determine which BMP is best for site-specific conditions. Each fact sheet typically includes the following:

- Description: Provides a basic description of the BMP.
- Pollutant removal potential: Shows pollutant removal based on multiple studies.
- Initial costs: Provides projected capital costs and costs observed during case studies.
- Maintenance costs: Provides estimates and case study results of maintenance costs as well as required maintenance hours.
- Siting constraints: Identifies applicable locations and conditions for the BMP.
- Maintenance and operation considerations: Identifies ways to prevent and repair potential problems with the BMP.

12.1.1 Vegetated Filter Strip

Figure 12.1 Roadside vegetated filter strip (TWG 2008)

12.1.1.1 Description

Vegetated filter strips, also known as vegetated buffers or grass filter strips, are sloped vegetated surfaces which are intended to treat runoff from adjacent impervious areas. These areas must have sufficient vegetative cover and minimal slope perpendicular to flow (cross slope) to facilitate treatment. Treatment of runoff is accomplished primarily through filtration, biological processes associated with the vegetation, and infiltration.

The primary requirement with vegetated filter strips is maintaining sheet flow. If runoff is allowed to channelize there are two primary drawbacks. The first drawback is the formation of rills, which can occur when concentrated flows locally erode surface soils. This eroded material then adds to the solids load of the runoff. The second problem comes from short-circuiting associated with rill formation. Rills allow runoff to bypass the vegetation where treatment occurs. Vegetated filter strips become largely ineffective if channelization is allowed to occur.

One way to maintain sheet flow is through the use of a level spreader. Level spreaders are used to slow and evenly distribute runoff. Roadside level spreaders include gravel filled trenches, earthen berms, rip-rap, or treated lumber which have minimal cross slope. It is recommended to use level spreaders at the top of the buffer.

12.1.1.2 Pollutant Removal Potential

Vegetated filter strips primarily remediate runoff through filtration, biological processes, and infiltration. High solids removal has been shown in the first 13 ft (4 m) of the strip (Barrett 2005), and it plateaus after 33 ft (10 m) (Zhang et al. 2010). The slope should not exceed 15% to keep velocities low and pollutant removal high. Removal of solids peaks at 10% slope, though has been shown to be effective at steeper slopes (Zhang et al. 2010). Table 3 shows observed pollutant removals by vegetated filter strips.

Table 3.1 Pollutant removal potential for vegetated filter strips

	Zhang et al.[1] 2010	Li et al. 2008	Caltrans 2004	Winston and Hunt [3] 2010	Barrett and Walsh [4] 1998
	Removal %	Removal %	Removal %	Removal %	Removal %
Pollutant					
Total Suspended Solids (TSS)	86	35.7	83	68	85
Total Nitrogen (TN)	68.3	4.7	44	13	48
Total Phosphorous (TP)	71.9	-121	-76	12	45
Total Metals (TM)	-	49.7	89.3[2]	-	63[5]

1) Results of a literature review
2) Average of Cu, Pb, & Zn
3) Average of 6 sites
4) Load reduction of existing infrastructure
5) Average of Zn, PB, & Fe

12.1.1.3 Cost Considerations

Initial cost. The small amount of design and infrastructure associated with vegetated filter strips makes them a relatively inexpensive BMP. The construction costs of vegetated buffers included grading, vegetating the strip, and installation of a level spreader. The cost of grass installation has been estimated at $13,000 per acre for seeding and $30,000 per acre for sod as of 2006 (EPA 2006d). Level spreader costs range from $5 to $20 per foot as of 2006 (DEPBWM 2006), and grading costs vary with site size and conditions.

Another major expense was the availability of the land required to place this BMP. The large foot print can make vegetated filter strips impractical in urban areas where acquiring the necessary land is expensive. However, it has been shown that existing vegetation along roadways can act as vegetated filters (Barrett 2005). Sites which are already acting in this capacity require very little initial capital.

Maintenance costs. Maintenance costs are also low with vegetated filter strips. Annual maintenance costs have been estimated at $350 per acre of filter strip based on a report from 1991 (EPA 2006d). A study (CalTrans 2004) demonstrated that the majority of maintenance overlapped with general roadside maintenance. A related study was performed which showed that the pollutant removal effectiveness of existing roadside vegetation, which had only regular maintenance, compared favorably with filter strips designed for water quality improvement (CalTrans 2003).

12.1.1.4 Siting Constraints

Vegetated filter strips are applicable for use in most areas, and are effective as pretreatment BMPs in a treatment train. Runoff from small areas, such as parking lots or roadways, is a good candidate for treatment by vegetated strips. However, the relatively large spatial requirement of filter strips is a major restricting factor.

Although filter strips are suited for most climates, they may need some climate-specific considerations. For example, in cold or seasonal climates vegetation should be selected that is salt tolerant, especially when adjacent to roadways. In more arid regions lack of rainfall may require irrigation to maintain acceptable vegetated cover which may make vegetated filter strips cost-prohibitive.

The large size requirement creates the potential for the required width of the buffer to extend beyond the standard right of way. Intrusion on neighboring properties causes an increase in cost which may limit the practicality of filter strips. Other BMPs, with a smaller footprint, may be better suited for densely developed areas.

Another constraining factor is the requirement for minimal slope perpendicular to flow. This is of particular concern for some roadway applications because the land adjacent to roads generally has a similar topography. Highways which have vertical curves of more than 2% will likely not be able to effectively accommodate a vegetated filter strip. Other forms of vegetated filtration such as vegetated swales may be considered in these areas. See the Vegetated Swale Fact Sheet of this work to determine their applicability.

There must be safe access to all parts of the filter strip. Due to the nature of this BMP, maintenance vehicle access at the top of the slope should be sufficient for the majority of maintenance activities. Any necessary vehicle traffic on the strip should occur when the ground is dry, and vehicles should travel horizontally across the strip as much as possible. The ruts formed decrease vegetated cover which can reduce the performance of this BMP, and if the ruts are created running down-hill they will promote channelization.

12.1.1.5 Maintenance and Operation Considerations

It is very likely that much of the cost of operation and maintenance will overlap with general vegetation maintenance along roadways (Barrett 2005). The primary focuses of maintenance are maintaining healthy vegetation, removing litter and detritus, and the preservation of sheet flow throughout the length and width of the filter strip. Maintaining healthy vegetation consists of keeping vegetated cover above 80%. This should be done, as much as possible, without the use of pesticides or herbicides which can contribute to contaminants in the runoff.

In a retrofit study, the California Department of Transportation (Caltrans 2004) found that 105 hrs/yr were required to maintain an effective filter strip serving 4.9 acres. Sixty-seven of these hours were spent mowing and removing woody vegetation, which are standard roadside maintenance activities.

Table shows the potential maintenance and operation requirements of vegetated filter strips which could be observed during inspection and suggests corrective procedures.

Table 12.2 Operations and maintenance considerations and suggested corrective procedures

Inspection Frequency	Problem	Suggested Corrective Procedure
Annually	Level spreader is not distributing runoff evenly across strip due unevenness or clogging.	Level the flow spreader and clean out clogs (NCDENR 2007).
	Substantial channelization or rilling.	Regrade and reseed the strip (DEPBWM 2006).
Semi-Annually	Burrowing animals cause vegetated cover to drop below 80%.	Take applicable action which will vary with pest type.
	Sediment accumulation of 3 or more inches near outlet or enough to cover vegetation within the strip.	Remove sediment, re-level, and replant where applicable.
Regularly/ As Needed	Grass becomes unacceptably tall.	Maintain grass length from 2”–6”. Clippings should be removed if nutrients are pollutants of concern. Mowing should be performed across the slope when it is dry so rutting caused by tires will not promote channelization (DEPBWM 2006).
	Weeds or unwanted vegetation begin to dominate strip.	Weeds should be removed by hand ideally, otherwise a herbicide which is not toxic to recommended vegetation should be used (NCDENR 2007).
	Bare areas form within strip.	Remulch and reseed bare areas.
	Rills of less than 8” wide form.	Fill rills with gravel which will soon be overtaken by grass (SEMCOG 2008).
	Litter and detritus build up.	Remove litter which is aesthetically unpleasant, negatively affects performance of the strip, or is itself harmful to the environment (CalTrans 2004).
	Not enough rainfall to sustain vegetation.	Irrigation may be necessary to maintain adequate cover. It is suggested that grasses be selected which are drought tolerant and will not require irrigation.
	Standing water beyond 48 hrs of isolated storm event.	Repair grade where runoff pools and take any necessary vector control measures (SEMCOG 2008).

12.1.2 Vegetated Swale

Figure 12.2 Roadside swale (CalTrans 2012)

12.1.2.1 Description

Vegetated swales are open channels which have vegetative (usually grass) linings that provide water quality benefits while conveying stormwater runoff. Swales rely on maintaining low flow velocities to promote sedimentation, filtration through the vegetation, and infiltration. The low velocity also decreases peak runoff rates from impervious drainage areas. The vegetated channels also have more aesthetic appeal than rock or concrete lined channels.

Swales can be enhanced with check dams to reduce flow velocity and to create temporary ponding which promote sedimentation and infiltration. Check dams can improve the functionality of the BMP as well as increase the life span of vegetated swales (Landphair et al. 2000).

12.1.2.2 Pollutant Removal Potential

Vegetated swales have shown good removal for solids and metals and moderate removal for nutrients, such as phosphorous and nitrogen (UDFCD 2010). Pollutant removals by vegetated swales, as reported by several researchers, are presented in

Table .

Table 12.3 Pollutant removal potential for vegetated swales

	Landphair [1] et al. 2000	MSSC 2005	CalTrans 2004[3]	DEPBWM 2006	Clar et al. 2004
Pollutant	Removal %	Removal %	Removal %	Removal %	Removal %
Total Suspended Solids (TSS)	81–98	85	76	50	83
Total Nitrogen (TN)	40–99	35	67	50	25
Total Phosphorous (TP)	18–99	50	1	20	29
Total Metals (TM)	78.5[2]	80	85[4]		59[5]
Hydrocarbons (oil and grease)	-	80	-		75

1) Average of 6 sites
2) Zn: 60–99; Pb: 50–99
3) Average of 6 sites
4) Average of Pb, Cu, and Zn
5) Average of Pb, Cu, and Zn

12.1.2.3 Cost Considerations

Initial cost. Initial capital costs for vegetated swales are generally low. Existing infrastructure should be used as much as possible to keep costs low. In many cases it is possible to meet municipal separate storm sewer discharge permit requirements as specified in the Clean Water Act Section 401 by adding check dams to existing drainage measures (Landphair et al. 2000). Construction costs can result from swale size, grading, clearing, grubbing, or plant establishment. The EPA has predicted swale construction costs to range from \$0.25 to \$0.50 per ft^2 (\$2.75 to \$5.50 per m^2) based on a report from 1997 (EPA 2006b).

The Pennsylvania Stormwater BMP Manual reported costs of \$8.50 to \$50 per linear foot (\$28 to \$165 per meter) in 2006 (DEPBWM 2006). The Michigan LID Manual predicts costs ranging from \$4.50 to \$8.50 per linear foot (\$15 to \$28 per meter) for seeding and \$15 to \$20 per linear foot (\$50 to \$66 per meter) for sodding as of 2008 (SEMCOG 2008). These values compare favorably to capital costs for underground pipes (\$2 per foot per inch of diameter) and curb and gutter systems (\$13 to \$15 per foot) (SEMCOG 2008).

Another method of cost estimation is based on cost per volume treated. Cost per volume can range from \$0.50 per ft^3 (\$18 per m^3) (CH2MHILL 2008) to \$1.50 per ft^3 (\$52 per m^3) (CalTrans 2004). The cost of the swale per volume treated can vary based on the size of the contributing watershed and the scope of the construction project. Although these values are good for estimation, larger drainage areas have been shown to have lower costs per volume treated (CalTrans 2004), so a linear relationship may not be reliable. Construction costs can also be mitigated by constructing the swale in conjunction with other construction activities within a larger project (Lampe et al. 2005).

Maintenance costs. Vegetated swales are considered to have a low life cycle cost when compared to other BMPs (UDFCD 2010). Annual maintenance costs for swales are expected to be 5% to 7% of the construction costs (CH2MHILL 2008). This estimate fits with a 2004 study which projects \$2,736 of annual maintenance for a swale serving 6 acres (CalTrans 2004).

12.1.2.4 Siting Constraints

Vegetated swales are useful along roadways, parking lots, and as components of treatment trains (KCDNRP 2009). Their linear nature and combination of drainage and water quality benefits make them ideal for use along roadways (KCDNRP 2009). Existing drainage areas within the right-of-way, such as ditches and medians, are often compatible with the use of vegetated swales. Existing drainage infrastructure (e.g., ditches) may already be functioning as a vegetated swale, but any retrofit project requires confirmation with the constraints laid out in the Vegetated Swale Design Guide section of this work. When using swales along roadways, they can effectively replace the curb and gutter system (UDFCD 2010).

Contributing drainage area also limits the applicability of what vegetated swales are best suited for. Ideally, swales will not treat more than 5 acres (SEMCOG 2008). However, guidance of up to 10 acres has been given (Clar et al. 2004). If treating more than 5 acres, less than 5 acres of the contributing area should be impervious (KCDNRP 2009).

If vegetated swales have a gentle slope (i.e., < 1%) they should not be used where the seasonal high watertable, or bedrock is within 2 feet (0.61 m) of the bottom of the swale. Building the swale with inadequate drainage considerations could result in dewatering problems which can lead to mosquito breeding grounds (SEMCOG 2008). Dewatering is also a concern with NRCS type D (i.e., clay) soils (Landphair et al. 2000). Swales may still be used in type D soils, but an adequate slope (i.e., greater than 1%) must be maintained throughout the course of the swale to facilitate drainage. When considering swales for urban or residential applications, the number of driveways crossing the swale must be considered. Driveways crossing the swale require culverts to pass flows. Culverts can reduce pollutant removal by vegetated swales (Clar et al. 2004).

12.1.2.5 Maintenance and Operation Considerations

Maintenance of vegetated swales overlaps significantly with normal vegetated roadside maintenance (Landphair et al. 2000). These maintenance considerations are focused on supporting healthy grass, removing trash, mowing, and keeping woody vegetation down. Additional considerations for water quality swales include sediment removal, preventing and fixing erosion, providing even distribution of flow across the channel, and maintaining check dams (if present). A study found that vegetated swales, when designed properly, should require approximately 50 hrs of maintenance annually for a swale serving 6 acres (CalTrans 2004).

Table 12.4 Operations and maintenance considerations and suggested corrective procedures

Inspection Frequency	Problem	Suggested Corrective Procedure
Annually	Sediment inhibits grass growth in more than 10% of the swale length or inhibits even spread of runoff	Remove sediment by hand or with flat shovel and reseed with same mix as soon as possible (KCDNRP 2009)
	Substantial channelization or rilling.	Regrade and reseed the swale (KCDNRP 2009)
Semi-Annually	Burrowing animals cause vegetated cover to drop below 80%.	Take applicable action which will vary with pest type.
	Sediment accumulation of 3 or more inches near outlet or enough to cover vegetation within the strip.	Remove sediment, re-level, and replant where applicable (Clar et al. 2004, CalTrans 2004)
	Check dam gets clogged with debris or sediment	Remove sediment or debris and reseed with same mix as soon as possible (Landphair et al. 2000)
Regularly/ As Needed	Grass becomes unacceptably tall.	Maintain grass length from 3–4 in. (FHA 1997b). Clippings should be removed if nutrients are concern pollutants (Clar et al. 2004).
	Weeds or unwanted vegetation begin to dominate strip.	Weeds should be removed without using tactics which adversely affect recommended vegetation (CalTrans 2004).
	Rills of less than 8” wide form.	Fill, compact, and reseed eroded area with same seed mix (Clar et al. 2004)
	Litter and detritus build up.	Remove litter which is aesthetically unpleasant, negatively affects performance of the swale, or is itself harmful to the environment (FHA 1997b).
	Not enough rainfall to sustain vegetation.	Irrigation may be necessary to maintain adequate cover (SEMCOG 2008). It is suggested that grasses be selected which are drought tolerant and will not require irrigation.
	Standing water beyond 48 hrs of isolated storm event.	Repair grade where runoff pools and take any necessary vector control measures (MSSC 2005).

12.1.3 Bioretention Cell

Figure 12.3 Highway median bioretention in Delaware (DelDOT 2012)

12.1.3.1 Description

Bioretention BMPs are highly customizable and flexible vegetated soil filters that are designed to retain and treat the water quality volume (WQV) and filter it through an engineered soil mix. Remediation is accomplished through filtration, plant uptake, and potentially, infiltration. The soil mix must allow the retained runoff to drain in 24 to 48 hrs while performing remediation functions and supporting the vegetation in the system (MDEP 2009).

The vegetation can be very diverse in bioretention; however, using grass as the only vegetation can produce excellent water quality results (Davis et al. 2009). Trees should not be used near roadways due to safety concerns. If vegetation is properly selected and maintained bioretention cells can be very beneficial aesthetically along with their environmental benefits. Vegetation selection and planting strategies are discussed in the Bioretention Design Guide.

Bioretention BMPs can be designed as either infiltration or filtration facilities. Infiltration is encouraged if it does not threaten surrounding buildings or roadways. Infiltrating the WQV contributes to ground water recharge as well as decreasing runoff which could contribute to stream channel erosion. In situations where infiltration is not desirable, an under- drain is used to discharge treated runoff. Under-drain systems are ideal for areas with impermeable soils or in highly developed areas. Figure shows a bioretention facility in a roadway median, and Figure shows a plan and section view of a potential bioretention layout.

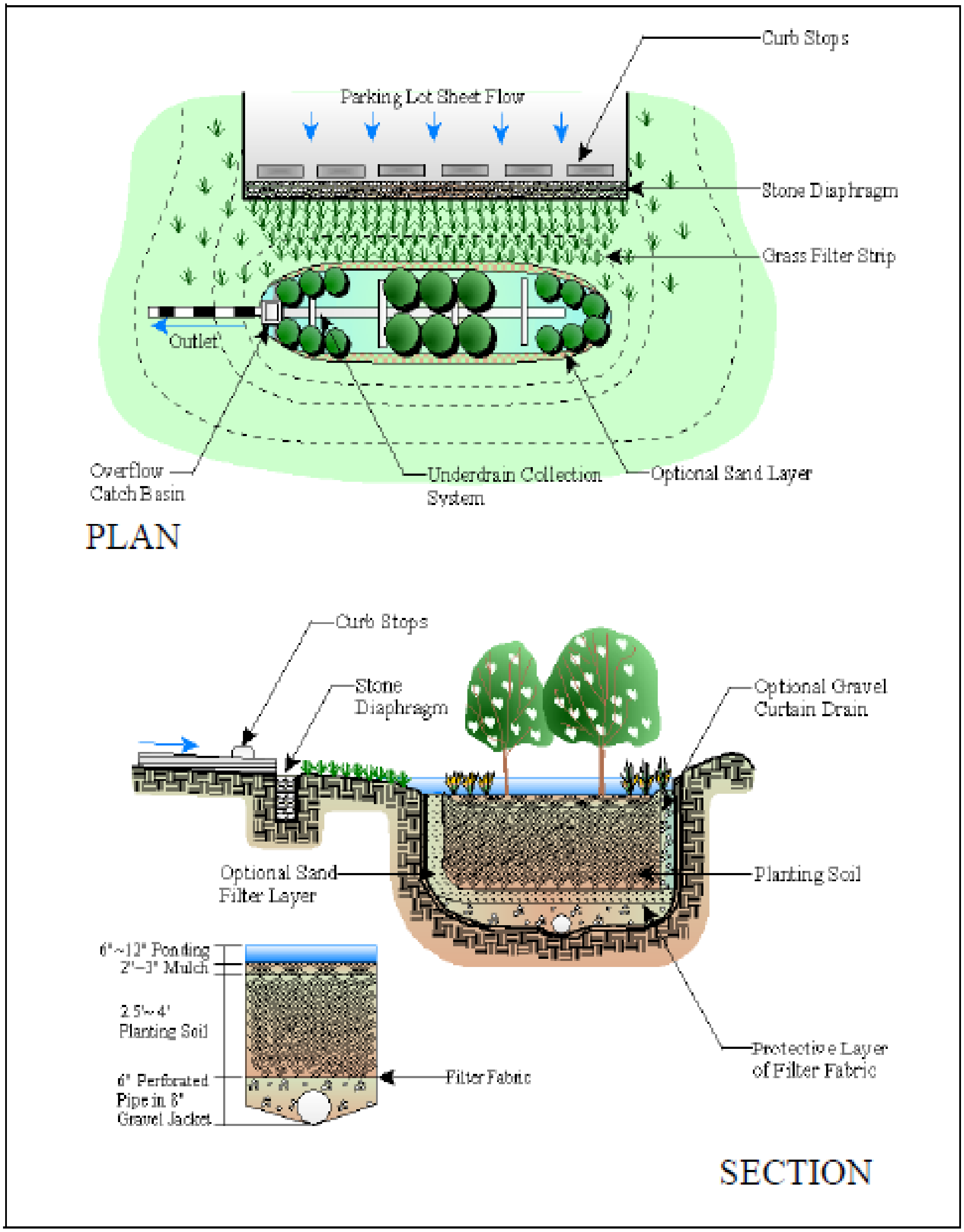

Figure 12.4 Bioretention facility plan and section view (Landphair et al 2000)

12.1.3.2 Pollutant Removal Potential

Pollutant removal was primarily achieved through filtration and uptake from plants and microbials. Solid removal was high but has been shown to plateau at 10 $\frac{\text{mg}}{\text{L}}$ regardless of initial concentration (Lampe et al. 2005). Table 12.5 shows the pollutant removal from several studies.

Table 12.5 Pollutant removal potential for bioretention

	Li and Davis[1] 2009	Atchison et al. 2006	MSSC 2008	Davis et al.[2] 2009	Passeport et al.[7] 2009
Pollutant	Removal %	Removal %	Removal %	Removal %	Removal %
Total Suspended Solids (TSS)	96	90	85	54–99[3]	-
	99				
Total Nitrogen (TN)	-3	65–75	45	32–97[4]	56
	97				47
Total Phosphorous (TP)	-36	80	50	-240–79[5]	53
	100				68
Total Metals (TM)	75	95	95	57–99[6]	
	99				
Fecal Coliforms	95	90	35	-	95
	100				85

1) 2 sites
2) Average of multiple studies
3) 5 studies
4) 5 studies
5) 7 studies
6) 5 studies for Zinc
7) 2 grass only sites

12.1.3.3 Cost Considerations

Initial Cost. Initial capital costs for bioretention facilities are considered low to moderate (WSDOT 2010). The city of Bellingham, Washington installed rain gardens in place of in-ground storage and saved 75% to 80% on construction costs (LeCroix et al. 2004). Bioretention facilities installed in 2004 cost $12,800 to treat 4400 ft^3, which is equivalent to the WQV for 2.4 acres, and $5,600 to treat 2300 ft^3, which is equivalent to the WQV for 1.3 acres (LeCroix et al. 2004). These costs were supported by the EPA who projected new construction of bioretention in commercial areas to be $12,357 and retrofits in commercial areas to be $12,355 for drainage areas no greater than 1 acre in 2004 (Clar et al. 2004). The precise initial capital requirement is site-specific and related to availability of materials, size of contributing drainage area, and necessity of under-drains.

Maintenance Costs. The average expected maintenance cost for bioretention facilities was estimated to be $1,000 annually in 2004 (Lampe et al. 2004). Maintenance will need to be more rigorous, and therefore more costly, until plants can be established.

Maintenance costs can be tempered through community involvement. Because bioretention facilities are aesthetically pleasing, the public may be more prone to embrace and support their use. Community groups or business associations might be willing to participate in maintaining these BMPs. However, inspections and some maintenance activities would still be required.

12.1.3.4 Siting Constraints

The flexibility of bioretention allows it to fit into most water treatment scenarios. Bioretention systems are very diverse and can be altered to site-specific conditions. The primary differentiation between types of bioretention systems are those which infiltrate the runoff and those which do not.

Infiltrating runoff benefits groundwater recharge as well as protects streams from erosion caused by high peak flows. However, infiltration is not always acceptable. The following instances do not allow for infiltration:

- The seasonal high ground water level is within 3 ft (0.9 m) of the bottom of the system (MSSC 2005).
- Treating a pollutant hot spot (i.e., gas station) where groundwater contamination is possible.
- Inadequately drained subgrades (hydraulic conductivity $\leq 0.50\ ^{\mathrm{in}}/_{\mathrm{hr}}$ (1.3 $^{\mathrm{cm}}/_{\mathrm{hr}}$)).
- Potential interference with foundations/infiltration into basements.
- Infiltration interferes with the subgrade of roadways.

For applications that do not permit infiltration, under-drains can be used. Bioretention facilities with under-drains can be used in a wide variety of situations, and can be easily integrated into an urban landscape. When incorporating an under-drain, nearby structures must still be considered. If the bioretention cell is located adjacent to a building, roadway, or sidewalk, a concrete vault should be employed to prevent possibly harmful infiltration.

12.1.3.5 Maintenance and Operation Considerations

Maintenance on bioretention BMPs focuses on keeping the plants healthy and preventing clogging of the filter media. Increased maintenance for these BMPs is required during the vegetation establishment period. Vegetation will require watering in times of little rainfall. Watering should be done weekly for the first 2 to 3 months and bi-weekly during summer months (Hartsig 2009).

Table shows expected maintenance and corrective procedures for the BMP.

Table 12.6 Operations and maintenance considerations and suggested corrective procedures for bioretention cells

Inspection Frequency	Problem	Suggested Corrective Procedure
Annually	Mulch layer thins	Evenly place mulch to a depth of 2–3 in. (5.1–7.6 cm) (Davis et al. 2009)
	Substantial rill formation	Fill rills with washed pea gravel and reconsider pretreatment to better attenuate flow velocity (DEPBWM 2006).
Semi-Annually	Burrowing animals cause vegetated cover to drop below 80%	Take applicable action which will vary with pest type.
	Sediment accumulation in fore-bay (if used)	Remove sediment and dispose of off-site (Clar et al. 2004).
Regularly/As Needed	Undesirable vegetation grows	All weeds and woody vegetation should be removed as soon as possible (SEMCOG 2008).
	Litter and detritus build up.	Remove and discard trash (MSSC 2008).
	Not enough rainfall to sustain vegetation.	Irrigation may be necessary to maintain adequate cover. It is suggested that vegetation be selected which is drought tolerant and will not require irrigation. Watering may be required to establish plants (LeCroix et al. 2004).
	Standing water beyond 48 hrs of isolated storm event.	Tilling the top layer should be done initially. If problems persist, remove filter media and replace with a better draining mix (NCDENR 2007).
	Vegetation becomes overgrown	Prune vegetation according to vegetation-specific requirements (Davis et al. 2009).
	Under drain clogs	Clean out pipes and dispose of sediment off-site (SEMCOG 2008).
	Under drain is damaged	Replace damaged pipe
	Vegetation is dead or diseased	Replace plants. If the plant species seems unsuited for this application select another species (Le Croix et al. 2004).

12.1.4 Basin Sand Filter

Figure 12.5 Sand filter for treatment of highway runoff (CalTrans 2004)

12.1.4.1 Description

Basin sand filters are flow-through BMPs which temporarily detain the water quality volume (WQV) and filter it through sand. Treatment is accomplished primarily through filtration and secondly through sedimentation which occurs in a sedimentation chamber before the runoff is introduced to the filter media. Systems are typically designed for the sedimentation chamber to drain in 24 hrs and the entire WQV to pass through the filter in 40 hrs.

Sand filters are well suited to treat the first flush, but to avoid over-loading they should be designed so that flows in excess of the WQV bypass the system. They should not be used as in-line BMPs. Therefore, flow splitters should be employed upstream of the filter to prevent flows in excess of the WQV from entering the system.

12.1.4.2 Pollutant Removal Potential

Sand filters have been shown to be very effective at removing sediment and metals from stormwater runoff. However, the moderate removal of nutrients provided by the sand filter prevents it from being a stand-alone BMP if discharging into nutrient impaired waterways. Observed removal rates are presented in Table .

Table 12.7 Pollutant removal potential for filter

	SEMCOG 2008	MSSC 2005	CalTrans 2004	NCDENR 2007	Young et al. 1996
Pollutant	Removal %	Removal %	Removal %	Removal %	Removal %
Total Suspended Solids (TSS)	80–92	75–85	90	85	70–86
Total Nitrogen (TN)	30–47	0–35	32	35	31–47
Total Phosphorous (TP)	41–66	0–50	39	45	50–65
Total Metals (TM)	-	45–85	72[2]	-	78–84[4]
Hydrocarbons (oil and grease)	-	80	28[3]	-	-
1) 18 studies 2) Average of Pb, Cu, Zn			3) Average of TPH as oil and diesel 4) Average of Pb and Zn		

12.1.4.3 Cost Considerations

Initial Cost. Sand filters have relatively high construction costs. High costs are due in large part to construction costs for the concrete vaults which house many filters. These costs can be tempered by substituting earthen barriers or prefabricated vaults (SEMCOG 2008). Cost estimates have projected the treatment costs to be $16,000 per impervious contributing acre for filters less than 2 acres in 2002 (FHA 2002b). The cost-benefit of using prefabricated vaults is shown by a study which found costs of approximately $10,000 to treat 0.8 acres in 2008 (SEMCOG 2008).

Contributing watershed size is a major factor in the cost-effectiveness of sand filters. Watersheds greater than 10 acres are suggested to provide the greatest treatment value (Landphair et al. 2000). In 2002 the Federal Highway Administration projected initial filter costs to be $16,000 per impervious contributing acre when treating 2 acres or less and $3,400 per impervious contributing acre for watersheds greater than 5 acres (FHA 2002b).

Construction costs vary widely between studies. In a 2004 retrofit study, construction costs at 5 sites ranged from approximately $200,000 to approximately $315,000. The treated area in these sites ranged from 0.74 to 2.7 acres (CalTrans 2004). These wide ranges make it difficult to project construction costs based on area treated. Site-specific factors, such as excavation requirements can have effects on construction costs, and should be closely assessed when projecting facility costs.

Maintenance Costs. A 2004 retrofit study projects that 43 hrs will be spent servicing filters annually (CalTrans 2004), which corresponds with approximately $2,900 maintenance costs (CalTrans 2004). This budget is projected for years in which the filter media needs to be replaced. Since media rehabilitation is not an annual expense, maintenance costs will be lower on the off years.

12.1.4.4 Siting Constraints

Applicable locations for sand filters include highway medians or within the roadway setbacks (Hubert et al. 2006). When being deployed near roadways, some safety concerns must be addressed. Sand filters or their components can act as fixed object hazards. Impact concerns can be mitigated by minimizing facility heights, employing appropriate setbacks, traffic barriers, and designing the structures to crumple when struck (Hubert et al. 2006).

Roadways and other transportation infrastructure, such as fueling and maintenance stations or park and rides are also ideal contributing watersheds because sand filters perform the best when treating runoff from highly impervious areas (MSSC 2005, DEPBWM 2006, CalTrans 2004). Sand filters may also be designed to occupy limited open space within right-of-ways or in an urban street setting where vegetated BMPs are impractical (Hubert et al. 2006). Although sand filters are adaptable for urban settings, industrial settings may be the most applicable due to a lack of aesthetic appeal compared to bioretention (MSSC 2005).

In order to facilitate gravity flow, and to avoid using pumps, there must be at least 3 ft (1 m) of elevation difference between the inlet of the system and the discharge point (Hubert et al. 2006, CalTrans 2004). The bottom of the facility should be at least 2 ft (0.61 m) above the high groundwater table to prevent possible facility damage and flooding of the under-drain (Hubert et al. 2006, CalTrans 2004). In areas where achieving sufficient heads causes interaction with the groundwater, the facility must be designed with sufficient mass to avoid buoyancy effects (Hubert et al. 2006). Leaching of groundwater into the system can be mitigated by lining the areas beneath the groundwater table with impervious geotextiles or using a concrete vault to house the filter.

12.1.4.5 Maintenance and Operation Considerations

Sand filters should be inspected after the first storm of each year to ensure proper drainage and system functions (KCDNRP 2009). Inspections of contributing area should also be performed. If the contributing area is unstable or erosive the maintenance for the sand filter will be more intensive (Hubert et al. 2006). Removal of the top 2 to 5 in. (50 to 125 mm) of filter media is generally required every 3 to 5 yrs for properly designed filters (Landphair et al. 2000, MSSC 2005). The maintenance burden will be lower for contributing drainage areas with higher impervious areas, as there are typically fewer fines in the runoff (FHA 2002b).

Table shows potential operations and maintenance issues along with suggested procedures to correct them.

Table 12.8 Operations and maintenance considerations and suggested corrective procedures

Inspection Frequency	Problem	Suggested Corrective Procedure
Annually	Filter bed is not draining in design time	Manually manipulate surface, if this is inadequate remove top 2 to 5 in. (50 to 125 mm) and replace (if removal drops media depth under 18 in. (460 mm)). (MSSC 2005)
	Substantial channelization or rilling.	Fill any rills with sand and ensure level spreader is not clogged or damaged (KCDNRP 2009). If level spreader is in working order add erosion protection. (NCDENR 2007)
	Flow spreader is clogged or damaged	For clogs remove and dispose of sediment. For damage make necessary repairs or replace depending on severity. (NCDENR 2007)
Semi-Annually	Surface of media has hardened	Rake to break up surface. (Huber et al. 2006, SEMCOG 2008)
	Deterioration, spalling, or cracking of concrete	Patch damaged area. (Huber et al. 2006, MSSC 2005)
	6 in. (150 mm) or more of sediment built up in sedimentation chamber	Remove sediment. (MSSC 2005, Landphair et al. 2000)
Regularly/As Needed	Under-drains are clogged	Flush out under-drains (NCDENR 2007)
	Litter and detritus build up.	Remove litter and detritus. (NCDENR 2007)
	Contributing area is erosive	Stabilize contributing area. (Hubert et al. 2006)
	Flow diversion structure (if used) is clogged or damaged	For clogs remove and dispose of sediment. For damage make necessary repairs or replace depending on severity. (NCDENR 2007)
	Runoff is short circuiting the filter	Check clean out pipes and ensure there are no leaks in the filter or sediment chambers.

12.1.5 Horizontal Filter Trench

12.1.5.1 Description

Horizontal filter trenches are sloped pea gravel-filled trenches which intercept runoff, pass it through the gravel filter media, and discharge it from the downstream end. Cobbles are used as armoring on top of the gravel-filled trench to prevent higher flows from washing away the pea gravel as well as slowing flows. The primary treatment processes in horizontal filter trenches is filtration, but infiltration can also be substantial depending on the characteristics of native soils.

The cobble armoring may not be sufficient for scour protection if flow velocities become too high. Therefore, stone check dams may be employed to slow the runoff. Check dams for horizontal filter trenches should not be earthen due to the potential for fines to migrate into and clog the filter. Rip-rap check dams function to slow runoff while not damaging the filter.

To ensure the filter trench is draining properly observation wells should be installed along the length of the trench. Observation wells will typically be 1 to 2 in. PVC pipe with perforations at the base. The PVC should be wrapped in filter fabric and capped to prevent clogging or contamination from outside sources.

Figure shows an observation well. Observation wells should be located at a minimum of 50 ft intervals for the length of the filter trench.

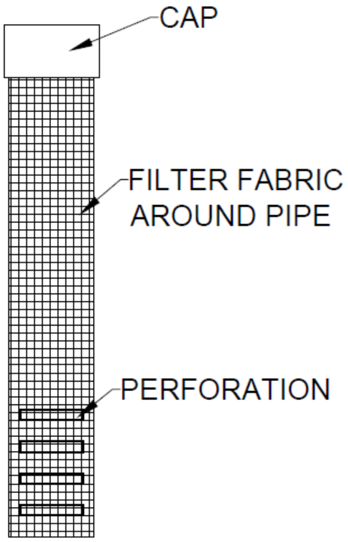

Figure 12.6 Observation well

12.1.5.2 Pollutant Removal Potential

The horizontal filter trench is a BMP which is currently being developed for the Nebraska Department of Roads (NDOR), so there have not been opportunities to study pollutant removal potential. Horizontal filter trenches are expected to show high removal of solids, metals, and particulate phosphorous while nitrogen removal is expected to be low.

12.1.5.3 Cost Considerations

Initial Cost. Construction materials associated with horizontal filter trenches are well known, and accurate cost assessments can be made by contacting local vendors. Costs of materials in filter trenches include filter media, cobble armoring, geotextile, and PVC for the observation wells. Besides material costs, site preparation must be considered in cost assessments. The major costs of site preparation are excavation and stabilizing the contributing area.

The materials, processes, and designs required for construction of horizontal filter trenches are very similar to those required for construction of infiltration trenches, so reasonable cost estimates for the filter trench construction can be drawn from construction costs of infiltration trenches. Observations during a 2004 retrofit study indicated construction costs of nearly $150,000, or $21 per cubic foot treated (CalTrans 2004). The EPA estimated a lower cost of $5 per cubic foot of runoff treated in 2006 (EPA 2006c). Costs will vary with availability of aggregate.

Maintenance Costs. Maintenance costs for horizontal filter trenches will also be similar to those for infiltration trenches. A 2004 retrofit study predicts 27 hrs will be required annually for maintenance with costs of approximately $2,600 for a 4.9 acre contributing area (CalTrans 2004). Trench refurbishing costs are expected to be higher than initial construction costs.

12.1.5.4 Siting Constraints

Horizontal filters are ideally located in long, narrow spaces with moderate slopes. Therefore, roadside applications are well suited for using horizontal filters. Existing roadside ditches are likely prime candidates for retrofit with horizontal filter trenches. Horizontal filter trenches can be incorporated into any swale or ditch system which has pretreatment for removal of particulates. The variability of sizing allows horizontal filter trenches to be incorporated into areas which may not otherwise be utilized (DEPBWM 2006).

For areas with flat topography the horizontal filter will act as an infiltration trench. Infiltration should not be allowed in the following circumstances:

- The seasonal high ground water level is within 3 ft (0.9 m) of the bottom of the system (MSSC 2005).
- Treating a pollutant hot spot (e.g., gas station) where groundwater contamination is

possible.

- Inadequately drained subgrades (hydraulic conductivity $\leq 0.50\ ^{in}/_{hr}$ (1.3 $^{cm}/_{hr}$)).
- Potential interference with foundations/infiltration into basements.
- Infiltration interferes with the subgrade of roadways.

If used where high solids loadings could occur, horizontal filter trenches should be located downstream of a pretreatment system which removes solids. When receiving sheet flow, vegetated filter strips are an ideal pretreatment. If remediating concentrated flow (e.g., end of pipe scenarios) a vegetated swale or rip-rap lined fore-bay can be employed. Pretreatment is important for these systems to prevent clogging with particulates and to avoid the large costs of rehabilitation.

12.1.5.5 Maintenance and Operation Considerations

Maintenance associated with horizontal filter trenches focuses on limiting particulate loading to the trench. As with all BMPs, proper maintenance is required to extend the functional life of horizontal filter trenches and to prevent failure and costly rehabilitation. A summary of typical maintenance activities is provided in

Table .

Table 12.9 Operations and maintenance considerations and suggested corrective procedures for horizontal filter trenches

Inspection Frequency	Problem	Suggested Corrective Procedure
Annually	Filter media clogs with sediment.	Remove and wash or replace clogged media.
	Filter fabric clogs.	Remove sediment from filter fabric. Cobbles may need to be replaced as well.
Semi-Annually	Trees growing near filter trench.	Remove woody vegetation without harmful chemicals and with minimal soil disturbance. Re-vegetate with grass as soon as possible (MSSC 2005).
	Erosion at the inlet or outlet of the trench.	Fill eroded area with cobbles.
	Solids deposit on cobble armoring.	Replace cobbles or wash in a location that does not drain to the trench.
	Check dam gets clogged with debris or sediment	Remove debris and replace rip-rap or wash in a location which does not drain into the trench.
Regularly/As Needed	Contributing area shows rilling or substantial erosion.	Reseed or otherwise stabilize contributing area.
	Weeds or unwanted vegetation begin to dominate the trench.	Weeds should be removed without using environmentally harmful chemicals (CalTrans 2004).
	Sediment build-up unacceptable in pretreatment (dependent on type of pretreatment).	Remove sediment from pretreatment.
	Litter and detritus build up.	Remove litter which is aesthetically unpleasant, negatively affects performance of the trench, or is itself harmful to the environment (FHA 1997b).

12.2 Design Guides

Once the fact sheets are reviewed and the ideal BMP for a site is selected, the BMP design guide is consulted to ensure proper use of the BMP. The design guides typically include:

- Design process: Provides the procedure for designing the BMP.
- Design criteria: Identifies BMP-specific design parameters.
- Design example: Provides an example site and performs the design process.

Design guides for the vegetated filter strip and vegetated swale will be based on the peak flow of the water to be treated. The bioretention cell and basin filter design will be based on the volume of water to be treated (WQV), and design of the horizontal filter trench will be based on the peak flow as well as the WQV.

12.2.1 Vegetated Filter Strip

12.2.1.1 Design Process

Step 1: Evaluate applicability of vegetated filter strip considering site constraints.

Step 2: Calculate Peak Water Quality Flow (Q_{WQV}).

Step 3: Calculate Water Quality Flow Depth (D_{WQV}).

Step 4: Calculate Water Quality Flow Velocity (V_{WQV}).

Step 5: Check Scour Velocity for 10-yr Storm (V_S).

Step 6: Determine Pretreatment Method.

Step 7: Specify Vegetation Plan.

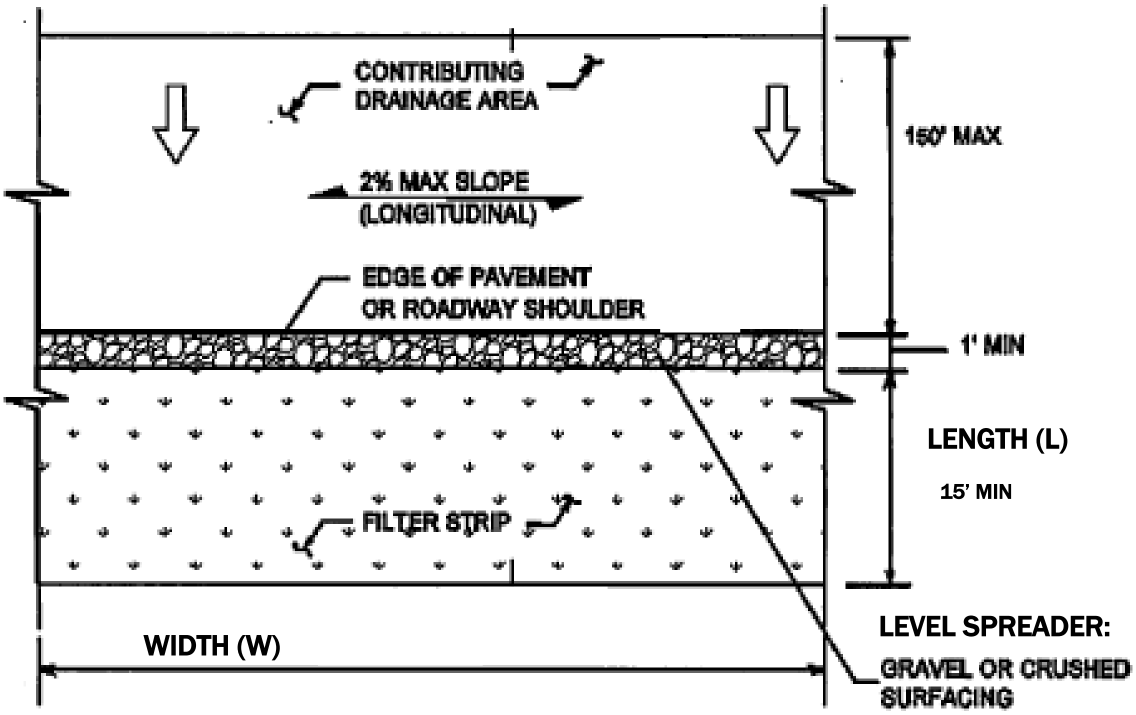

Figure 12.7 Plan view of vegetated filter strip (adapted from WSDOT 2010)

12.2.1.2 Design Criteria

Table contains the criteria to be considered while working through the design process.

Table 12.10 Design criteria for vegetated filter strip

Design Parameter	Minimum	Maximum
Vegetated strip slope parallel to flow	$2\%^{6,7,8}$	$15\%^{2,4,7,8}$
Strip length (parallel to flow)	15 ft (4.6 m)^{1,5,8}	Pollutant removal plateaus at 65 ft (20 m)^{2,8}
Ground cover	$80\%^{1}$	-
Flow through strip	-	Must not cause erosion during events larger than the Water Quality Flow.
Side slope (perpendicular to flow)	-	$2\%^{4,7}$
Velocity through strip	-	$1\ \frac{\text{ft}}{\text{s}}\ (0.3\ \frac{\text{m}}{\text{s}})^{1,3}$
Depth through strip	-	1 in. (0.39 cm)^{1,4,7}
Runoff flow path before entering BMP	-	75 ft (23 m)^{2,3} over impermeable surface or 150 ft (46 m)^{2,3,4,7} over permeable surface.

1) Caltrans (2010a)	5) Li et al. (2008)
2) Clar et al. (2004)	6) MSSC (2005)
3) FHA (2002a)	7) WSDOT (2010)
4) KCDNRP (2009)	8) Zhang et al. (2010)

Step 1: Evaluate Applicability of Vegetated Filter Strip Considering Site Constraints. Vegetated filter strips can be applied adjacent to roadways, parking areas, or as an end-of-pipe (i.e., storm sewer outlet) BMP. They are best suited in locations where they can receive sheet flow from relatively horizontal surfaces such as parking lots or level roadways. When adjacent to roadways the cross slope (parallel to the roadway) is often the controlling factor in hilly areas. The cross-slope must be smaller than 2% in order for runoff to flow parallel to the design length

of the strip. Locating a vegetated strip adjacent to a roadway in an urban setting may require too much area. If there is not enough space for the strip next to the road, it may be possible to install a vegetated filter strip as an end-of-pipe BMP, or another BMP more suited to an ultra-urban environment may be selected. For end-of-pipe applications vegetated filter strips generally must incorporate level spreaders and may require pretreatment such as sediment basins or velocity reduction systems. Design considerations for these facilities can be found in step 6.

Outlet works for vegetated filter strips include unmanaged discharge directly into receiving waters or swale systems. Direct discharge may require slope stabilization, such as rip-rap if the slope to the waterway is susceptible to erosion. When direct discharge is not an option, a swale system may be constructed at the base of the strip to transport the runoff to receiving waters or another intermediate conveyance system such as a pipe. Adequately designed swales can also provide additional treatment. Design for swale systems can be found in the Vegetated Swale Design Guide section of this work.

Step 2: Calculate Peak Water Quality Flow (Q_{WQV}*).* Peak flows have been calculated and displayed in

Table for impervious surfaces, such as pavement, up to 5 acres. For pervious areas or areas larger than 5 acres, peak flow rates are determined by using the 0.75 in. design storm with a type II NRCS 24 hr distribution and equation 12.1(NRCS 1986). A detailed description of the use of this equation is given in section 11.3.

$$\mathrm{q_p = q_u A_m Q F_p} \tag{12.1}$$

Where:

q_p: Peak discharge (cfs)

q_u: Unit peak discharge $\left(\frac{cfs}{mi^2*in}\right)$ (Figure or Table)

A_m: Drainage area (mi^2)

Q: Runoff corresponding to 24-hr rainfall (in.) (Table)

F_p: Pond or swamp adjustment factor (1.0 for Nebraska)

When considering a watershed with both impervious and pervious ground cover, the area can either be considered completely impervious, or a weighted flow may be calculated. Assuming total imperviousness would result in larger than actual flows and, therefore, oversized BMPs. Therefore, the weighted flow method is recommended, as described in the Hydrology Section of this work.

Step 3: Calculate Water Quality Flow Depth (D_{WQV}). The design flow depth can be calculated using the peak flow rate (Q_{WQV}) found in step 2 and equation 12.2, which is derived from the Manning equation (WSDOT 2010, Cal Trans 2010):

$$\mathrm{d} = \left(\frac{\mathrm{Q_{wqf}n}}{\mathrm{kWS}^{1/2}}\right)^{3/5} \tag{12.2}$$

Where:

Q_{wqv}: Water Quality Flow (cfs or cms)

S: Slope parallel to flow $\left(\frac{\mathrm{ft}}{\mathrm{ft}} \text{ or } \frac{\mathrm{m}}{\mathrm{m}}\right)$

n: Manning’s coefficient (0.24 for well-established dense grass (CalTrans 2010a)) k: constant (1 for Metric Units 1.486 for English Units)

W: Width of strip perpendicular to flow (ft or m)

d: Depth (ft or m)

Assuming that the width of the sheet flow is significantly larger than the depth, equation 12.2 can be rearranged into equation 12.3:

$$Q_{wqv} = \frac{k}{n} W d^{5/3} S^{1/2} \qquad (12.3)$$

If the depth is greater than 1 in. (0.39 cm), measures need to be taken to reduce flow or to expand width; otherwise, vegetated filter strips should not be used (Caltrans 2010a, KCDNRP 2009, WSDOT 2010). Depths greater than 1 in. (0.39 cm) will not be effective in treatment and will pose a higher risk of scour.

For new construction or end-of-pipe considerations, solving for the minimum width may be beneficial. A maximum depth of 1 in. (0.39 cm) will be used to determine the minimum width of the filter strip. Solving for W, equation 12.3 is reorganized into equation 12.4:

$$W = \frac{Q_{wqv} n}{k S^{1/2} d^{5/3}} \qquad (12.4)$$

For existing grass filter strips adjacent to roadways, the width generally coincides with the length of the roadway. This existing infrastructure should be checked against equation 12.4 to determine if it will act as a properly designed vegetated filter strip.

Step 4: Calculate Water Quality Flow Velocity, (V_{WQV}). The flow rate and flow depth can be used to calculate the runoff velocity through the BMP with equation 12.5.

$$\mathrm{V_{wqv}} = \frac{\mathrm{Q_{wqv}}}{\mathrm{Wd}} \qquad (12.5)$$

The velocity of the water quality volume (V_{wqv}) must be less than 1 $\frac{ft}{s}$ (0.3$\frac{m}{s}$) over the entire length of the filter strip (FHWA, Caltrans 2010a). Excess velocities will result in scour and short circuiting of the system. Short circuiting will adversely affect pollutant removal by not allowing the runoff to interact with an adequate amount of vegetation.

Step 5: Check Scour Velocity for 10-yr Storm (V_S). Vegetated filter strips are often flow-through BMPs. This means that they will be required to facilitate flows greater than the water quality design flow. Vegetated filter strips must be able to accommodate these flows without being damaged.

Scour velocity will be calculated with the same process used for the water quality flow analysis (i.e., equations 12.1 to 12.5); however, scour velocity was calculated for the 10-yr, 24-hr storm, which was 5 in. according to TP 40 (Hershfield 1961). The resulting velocity will then be compared to the values in

Table , which show the scour velocities for common soil classes and their retardance classes.

Retardance classes are defined in

Table .

Table 12.11 Scour velocities in channels with various soil types and ground covers (USDA 1979)

Soil Texture	Bare Channel Scour Velocity (ft/s)	Vegetated Channel Scour Velocity (ft/s)			
		Retardance Class	Vegetation Condition		
			Poor	Fair	Good
Sand, silt, sandy loam, silty loam	1.5	B	1.5	3	4
		C	1.5	2.5	3.5
		D	1.5	2	3
Silty clay loam, sandy clay loam	2	B	2.5	4	5
		C	2.5	3.5	4.5
		D	2.5	3	4
Clay	2.5	B	3	5	6
		C	3	4.5	5.5
		D	3	4	2

Table 12.12 Ground cover retardance classes (Kilgore & Cotton 2005)

Retardance Class	Ground Cover	Condition
B	Kudzu	Very dense growth, uncut
	Bermuda Grass	Good stand, tall, average 300 mm (12 in)
	Native Grass Mixture (little bluestem, bluestem, blue gamma, and other long and short midwest grasses)	Good stand, unmowed
	Weeping lovegrass	Good stand, tall, average 610 mm (24 in)
	Lespedeza sericea	Good stand, not woody, tall, average 480 mm (19 in)
	Alfalfa	Good stand, uncut, average 280 mm (11 in)
	Weeping lovegrass	Good stand, unmowed, average 330 mm (13 in)
	Kudzu	Dense growth, uncut
	Blue Gamma	Good stand, uncut, average 280 mm (11 in)
C	Crabgrass	Fair stand, uncut 250 to 1200 mm (10 to 48 in)
	Bermuda grass	Good stand, mowed, average 150 mm (6 in)

	Common Lespedeza	Good stand, uncut, average 280 mm (11 in)
	Grass-Legume mixture--summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut, 150 to 200 mm (6 to 8 in)
	Centipede grass	Very dense cover, average 150 mm (6 in)
	Kentucky Bluegrass	Good stand, headed, 150 to 300 mm (6 to 12 in)
D	Bermuda Grass	Good stand, cut to 60 mm (2.5 in) height
	Common Lespedeza	Excellent stand, uncut, average 110 mm (4.5 in)
	Buffalo Grass	Good stand, uncut, 80 to 150 mm (3 to 6 in)
	Grass-Legume mixture-fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut, 100 to 130 mm (4 to 5 in)
	Lespedeza sericea	After cutting to 50 mm (2 in) height. Very good stand before cutting.

Step 6: Determine Pretreatment Methods. Vegetated filter strips may require pretreatment to slow runoff, remove coarse sediment, and evenly distribute flow over the width of the BMP. Level spreaders can be used to adequately address these three concerns. Runoff must be slowed and evenly distributed if it is entering the system as concentrated flow, or if it has traveled greater than 75 ft over impervious ground cover or greater than 150 ft over impervious ground cover (Clar et al. 2004).

When located adjacent to an impervious surface, a simple gravel trench, such as shown in Figure , is adequate as a level spreader. These trenches should be 1 ft (0.3 m) wide and 2 to 3 ft (0.61 to 0.91 m) deep. The fill gravel should consist of clean washed, uniformly graded coarse aggregate to the AASHTO # 3 specification (SEMCOG 2008). There should also be a 1 to 2 in. (2.5 to 5.1 cm) drop from the impervious surface to the trench (SEMCOG 2008).

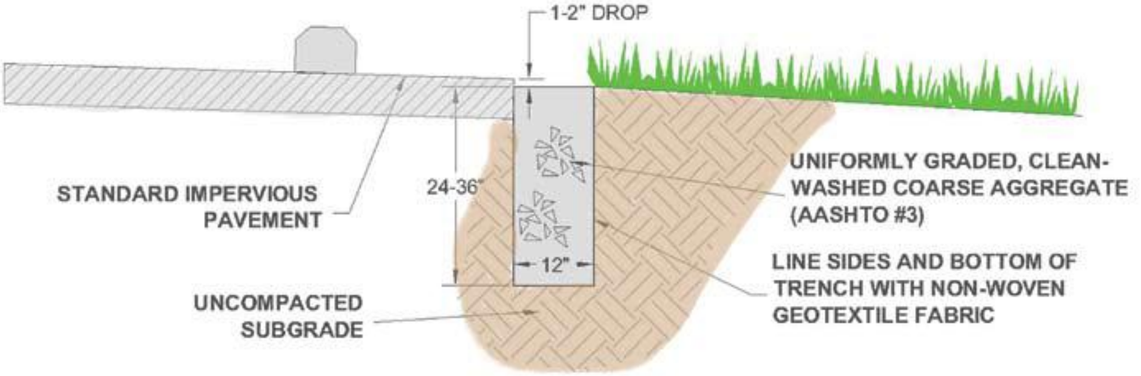

Figure 12.8 Level spreader adjacent to roadway or parking lot (SEMCOG 2008)

Level spreaders are made up of a trench with one edge which is lower and level allowing water to exit evenly along its length. This trench can be open or filled with gravel. If the trench is open, it is acceptable to line it with vegetation or concrete. Pipes discharging into the level spreader should be oriented parallel to the trench. Discharging into the trench lengthwise will minimize overloading and over-flow in a localized section of the level spreader. Figure shows proper entrance to a flow spreader, and Figure shows an improper entry angle.

Figure 12.9 Proper entry to level spreader (Winston et al. 2010)

Figure 12.10 Improper entry to level spreader (Winston et al. 2010)

The downstream (level) edge of the level spreader may be reinforced with treated wood, gravel, or concrete. Regardless of reinforcement the downstream edge must be level and straight to uniformly distribute the runoff. It must also be more than 1 in. (2.54 cm) lower than the uphill edge. If flow enters a level spreader as sheet flow the trench may be filled with evenly graded coarse aggregate. The gravel adds filtration, as well as controlling mosquito breeding. Gravel may not be ideal for trenches which accept concentrated flow because the gravel would inhibit uniform filling of the trench, causing uneven discharge along the length of the level spreader.

The storage volume in the level spreader must be large enough to adequately handle and distribute the peak runoff flows. Level spreaders designed for handling concentrated flow should not have depths exceeding 1 ft (0.3m), and they should be as wide as the vegetated filter strip it discharges into. Level spreaders should be wide enough to discharge the WQV flow, which was found with equation 12.1, without exceeding a flow depth of 1 in. (2.54 cm). Equation 12.4 can be used to find the minimum width of the level spreader. Gravel-filled level spreaders, which are ideal for handling sheet flow, may be 2 to 3 ft (0.6 to 0.9 m) deep (SEMCOG 2008).

Overflow bypass should be provided for large flows. The manner of bypass structure will be largely dependent on the BMP's surroundings. Bypass solutions may include a spillway at the end of the trench which discharges into a swale or under-drains discharging into a sewer system. Drainage measures must be implemented in open-channel level spreaders to allow draw-down within 24 hrs to control mosquitoes. Vegetated trenches may need an under-drain if local soils do not allow for the infiltration of the design storm within the required 24 hrs. The under-drains should discharge into the same structure as the overflow.

Step 7: Specify vegetation plan. The vegetation in vegetated filter strips should be able to survive periods of saturation and periods of drought. Plants must also be able to withstand salts associated with deicing processes necessary in Nebraska's seasonal climate. Vegetation should be limited to grasses, or other vegetation which provides low ground cover. Nebraska's regional climate and soil compositions make it impractical to identify a single seed mix for the entire state. The Nebraska Department of Roads (NDOR) has established 6 landscape regions and has determined applicable grass mixtures for each. These suggested mixes are presented in Appendix E.

12.2.1.3 Design Example

A 2-lane highway was being constructed which will add 0.5 acres of impervious area. There was an existing 30 ft adjacent grass strip at an 8% slope away from the roadway. The longitudinal slope of the highway, and subsequent cross slope of the vegetated filter strip, was 1%. Figure 1 shows the plan view for this design example.

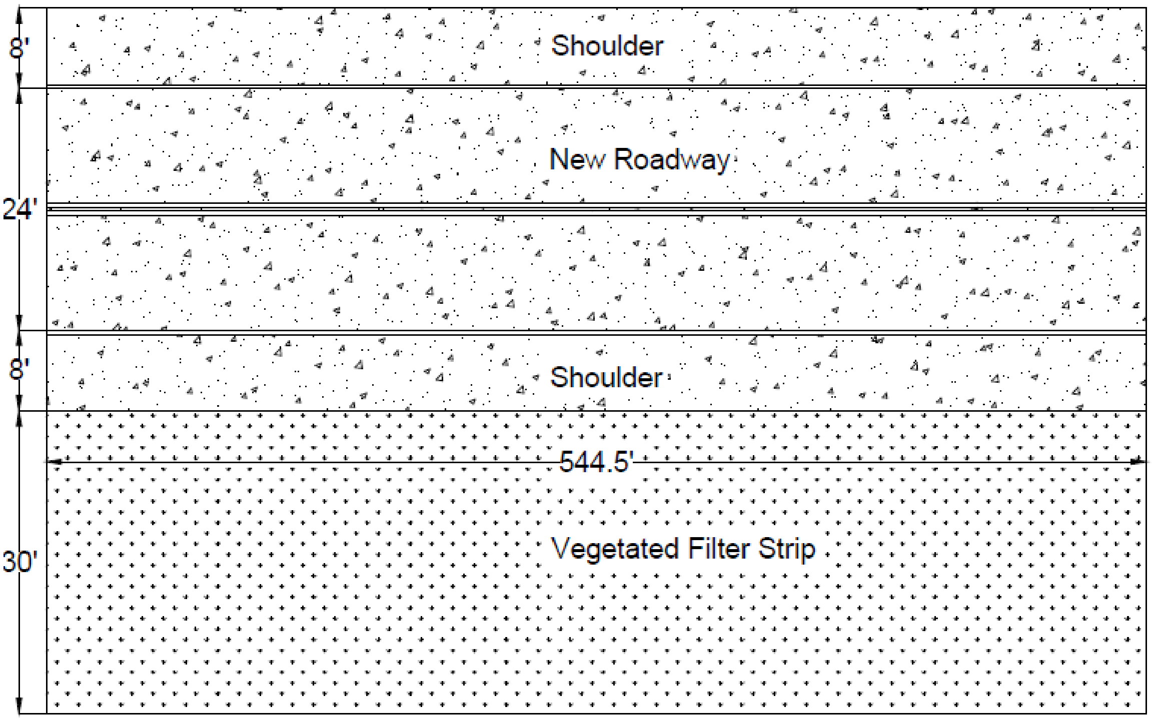

Figure 1 Plan view for vegetated filter strip design example 1

Step 1: Evaluate Applicability of Vegetated Filter Strip Considering Site Constraints.

The lack of other structures in the right-of-way and acceptable slopes make this an ideal site to employ a vegetated filter strip adjacent to the roadway.

Step 2: Calculate Peak Water Quality Flow (Q_{WQV}).

Table shows that the design peak water quality flow is 0.474 cfs for 0.5 impervious acres.

Step 3: Calculate Water Quality Flow Depth (D_{WQV}). Equation 12.2 was used to determine the flow depth:

$$d = \left(\frac{Q_{wqv} n}{k W S^{1/2}}\right)^{3/5} = \left(\frac{0.474 \frac{\mathrm{ft}^3}{\mathrm{s}} * 0.24}{1.486 * 544.5\, ft * \sqrt{0.08}}\right)^{3/5} = 0.125\, in$$

Using Q_{wqv}= 0.474 $\frac{\mathrm{ft}^3}{\mathrm{s}}$, n = 0.24, k = 1.486, S = 0.08, and W = 544.5 ft, d = 0.01 ft.

The width of the vegetated filter is equivalent to the length of the roadway. Two 12 ft wide lanes with an 8 ft shoulder were assumed for this example. The calculated depth of 0.125 in. is less than the maximum of 1 in., so the width is satisfactory.

Step 4: Calculate Water Quality Flow Velocity (V_{WQV}). Equation 12.5 was used to determine the flow velocity.

$$V_{wqv} = \frac{Q_{wqv}}{Wd} = \frac{0.474\ \frac{\mathrm{ft}^3}{\mathrm{s}}}{544.5\ \mathrm{ft} * 0.125\ \mathrm{in}} = 0.084\ \frac{\mathrm{ft}}{\mathrm{s}}$$

Using Q_{wqv}= 0.474 $\frac{\text{ft}^3}{\text{s}}$, W = 544.5ft, and d = 0.125 in.: $V_{wqv} = 0.084\ \frac{\text{ft}}{\text{s}}$, the calculated velocity is less than the 1 $\frac{\text{ft}}{\text{s}}$ maximum, so it is acceptable.

Step 5: Check Scour Velocity (V_S). The 10-yr, 24-hr storm was used to check scour velocities. Peak flows were found using equation 12.1:

$$q_p = q_u A_m Q F_p$$

Table shows a curve number of 98 for impervious areas. The curve number was then used with the ratio of initial abstraction (I_a) to precipitation (P) to find the unit peak discharge (q_u). Figure or Table can both be consulted for the q_u value. A curve number of 98 produces a q_u of 1100 $\frac{cfs}{mi^2 * in}$. Table shows a runoff depth (Q) of 4.76 in for the 10-yr storm. The swamp adjustment factor (F_p) for the state of Nebraska is 1. Using equation 12.1 gave:

$$q_p = 1100\ \frac{cfs}{mi^2 * in} * 0.00078\ mi^2 * 4.76\ \text{in} * 1 = 4.08\ \text{cfs}$$

Equation 12.2 was then used to find flow depth:

$$d = \left(\frac{Q_{10-yr} n}{kWS^{1/2}}\right)^{3/5} = \left(\frac{4.08\frac{\text{ft}^3}{\text{s}} * 0.24}{1.486 * 544.5\ ft * \sqrt{0.08}}\right)^{3/5} = 0.455\ in$$

Equation 12.5 was then used to find the flow velocity:

$$V_{10-yr} = \frac{Q_{10-yr}}{Wd} = \frac{4.08\ \frac{\text{ft}^3}{\text{s}}}{544.5\ \text{ft} * 0.455\ \text{in}} = 0.2\ \frac{\text{ft}}{\text{s}}$$

The calculated value is less than any value on

Table and therefore passes for any ground condition. For example, a fair stand of Kentucky Bluegrass, which has a retardance class of C according to

Table , in a silty loam soil would be adequate as it resists velocities of 2.5 $\frac{\mathrm{ft}}{\mathrm{s}}$.

Step 6: Determine Pretreatment Methods. Because the runoff did not travel 75 feet or more over an impervious surface before entering the filter strip it will enter as sheet flow, which does not require pretreatment. Had the runoff traveled over 75 ft, a 1-ft wide, 2-ft deep gravel filled level spreader would be a sufficient pretreatment.

Step 7: Specify Vegetation Plan. A grass mixture should be selected which can survive the climatic and roadway conditions (e.g., salt) expected at the site. Suggested mixtures are described in Appendix E.

12.2.1.4 Design Example 2

A 2-lane highway was being constructed which will add 0.5 acres of impervious area. There was little adjacent land area available, and acquiring it would have been prohibitively expensive. However, there was ample room at the outfall, so an end-of-pipe vegetated filter strip will be employed. A slope of 8% will be used for the vegetated filter strip.

Figure **2** shows the plan view for design example 2.

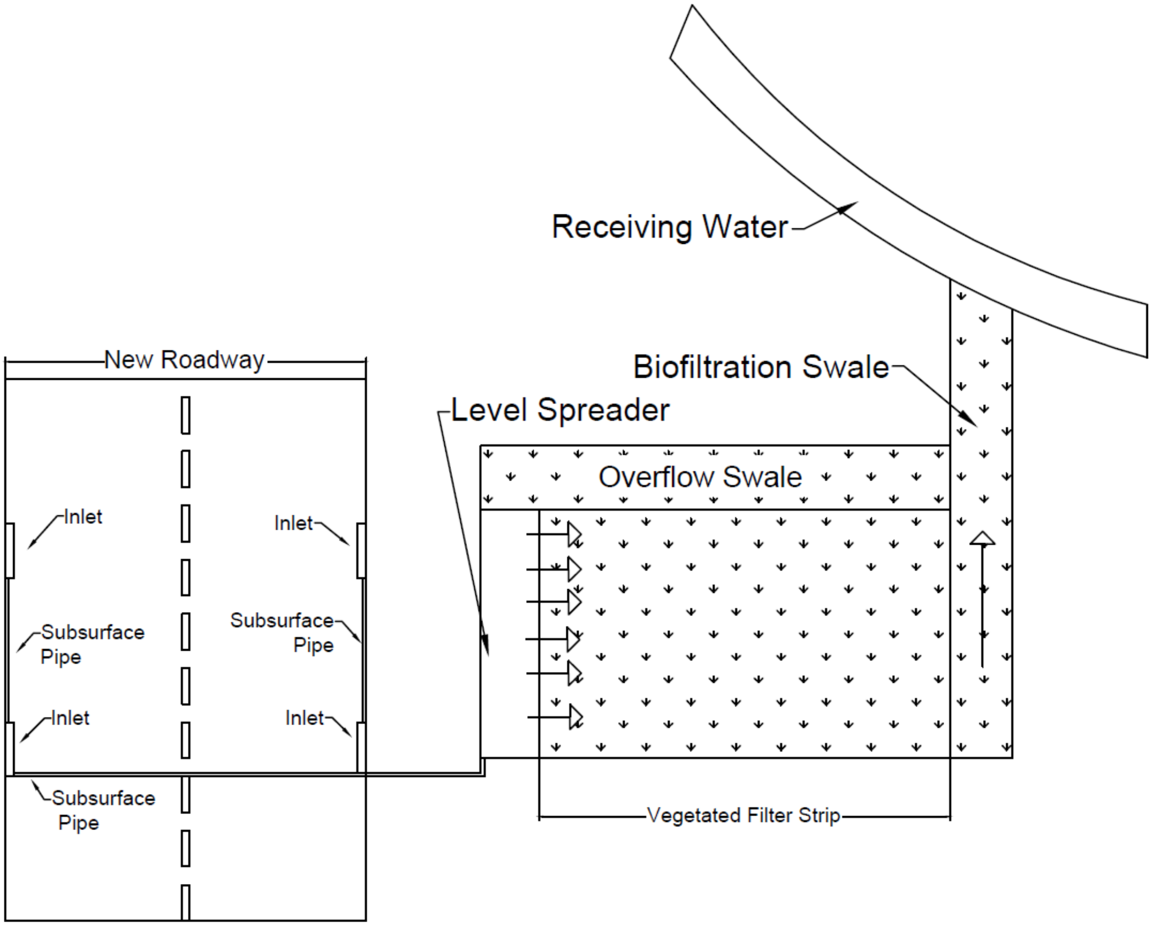

Figure 2 Plan view for vegetated filter strip design example 2

Step 1: Evaluate applicability of vegetated filter strip considering site constraints. The lack of available space next to the road requires the vegetated filter strip to be used off-site as an end-of-pipe BMP.

Step 2: Calculate Peak Water Quality Flow (Q_{WQV}).

Table gives a design peak flow of 0.474 cfs for 0.5 impervious acres.

Step 3: Calculate Water Quality Flow Depth (D_{WQV}). For end-of-pipe applications, the filter width must be calculated. The minimum width is found using equation 12.4 and assuming the flow depth to be the maximum 1 in.

$$W = \frac{Q_{wqv}n}{KS^{1/2}d^{5/3}} = \frac{0.474\ \frac{\mathrm{ft}^3}{\mathrm{s}} * 0.24}{1.486 * 0.08^{1/2} * 1\ inch^{5/3}} = 17ft$$

Using $Q_{wqf} = 0.474\ \frac{\mathrm{ft}^3}{\mathrm{s}}$, n = 0.24, k = 1.486, S = 0.08, and d = 1 in, $W = 17\ ft$.

Step 4: Calculate Water Quality Flow Velocity (V_{WQV}). Equation 12.5 was used to determine the flow velocity.

$$V_{wqv} = \frac{Q_{wqv}}{Wd} = \frac{0.474\ \frac{\mathrm{ft}^3}{\mathrm{s}}}{17\ ft * 1\ \ inch} = 0.334\ \frac{\mathrm{ft}}{\mathrm{s}}$$

Using $Q_{wqv} = 0.474\ \frac{\mathrm{ft}^3}{\mathrm{s}}$, W = 17 ft, and d = 1 in, $V_{wqv} = 0.334\ \frac{\mathrm{ft}}{\mathrm{s}}$, which is < 1 $\frac{\mathrm{ft}}{\mathrm{s}}$ so it is acceptable.

Step 5: Check Scour Velocity (V_S). Using a vegetated filter strip in this configuration will not require a scour check, because it is not set up as a flow through BMP. An overflow weir is located 1 in. above the lip of the level spreader to allow the WQV to discharge at its maximum allowable depth while allowing excess flows to bypass. The level spreader configuration is shown in Figure 3.

Step 6: Determine Pretreatment Methods. The runoff is being transported as concentrated flow, so a level spreader must be employed to slow and evenly distribute the design flow. The level spreader will be a trapezoidal trench with 3:1 side slopes, 1 ft of depth, and a bottom width of 2 ft. The downstream (level) edge of the trench will be reinforced by treated lumber and gravel armoring. Overflow bypass will be provided by a rectangular weir at the end of the trench, which is 1 in. strip. The overflow weir will discharge into a swale running parallel with the filter strip and discharge into the same receiving water. Figure 3 shows the level spreader set-up.

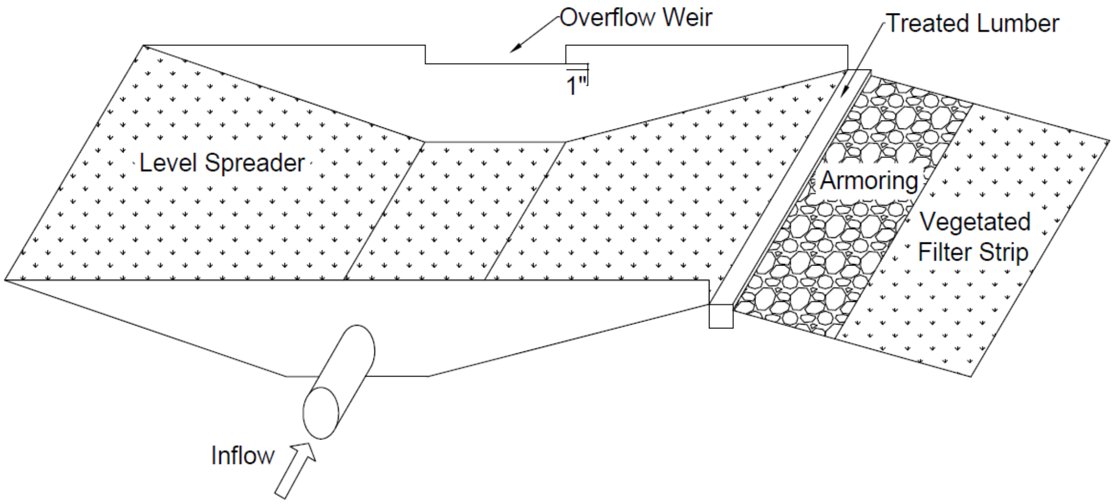

Figure 3 Level spreader for vegetated filter strip design example 2

Step 7: Specify Vegetation Plan. A grass mixture should be selected which can survive the climatic and roadway conditions (e.g., salt) expected at the site. Suggested mixtures are described in Appendix E.

12.2.2 Vegetated Swale

12.2.2.1 Design Process

Step 1: Evaluate Applicability of Vegetated Swale Considering Site Constraints.

Step 2: Calculate Peak Water Quality Flow (Q_{WQV}).

Step 3: Dimension the Swale.

Step 4: Calculate the Water Quality Flow Depth (D_{WQV}).

Step 5: Calculate Water Quality Flow Velocity (V_{WQV}).

Step 6: Check Scour Velocity for 10-yr Storm (V_S).

Step 7: Design and Position Check Dams (if necessary).

Step 8: Specify Vegetation Plan.

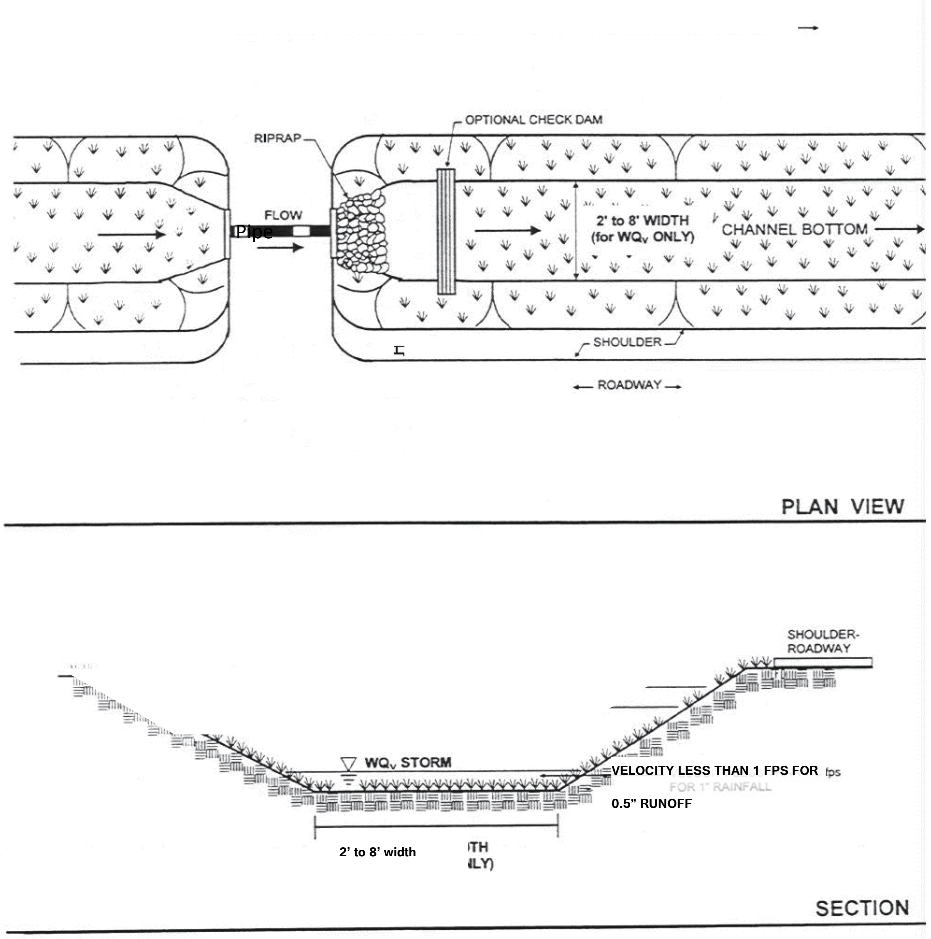

Figure 4 Plan and profile view of vegetated swale (adapted from Clar et al. 2004)

12.2.2.2 Design Criteria

Table **4** contains the criteria to be considered while working through the design process.

Table 4 Design criteria for vegetated swale

Design Parameter	Minimum	Maximum
Bottom Channel Slope of Swale	1%[4,5,6]	5%[1,3,5,7]
WQV Flow Depth Across Swale	-	4 in[4,5]
WQV Velocity Parallel to Swale	-	$1\ \frac{\text{ft}}{\text{s}}$[1,2,4,7]
Bottom Channel Width of Swale	2 ft[1,2,4,5,6,7]	8 ft[1,5,6]
Channel Side Slope	-	3:1[2,5,6,7]

1) CalTrans (2010b) 2) CalTrans (2004) 3) Clar et al. (2004) 4) KCDNRP (2009)	5) MSSC (2005) 6) SEMCOG (2008) 7) WSDOT (2010)

Step 1: Evaluate Applicability of Vegetated Swale Considering Site Constraints. Vegetated swales may not provide enough treatment to be considered a stand-alone BMP (EPA 2006b). However, when site conditions are satisfactory, vegetated swales are a significant and viable BMP. They are particularly useful where soils are relatively permeable (NRCS hydrologic soil groups A through C); soils should have infiltration rates of $0.18\ \frac{\text{in}}{\text{hr}}$ ($4.5\ \frac{\text{mm}}{\text{hr}}$) or higher (Landphair et al. 2000). Vegetated swales are often effectively located up or down stream of other BMPs. When upstream they provide pretreatment by filtering out debris and other solids. When employed downstream they provide additional treatment while transporting the treated runoff from the primary BMP to a discharge point. In addition to the treatment benefits, vegetated conveyance systems are more aesthetically pleasing than concrete-lined channels.

The linear nature of vegetated swales makes them excellent treatment and conveyance systems for runoff from roadways. Roadway drainage systems may already be functioning swale systems, or they may be easily retrofit for pollutant removal (CalTrans 2003).

Vegetated swales may not be suited for ultra-urban areas due to the necessity for relatively large areas. For densely developed areas, pipes are likely a more efficient and cost effective conveyance system, as they do not require as much area.

Step 2: Calculate Peak Water Quality Flow (Q_{WQV}*).* Peak flows have been calculated and displayed in

Table for impervious surfaces, such as pavement, up to 5 acres. For pervious areas or areas larger than 5 acres, peak flow rates were determined by using the 0.75 in. design storm with a type II NRCS 24 hr distribution and equation 12.6 (NRCS 1986).

$$q_p = q_u A_m Q F_p \tag{12.6}$$

Where:

q_p: Peak discharge (cfs)

q_u: Unit peak discharge $\left(\frac{cfs}{mi^2 * in}\right)$ (Figure or Table)

A_m: Drainage area (mi^2)

Q: Runoff corresponding to 24-hr rainfall (in) (Table for WQV)

F_p: Pond or swamp adjustment factor (1.0 for Nebraska)

When considering a watershed with both impervious and pervious ground cover, the area can either be considered completely impervious, or a weighted flow may be calculated. Assuming total imperviousness would result in larger than actual flows and, therefore, oversized

BMPs. Therefore, the weighted flow method is recommended, as described in the Hydrology Section of this work.

Step 3: Dimension the Swale. Swale dimensions include the channel's bottom width, side slopes, and longitudinal slope. The design guidelines and limitations for these parameters are presented in

Table **4**. Swale dimensions will largely rely on site-specific considerations and existing drainage strategies.

Step 4: Calculate the Water Quality Flow Depth (D_{WQV}*).* Once the shape of the swale is decided upon, equation 12.7 (Manning's Equation) can be applied to determine flow depth (NRCS 1986).

$$Q_{wqv} = \frac{k}{n} A R^{2/3} S^{1/2} \qquad (12.7)$$

Where:

Q_{wqv}: Peak Water Quality Flow (cfs or cms)

S: Slope in direction of flow $\left(\frac{\text{ft}}{\text{ft}} \text{ or } \frac{\text{m}}{\text{m}}\right)$

R: Hydraulic Radius $\left(R = \frac{A}{P_w}\right)$

A: Cross sectional area of flow (ft^2 or m^2)

P_w: Wetted Perimeter (ft or m)

n: Manning's coefficient (0.24 for well-established dense grass (Caltrans 2010))

k: constant (1 for Metric Units; 1.486 for English Units)

The necessary equations for the elements of trapezoidal cross-sections can be found in

Table **5**.

T

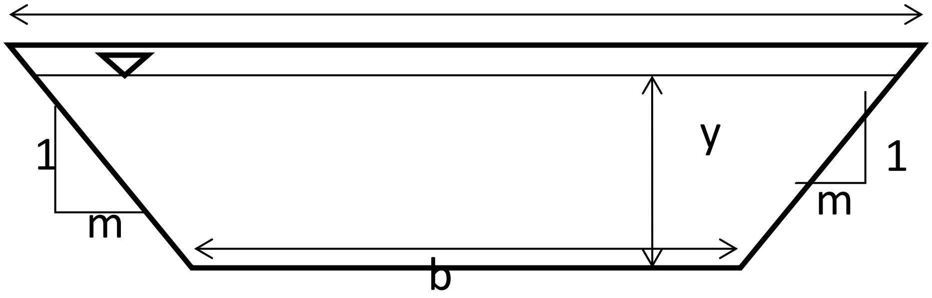

Figure 5 Reference shape for table 12.14

Table 5 Geometric elements of trapezoidal cross section (Adapted from WSDOT 2010)

Area of flow (A) (ft^2 or m^2)	$(b+my)y$
Wetted perimeter (P_w) (ft or m)	$b+2y\sqrt{1+m^2}$
Hydraulic radius (R) (ft or m)	$\frac{(b+my)y}{b+2y\sqrt{1+m^2}}$

Inserting these geometric elements into equation 12.7 results in equation 12.8:

$$Q_{wqv} = \left(\frac{k}{n}\right) * (b+my)y * \left[\frac{(b+my)y}{b+2y\sqrt{1+m^2}}\right]^{2/3} * S^{1/2} \qquad (12.8)$$

Equation 12.8 with the peak water quality flow found in step 2 and the dimensions decided upon in step 3 can be used to verify whether the depth of the flow will be less than 4 in. (7.6 cm) (table 12.13). If the depth is > 4 in. the swale will need to be redimensioned, or check dams can be employed.

Step 5: Calculate Water Quality Flow Velocity (V_{WQV}*).* The velocity of the flow through the BMP can be determined with equation 12.9 through the flow rate and the cross-sectional area of flow. The cross-sectional area can be found using

Table **5**.

$$V = \frac{Q_{wqv}}{A} \qquad (12.9)$$

The velocity for the water quality flow parallel to swale should not exceed 1.0 $\frac{ft}{s}$ (table 12.13). Higher flows will result in less treatment of the runoff.

Step 6: Check Scour Velocity for 10-yr Storm (V_S*).* Vegetated swales are often flow-through BMPs. This means they will be required to handle flows greater than the water quality flow. Vegetated swales must be able to accommodate these flows without being damaged.

Scour velocity was found using the same methodology as the WQV velocity (steps 2 through 5). However, scour velocity analysis was performed based on the 10-yr, 24-hr storm. For the state of Nebraska, the maximum rainfall depth for the 10-yr, 24-hr storm was 5 in. according to TP 40 (Hershfield 1961). The resulting velocity (calculated using steps 2 through 5) was then compared to the values in Table 6, which shows the appropriate scour velocities for common soil classes and their retardance classes. Retardance classes are defined in Table 7.

Table 6 Scour velocities in channels with various soil types and ground covers (USDA 1979)

Soil Texture	Bare Channel Scour Velocity (ft/s)	Vegetated Channel Scour Velocity (ft/s)			
		Retardance Class	Vegetation Condition		
			Poor	Fair	Good
Sand, silt, sandy loam, silty loam	1.5	B	1.5	3	4
		C	1.5	2.5	3.5
		D	1.5	2	3
Silty clay loam, sandy clay loam	2	B	2.5	4	5
		C	2.5	3.5	4.5
		D	2.5	3	4
Clay	2.5	B	3	5	6
		C	3	4.5	5.5
		D	3	4	2

Table 7 Ground cover retardance classes (Kilgore & Cotton 2005)

Retardance Class	Ground Cover	Condition
B	Kudzu	Very dense growth, uncut
	Bermuda Grass	Good stand, tall, average 300 mm (12 in.)
	Native Grass Mixture (little bluestem, bluestem, blue gamma, and other long and short midwest grasses)	Good stand, unmowed
	Weeping lovegrass	Good stand, tall, average 610 mm (24 in.)
	Lespedeza sericea	Good stand, not woody, tall, average 480 mm (19 in.)
	Alfalfa	Good stand, uncut, average 280 mm (11 in.)
	Weeping lovegrass	Good stand, unmowed, average 330 mm (13 in.)
	Kudzu	Dense growth, uncut
	Blue Gamma	Good stand, uncut, average 280 mm (11 in.)
C	Crabgrass	Fair stand, uncut 250 to 1200 mm (10 to 48 in.)
	Bermuda grass	Good stand, mowed, average 150 mm (6 in.)

	Common Lespedeza	Good stand, uncut, average 280 mm (11 in.)
	Grass-Legume mixture-summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut, 150 to 200 mm (6 to 8 in.)
	Centipede grass	Very dense cover, average 150 mm (6 in.)
	Kentucky Bluegrass	Good stand, headed, 150 to 300 mm (6 to 12 in.)
D	Bermuda Grass	Good stand, cut to 60 mm (2.5 in.) height
	Common Lespedeza	Excellent stand, uncut, average 110 mm (4.5 in.)
	Buffalo Grass	Good stand, uncut, 80 to 150 mm (3 to 6 in.)
	Grass-Legume mixture-fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut, 100 to 130 mm (4 to 5 in.)
	Lespedeza sericea	After cutting to 50 mm (2 in.) height. Very good stand before cutting.

Step 7: Design and Position Check Dams (if necessary). Check dams may be necessary to keep the WQV velocity below 1 $\frac{\text{ft}}{\text{s}}$. Check dams are installed perpendicular to the flow. Although certain check dams provide some treatment through sedimentation or filtration, those effects are secondary to velocity dissipation and are not the focus of check dam design.

Roadside check dams should be easily maintained while not interfering with maintenance of the swale itself. Swale mowing operations, in particular, should not be adversely affected by the check dams. This is done by maintaining small slopes (5:1 to 10:1 (Clar et al. 2004)) on the up and downstream sides of the check dams, respectively. The low slopes also prevent check dams from being a hazard to motorists who could potentially crash into or ramp off them.

A roadside check dam can be constructed by installing rip-rap, railroad ties, wood chips, or a vegetated berm across the width of a swale. Regardless of the material, the check dam height should not exceed 2 ft (0.61 m) (Landphair et al. 2000 ; Clar et al. 2004). A 1 ft (0.3 m) wide gravel trench may be required to protect the downstream edge of the check dam from erosion (Landphair et al. 2000). This trench will serve as a flow spreader to evenly distribute flows and act as armor for the soil. Figure 6 shows an example of a check dam design. It is important for the top of the check dam to be level, so it can evenly distribute detained flows. If flows are allowed to concentrate, erosion will occur, and the check dam will have a negative effect on both the flow and water quality.

Some check dams may require an under-drain or weep holes to discharge runoff trapped after storm events. Areas with NRCS soil types A, B, or C can safely assume that any trapped water will infiltrate prior to providing mosquito breeding habitat.

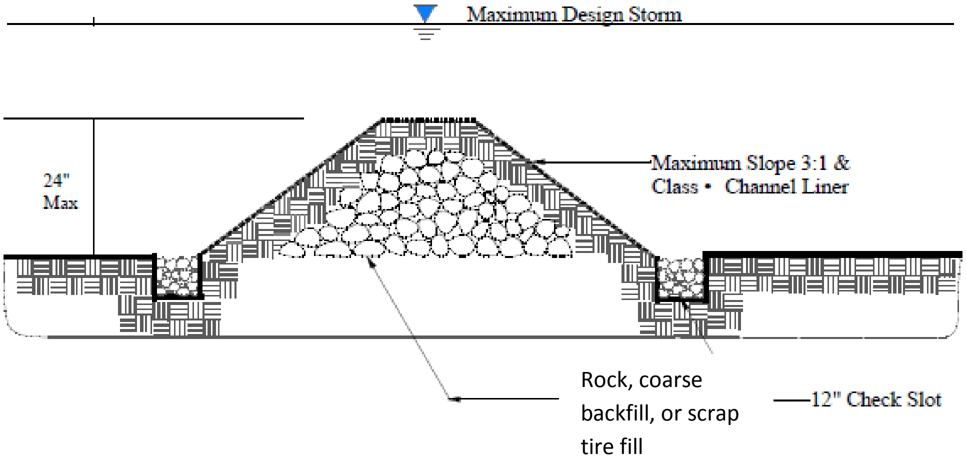

Figure 6 Check dam cross-section (Landphair et al. 2000)

The minimum spacing of check dams should be such that the lower edge of an upstream check dam is at the same elevation as the peak of a downstream check dam, as shown in Figure 7. Equation 12.10 was used to calculate the minimum check dam spacing (Landphair et al. 2000).

$$L = \frac{h}{g} \qquad (12.10)$$

Where:

L: Minimum horizontal distance between check dams (ft or m)

h: Height of check dam (ft or m)

g: Longitudinal channel slope $\left(\frac{\text{ft}}{\text{ft}} \text{ or } \frac{\text{m}}{\text{m}}\right)$

It is suggested (Landphair et al. 2000) that the check dams be placed at six times the minimum required distance. Spacing should, therefore, be found with equation 12.11:

$$L = 6 * \frac{h}{g} \qquad (12.11)$$

Spacing of check dams should also help maintain sheet flow in the BMP. Sheet flow typically channelizes after 150 ft (45.7 m) of flow over pervious ground cover (Clar et al. 2004); therefore, a check dam should be located every 150 ft (45.7 ft) regardless of whether flow velocities are calculated to be large enough to create scour (Clar et al. 2004).

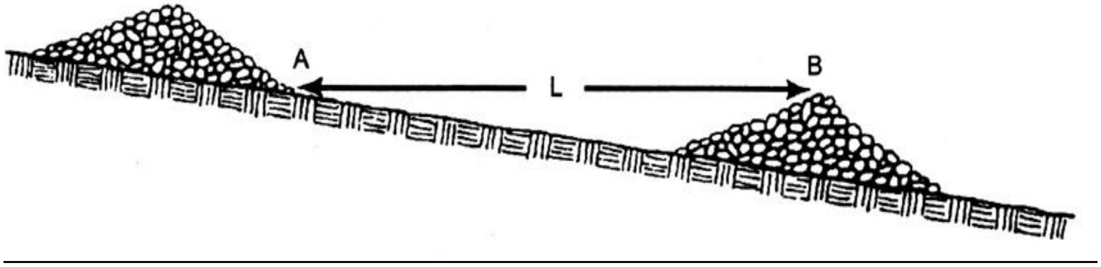

Figure 7 Minimum check dam spacing (BE 2001)

Step 8: Specify Vegetation Plan. The vegetation in vegetated swales should be able to survive periods of saturation and also be drought resistant. Plants must also be able to withstand salts associated with deicing processes necessary in Nebraska's seasonal climate. Vegetation should be limited to grasses, or other vegetation which provides low ground cover. Nebraska's regional climate and soil compositions make it impractical to identify a single seed mix for the entire state. The Nebraska Department of Roads (NDOR) has established 6 landscape regions and determined applicable grass mixtures for each. These suggested mixes are presented in Appendix E.

12.2.2.3 Design Example

A 0.5 mile long, 2-lane highway (Area = 1.94 ac; CN = 98) was being constructed, as shown in Figure 8. The highway drainage system will also have to handle run-on from an 8-ft wide grass segment running parallel to the highway (Area = 0.97 ac; CN = 80). A vegetated swale which has a longitudinal slope of 3% was being considered as a conveyance BMP for runoff from the highway which has passed through an end-of-pipe vegetated filter strip. The swale must transport the runoff 200 ft before discharging into receiving waters. To simplify the example, calculations will be done assuming no infiltration occurs in the filter strip.

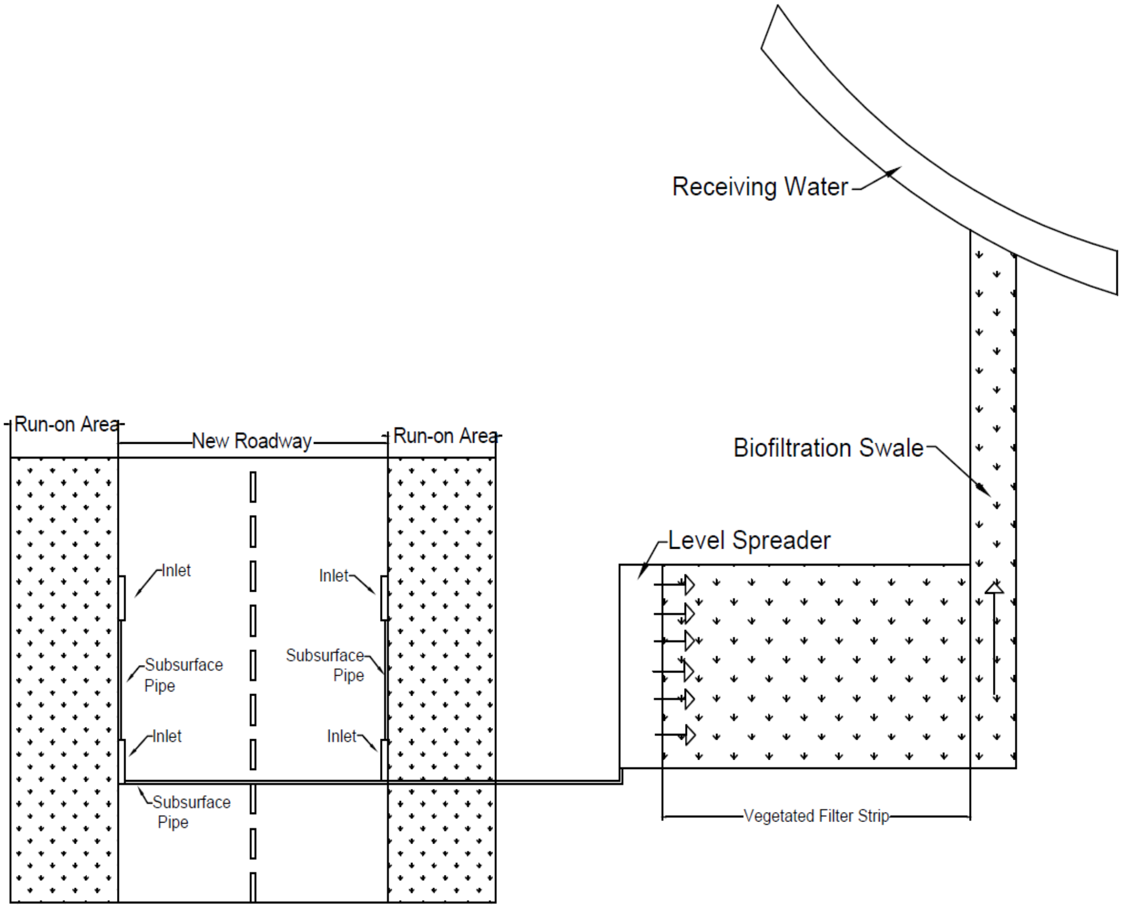

Figure 8 Plan view of vegetated swale design example

Step 1: Evaluate Applicability of Vegetated Swale Considering Site Constraints. A drainage ditch was selected to convey the flows from the vegetated filter strip to the receiving water. The drainage ditch can be designed so that it acts as a vegetated swale, thereby treating the water as it is conveyed.

Step 2: Calculate Peak Water Quality Flow (Q_{WQV}*).* Interpolation of

Table shows that the peak water quality flow is approximately 1.84 cfs from an impervious area of 1.94 acres. Table shows that there will be 0.023 in. of runoff from the run-on areas from the WQV storm. Equation 12.6 was then used to determine the flow from run-on:

$$q_p = q_u A_m Q F_p = 550\ \frac{\text{cfs}}{\text{mi}^2 * \text{in}} * 0.0015\ \text{mi}^2 * 0.023\ \text{in} * 1 = 0.019\ \text{cfs}$$

The Hydrology Section of this work contains the values for q_u in Table or Figure and Q in Table for the WQV, F_p is 1 for Nebraska.

The flow from the new development was then added to the run-on flow to find the flow occurring at the WQV storm, which results in a total flow of 1.86 cfs.

$$1.84\ \text{cfs} + 0.019\text{cfs} = 1.86\text{cfs}$$

Step 3: Dimension the Swale. Propose a side slope of 4:1 (table 12.13, max m = 3:1) with an 8 ft bottom width, as shown in Figure 9, and a longitudinal slope of 3% which matches the existing topography. If the WQV depth (from Design Step 4) or velocity (from Design Step 5) is not satisfactory, increase the bottom width and/or side slopes to reduce the values until they are within the requirements in

Table **4**.

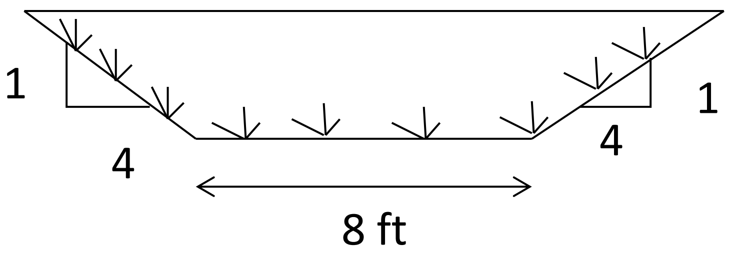

Figure 9 Design example swale cross-section

Step 4: Calculate Design Flow Depth (D_{WQV}). Equation 12.7 and the geometric elements associated with this channel were combined to obtain equation 12.8:

$$q_p=\left(\frac{k}{n}\right)*(b+my)y*\left[\frac{(b+my)y}{b+2y\sqrt{1+m^2}}\right]^{2/3}*S^{1/2}$$

$$1.86=\left(\frac{1.49}{0.24}\right)*\left((8+4y)y\right)*\left[\frac{(8+4y)y}{8+2y\sqrt{1+4^2}}\right]^{2/3}*(0.03)^{1/2}$$

The depth (y) was calculated to be 4.6 in., which is more than 4, so it is unacceptable based on the parameters in

Table **4**. The channel bottom width is already at the maximum allowable shown in

Table **4**, so either the side slope should be increased or check dams should be used to improve the design. Here, check dams will be employed to slow velocities and prevent the creation of rills along the swale.

Step 5: Calculate Design Flow Velocity (V_{WQV}). The flow and area are utilized to determine the velocity with equation 12.9:

$$v = \frac{Q}{A} = \frac{Q}{\left((8+4y)y\right)} = \frac{1.86}{\left((8+4*0.383)0.383\right)} = 0.5\ \frac{ft}{s}$$

Using the WQV flow of 1.86 cfs and the depth of 4.6 in. (0.383 ft) as found in the previous step, the velocity was 0.5 $\frac{ft}{s}$, which is less than 1 $\frac{ft}{s}$; therefore it is satisfactory.

Step 6: Check Scour Velocity for 10-yr Storm (V_S). The scour velocity was checked using the same process as the WQV design but with a 10-yr, 24-hr storm. The first step was to determine the peak flow. Peak flow was found with equation 12.6:

$$q_p = q_u * A_m * Q * F_p$$

Values for q_u for various CNs are found in Table , values for Q are found in Table .

Flow contribution from new construction (CN = 98):

$$q_p = 1100 \frac{cfs}{mi^2 * in} * 0.003 mi^2 * 4.76\ in * 1 = 15.7\ cfs$$

Flow contribution from run-on (CN = 80):

$$q_p = 1000\,\frac{\text{cfs}}{\text{mi}^2\text{*in}}\text{*}0.0015\ \text{mi}^2\text{*}2.89\ \text{in*}1 = 4.3\ \text{cfs}$$

The contributing flows were summed to find a total peak flow (q_p) of 20 cfs.

The flow depth in this BMP was found with equation 12.8:

$$20 = \left(\frac{1.49}{0.24}\right)\text{*}\big((8+4y)y\big)\text{*}\left[\frac{(8+4y)y}{8+2y\sqrt{1+4^2}}\right]^{2/3}\text{*}(0.03)^{1/2}$$

The flow depth (y) in this BMP was found to be 1.4 feet. The depth was then used to find the area which was used with the calculated flow to obtain velocity by the following equation:

$$v = \frac{Q}{A} = \frac{Q}{\big((8+4y)y\big)} = \frac{20}{\big((8+4\text{*}1.4)1.4\big)} = 1.1\ \frac{\text{ft}}{\text{s}}$$

The velocity was found to be 1.1 $\frac{\text{ft}}{\text{s}}$ which is less than the limiting velocities for all parameters shown in Table 6.

Step 7: Design Check Dams (if necessary). Since the flow depth for the water quality storm was unacceptable, a check dam is required. A check dam is also required because the swale has a length greater than 150 ft. The check dam height will be 6 in., to mitigate the unacceptable 4.6 in. flow depth for the WQV storm. Equation 5.11 was used to determine spacing of the check dams:

$$L = 6 * \frac{h}{g} = 6 * \frac{0.5 \text{ ft}}{0.03} = 100 \text{ ft}$$

The calculated spacing of 100 ft is acceptable because it does not allow flows to travel greater than 150 ft, which is the estimated length where rills begin to form for flows over pervious surfaces. The check dam will have a 5:1 front slope and 10:1 back-slope. This swale is being installed in NRCS type B soil so any water detained by the check dam after a rainfall event will infiltrate. An earthen check dam will be used. Establishment and maintenance of vegetation on the check dam will coincide with the vegetated swale.

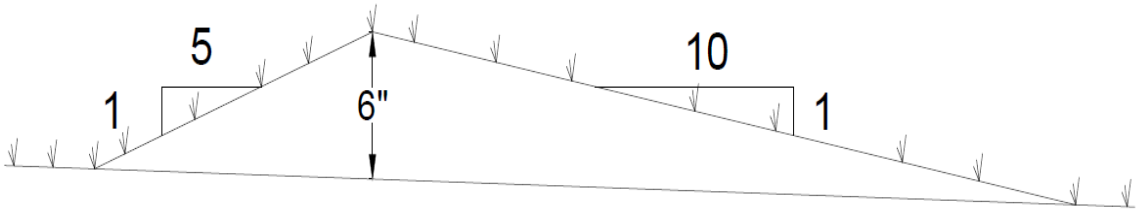

Figure 10 Swale design example check dam profile

Step 8: Specify Vegetation Plan. A grass mixture should be selected which can survive the climatic and roadway conditions (e.g., salt) expected at the site. Suggested mixtures are described in Appendix E.

12.2.3 Bioretention Cell

12.2.3.1 Design Process

Step 1: Evaluate Applicable Location Considering Site Constraints.

Step 2: Calculate Water Quality Volume to be Treated (WQV).

Step 3: Specify Filter Media Type.

Step 4: Determine Necessary Media Depth.

Step 5: Calculate Surface Area.

Step 6: Select Dimensions for Bioretention Area.

Step 7: Design Inlet System and Pretreatment.

Step 8: Design Under-drain (if necessary).

Step 9: Select and Size Overflow Method.

Step 10: Specify Vegetation Plan.

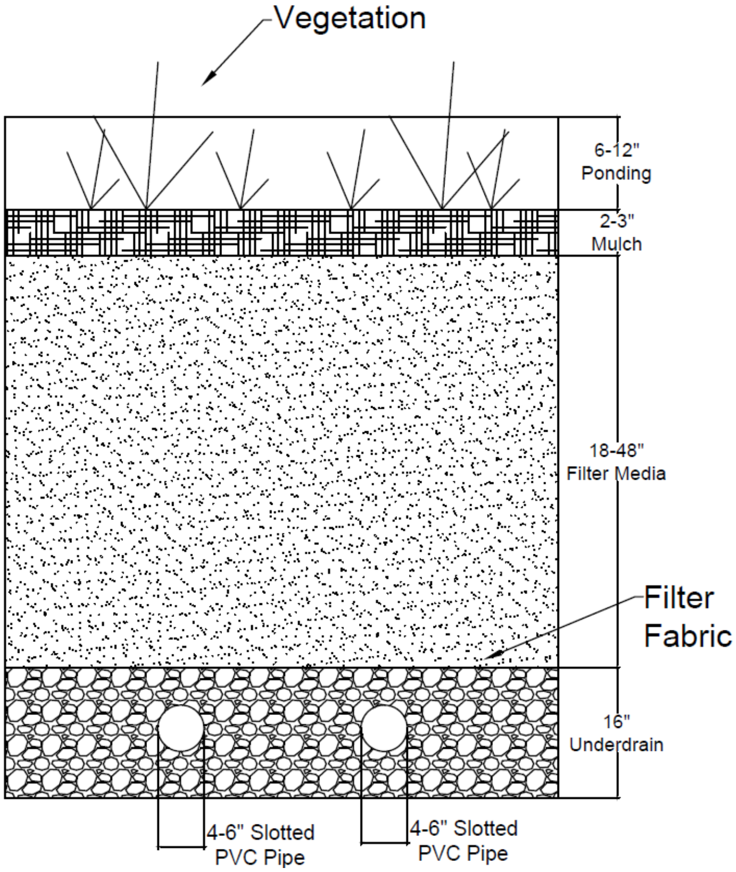

Figure 11 Bioretention cross-section

12.2.3.2 Design Criteria

Table **8** contains the criteria to be considered while working through the design process.

Table 8 Design considerations

Design Parameter	Minimum	Maximum
Depth of ponding	6 inches^{2,5}	12 inches^{2,3,4,6}
Depth of amended filter media	18 inches^{3,4,5}	48 inches^{2,6}
Ponding drawdown time	24 hours^{1,4,5}	48 hours^{2,5}
1) Atchison et al. (2006) 2) Clar et al. (2004) 3) Hartsig and Rodie (2010)	4) Hinman (2005) 5) MDEP (2009) 6) NCDENR (2007)	

Step 1: Evaluate Applicable Location Considering Site Constraints. Bioretention is a flexible BMP which can be located in many locations, from a downtown setting to the interchange of a rural highway. Bioretention's pleasing aesthetics result in a socially acceptable means of treating runoff. Bioretention is also flexible in that it does not require a large or contiguous footprint. If a watershed is too large for a single cell, there are often multiple locations on-site to facilitate the use of multiple cells. Bioretention BMPs are also strong candidates for retrofit projects due to the adaptability of their layout.

Bioretention facilities can be designed as either infiltration or filtration BMPs. Infiltration is encouraged to facilitate ground water recharge. However, when the subsurface has a permeability less than $0.5\,\frac{\text{in}}{\text{hr}}$ ($1.3\,\frac{\text{cm}}{\text{hr}}$), the bioretention cell will not drain properly and will function as a filter requiring an under drain (Davis et al. 2009). Under drains may also be included if infiltration will be detrimental to surrounding structures or roadways. Under drains should also be used when treating runoff from pollutant hot spots (e.g., gas stations).

Step 2: Calculate Water Quality Volume to be Treated (WQV). The water quality volume (WQV) is the amount of runoff requiring treatment. The water quality volume was calculated by summing the volume which came from newly constructed impervious areas and the volume of run-on from adjacent property which comingled with run off from the new development. The WQV can be found by summing equations 12.12 and 12.13:

The volume from the new development was found with equation 12.12:

$$WQV_{\text{New Dev}} = 0.5\text{in} * \frac{\text{Area Treated (ft}^2\text{)}}{12\frac{\text{in}}{\text{ft}}} \tag{12.12}$$

The volume of run-on was found with the following equation 12.13:

$$WQV_{\text{Run-On}} = Q\ (\text{in}) * \frac{\text{Area Treated (ft}^2\text{)}}{12\frac{\text{in}}{\text{ft}}} \tag{12.13}$$

Q is the runoff depth found in Table .

Step 3: Specify Filter Media Type. The filter media shall be a uniform mix, free of stones, stumps, roots or other similar objects larger than two in. (MDE 2000). Media in a bioretention cell needs to accommodate vegetation, drain adequately, and provide treatment. These goals can be accomplished with a variety of soil mixes, suggested by a variety of agencies. A common thread throughout is requiring a homogenous mix free of detritus or roots.

The Minnesota Pollution Control Agency provided two sets of soil media. The first was primarily based on water quality and was 55% to 65% construction sand, 10% to 20% top soil, and 25% to 35% organic leaf compost (MSSC 2008). The second mix was designed for enhanced filtration and includes 50% to 70% construction sand and 30% to 50% organic leaf compost (MSSC 2008). The water quality mix will have higher nutrient removal than the filtration mix, which was primarily designed to remove solids and metals. Construction sand for these two mixes should meet AASHTO M-6 or ASTM C-33 specifications (MSSC 2008), or have similar gradations as described in Appendix F. A bioretention garden design manual prepared for the Omaha region suggests a 50/50 mix of fine sand and compost or sphagnum peat mix (Hartsig and Rodie 2010). Loamy sand or sandy loam has been suggested by the North Carolina Department of Environment and Natural Resources (NCDENR 2007) and the Puget Sound Action Team (Hinman 2005), while the Maine Department of Environmental Protection suggests using a silty sand mix (MDEP 2009). The EPA has published specifications calling for loamy sand, sandy loam, or a loam, sand mix and notes that the minimum sand content should be 50%, and the maximum fines should be 10% (Clar et al. 2004). The EPA also states that amending the soil with 20% to 50% compost can be very beneficial for plant growth and pollutant removal (Clar et al 2004).

Selecting which mix is right for a certain location is at the discretion of the designing engineer. Site-specific problem pollutants should be considered, as well as media cost. If nutrient removal is the primary concern, a higher percentage of compost and top soil should be used. However, if solids or metals are the main problem using a higher percentage of sand will result in adequate treatment.

Step 4: Determine Necessary Media Depth. The depth of the filter media must be between 18 and 48 in. (45.7 to 121.9 cm) (Clar et al. 2004). The depth can vary depending on what types of pollutants require remediation. Metal concentrations have been shown to decrease exponentially while moving down through the soil column (Weiss et al. 2010). This was supported in another study which found that most metals accumulate within 4 to 8 in (10 to 20 cm) of the surface (Li and Davis 2008). Similar results were found for total suspended solids

(TSS) removal. TSS was shown to be removed within 2 to 8 in. (5 to 20 cm) of the surface (Li and Davis 2008). Lab and field tests have both shown that petroleum hydrocarbons were removed and biodegraded primarily in the layer of mulch (Davis et al. 2010). Sorbed phosphorous removal coincides with TSS removal, and dissolved phosphorous removal begins at approximately 12 in. (30.5 cm) below grade (NCDENR 2007). Nitrogen removal has been shown to begin at around 30 in. (76 cm) (NCDENR 2007). Researchers in North Carolina suggest that the addition of a permanent saturated zone, at least 12 in. deep (30.5 cm), within the media can increase nitrogen removal by facilitating de-nitrification (NCDENR 2007). An anaerobic zone can be created by having the under drain discharge through an upturned pipe or a weir in the discharge area.

Figure **12** shows general profiles of the riser pipe and weir method.

Media depth must also be thick enough to sustain the vegetation in the cell. Sufficient depth is needed for the root zone for the health of the plants and to keep roots away from the under drain system. Different types of vegetation have varying root penetration. Plant selection should be factored into selecting an adequate depth of filter media. Plants selection is discussed in step 10 of this section.

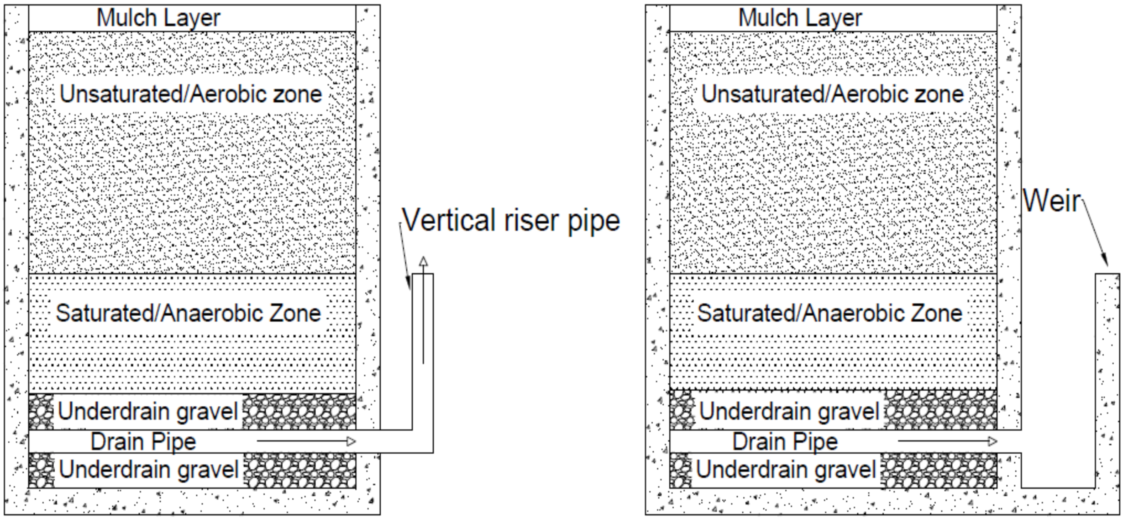

Figure 12 General saturated zone discharge designs

Step 5: Calculate Surface Area. The surface area of the bioretention facility must be large enough to accommodate the WQV while not exceeding the maximum ponding depth (6 to 12 in.). Equation 12.14 was used to determine the required surface area (NCDENR 2007):

$$A = \frac{WQV}{D_{Max\ Pond}} \tag{12.14}$$

Where:

A: Area of bioretention facility (ft^2 or m^2)

WQV: Water quality volume (ft^3 or m^3)

$D_{Max\ Pond}$: Maximum ponding depth (0.5 to 1 ft or 0.15 to 0.3 m)

Equation 12.14 conservatively calculates the required surface area due to the assumption that the entire WQV will require ponding before it enters the filter media. Equation 12.18

accounts for flow through the media and can be used if an appropriate hydraulic conductivity (K) can be found for the selected media (Clar et al. 2004).

Step 6: Select Dimensions for Bioretention Area. The bioretention system needs to be sized in conjunction with the area found in step 5. The required surface area does not need to be one centralized bioretention cell. The potential for division of the surface area over the watershed makes bioretention a flexible BMP. Although multiple cells can be employed, each bioretention cell must account for the first half inch of runoff from the sub-watershed draining into it. The Maine Department of Environmental Protection suggests that no single cell be greater than 2,000 ft^2 (186 m^2) (MDEP 2009).

Step 7: Design Inlet System and Pretreatment. Inflow to bioretention can be concentrated from a pipe, culvert, or curb, or it can enter the system as sheet flow. Bioretention cells receiving concentrated flow should incorporate a forebay which will slow runoff, reduce erosion, and function as pretreatment by allowing solids to settle out.

Figure 13 shows a properly constructed and utilized forebay. The volume of the forebay should be 0.05 in. (0.13 cm) multiplied by the impervious drainage area (Clar et al 2004). Rip-rap is suggested as lining for the forebay due to its drainage potential and its resistance to erosion during times of high flow.

Figure 13 Properly utilized forebay (NCDENR 2007)

Pretreatment will focus on removal of solids which could clog the media. Pretreatment methods for sheet flow include: grass filter strips, gravel diaphragms, or a mulch layer (MSSC 2005). Grass filter strips are excellent pretreatment systems, and their design can be found in the Vegetated Filter Strip Design Guide section of this work. Gravel diaphragm systems consist of a small gravel filled trench. These trenches should be at least 1 ft (0.3 m) wide and 2 to 3 ft (0.61 to 0.91 m) deep. The gravel fill should consist of clean washed, uniformly graded coarse aggregate to the AASHTO # 3 specification (SEMCOG 2008), as described in Appendix F. There should also be a 1 to 2 in. (2.5 to 5.1 cm) drop at the inlet to the gravel diaphragm (SEMCOG 2008). A layer of mulch can be used as pretreatment if grass is not selected as vegetation. The mulch should be 2 to 3 in. deep (5.1 to 7.6 cm) (MDEP 2009; Clar et al. 2004). Aged, shredded hard wood bark mulch is recommended (Clar et al. 2004).

When capturing runoff from gutters, a curb cut may be used, as shown in Figure 14. It is suggested to armor the entrance to the BMP from the curb cut to prevent erosion. Erosion needs to be avoided as it adds solids to the system which may result in clogging. Control measures for erosion include implementing a gravel diaphragm (as described above) or using rip-rap. The rip-rap in this case does not need to be as large as it does in forebays receiving concentrated flow. It can be decorative, as well as functional, and it can be used to complement the aesthetic appeal of the bioretention cell. Figure 14 demonstrates the use of aesthetically pleasing rip-rap to prevent erosion using a curb cut.

Figure 14 Curb cut inlet system (NCDENR 2007)

Curb cuts can be used in series to achieve a more uniform application to the bioretention cell. Using a series of curb cuts allows less flow, and velocity, entering at each location while also maintaining a curb for the majority of the roadway for traffic safety.

Step 8: Design Under-drain (if necessary). Bioretention facilities in areas where infiltration is an acceptable and possible alternative generally do not require under-drains. In fact, under-drains are not recommended in these situations to promote groundwater recharge and to decrease the impact of impervious areas on peak stream flows. However, the following situations will require the use of an under-drain:

- Inadequately drained subgrades (hydraulic conductivity ≤ 0.50 $^{in}/_{hr}$ (1.3 $^{cm}/_{hr}$)),
- Infiltration is harmful to surrounding structures (e.g., possible damage to foundations),
- The seasonal high groundwater table is within 3 ft (0.9 m) of the bottom of the bioretention cell (MSSC 2005),
- Treating a pollutant hot spot (e.g., gas station) where groundwater contamination is probable.

For situations where infiltration would be particularly harmful, a concrete vault is suggested to house the bioretention system. Not all systems which require an under-drain will call for a concrete vault encasement. Infiltration should not be avoided unless it is detrimental to the bioretention system or neighboring structures.

If required, the under-drain system will consist of 4 to 6 in. (10.2 to 15.3 cm) diameter slotted PVC pipes wrapped in geotextile and set in a 16-in. (40.6 cm) thick gravel bed at a 1% down slope to the outlet (NVPDC & ESI 1996). The gravel will over-top the pipes by at least 2 in. (5.1 cm) and conform to the AASHO #3 standard as described in Appendix F (VCSQMP 2001, NVPDC & ESI 1996). The pipes will be no more than 8 ft (2.4 m) apart (MDEP 2009). There must also be a nonwoven geotextile layer between the BMP filter media and the under drain media. The geotextile must meet the specification presented in

Table **9**.

Table 9 Geotextile specifications (VCSQMP 2001)

Geotextile property	Specification	Test
Grab strength	90 lbs	ASTM D4632
Elongation at peak load	50%	ASTM D4632
Puncture strength	24 lbs	ASTM D3787
Permitivity	$0.7\ \mathrm{sec}^{-1}$	ASTM D4491
Burst strength	180 psi	ASTM D3786
Toughness	5500 lbs	% Elongation * Grab strength
Ultraviolet resistance	70%	ASTM D4355

Step 9: Select and Size Overflow Method. Bioretention facilities can be designed as either on-line or off-line facilities. For on-line facilities any volume beyond the WQV must be allowed to bypass. For off-line facilities the WQV can be separated before it enters the system, while excess flows are allowed to bypass.

Flow splitters are the primary means for separating out the WQV before it enters the BMP.

Figure **15** is a potential layout for a bioretention cell using a flow splitter. Flow splitters can use a weir overflow device that is generally located in either a manhole or vault, as shown in Figure **16**. The elevation of the overflow weir is often set at the WQV elevation of the cell. Keeping these elevations constant will allow for bypass of flows beyond the allowable depth while ensuring the WQV enters the bioretention facility.

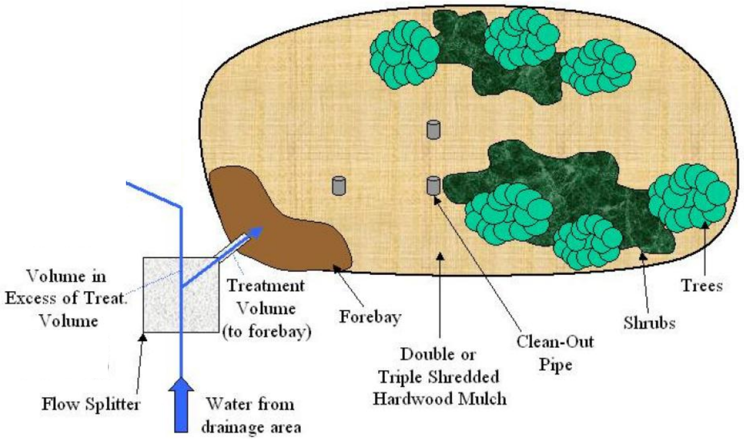

Figure 15 Off-line bioretention cell layout (adapted from NCDENR 2007)

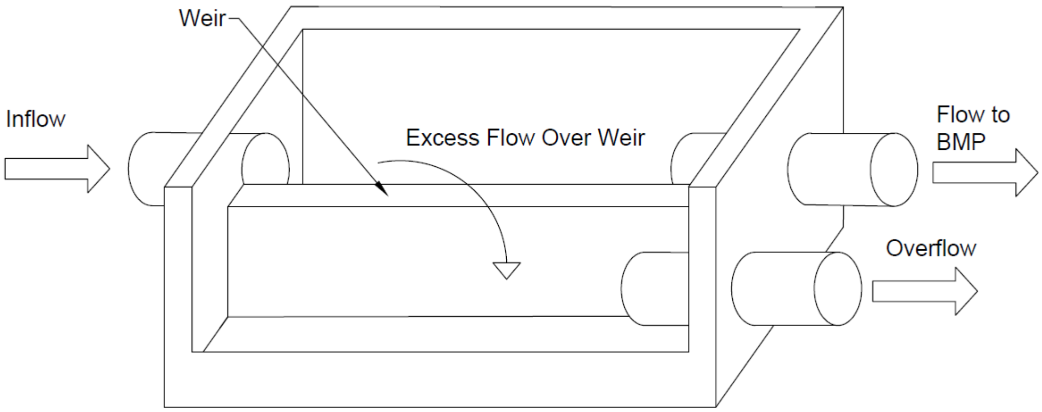

Figure 16 Flow splitter

The hydraulics of the flow splitter are very important design elements. A long weir is ideal to maximize flow rate while minimizing head. However, a longer weir will require a larger vault, which may not be as cost effective. The outlet pipe to the bioretention cell must be sized to pass the WQV regardless of storm intensity. If the pipe is inadequately sized, flows could back up and discharge over the weir prematurely.

When used as an on-line system, bioretention facilities should include an overflow structure, such as a weir or grate, to discharge excess runoff. Overflow structures should be sized to discharge volumes greater than the WQV. The outlet should be located at the design depth of the cell, which will ensure the WQV is trapped in the cell.

Step 10: Specify Vegetation Plan. Vegetation can be widely varied in bioretention cells. Although plants and shrubs are generally considered to be an integral part of the system, grass-only cells have been proven equally as effective in pollutant remediation (Davis et al. 2009), albeit without the aesthetic value which accompanies blooming plants.

The majority of vegetation used should be native to Nebraska or the Great Plains, although they can be integrated with non-native plants which are not intrusive and have proven they thrive regionally. *Nebraska Bioretention and rain Garden Plants Guide* is a publication which includes descriptions for a wide variety of applicable plants as well as their applications within bioretention facilities (Rodie & Todd 2010).

12.2.3.3 Design Example

Bioretention was selected as the BMP for a new roadway going through a developed downtown area. Bioretention was selected due to its flexibility in sizing and aesthetic benefits. A maintenance plan was developed with business owners to take care of day-to-day maintenance and monitoring of the cells.

Figure **17** is the plan view showing the area.

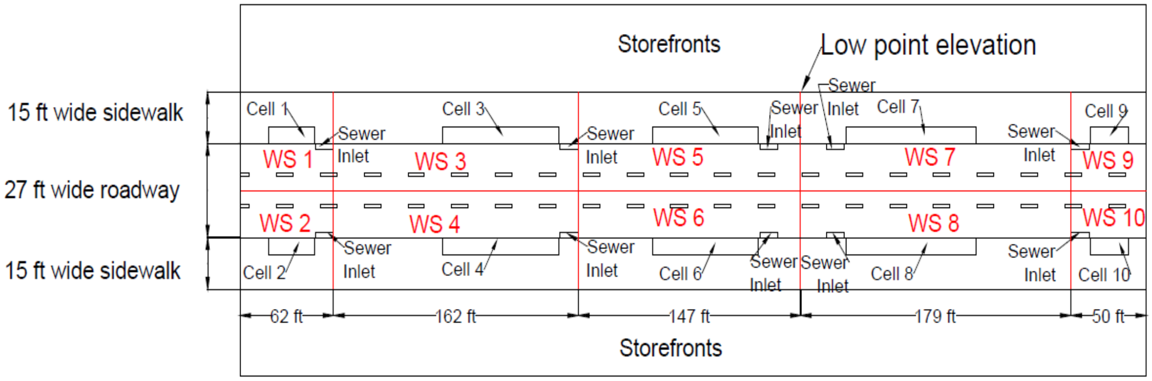

Figure 17 Site plan view for bioretention example

Step 1: Evaluate Applicable Location Considering Site Constraints. The watershed (i.e., the new roadway surface and area contributing run-on) has been broken down into 10 sub-watersheds labeled WS1-WS10 in

Figure **17**. The sub-watersheds discharge into the bioretention cells with the corresponding numbers. Each bioretention cell is responsible for treating the runoff from half of the new 27 ft wide roadway as well as the 15 ft wide sidewalk, no other run-on comingles with the roadway runoff. Each cell intrudes 5 ft into the sidewalk, which leaves 10 ft of walking room for pedestrian traffic at the bioretention areas.

Although the bioretention cells are not treating a pollutant hotspot, the seasonal high groundwater table is well below the bottom of the cells, and the subsurface has permeability greater than 0.5 $\frac{\text{in}}{\text{hr}}$, infiltration, for this example, cannot be used due to the harm it would cause the adjacent roadway and building foundations. Therefore, an under-drain system must be employed. In this situation a concrete vault should be employed to enclose each bioretention cell.

Step 2: Calculate Water Quality Volume to be Treated (WQV). The water quality volume must be calculated for each sub-watershed. The sidewalks and roadway are impervious and have a curve number of 98. The area of each sub-watershed can be found by multiplying its length by half the width of the roadway (13.5 ft) for contributing drainage area from new development or the width of the sidewalk (15 ft) for the contributing run-on area. It should be noted that the watersheds were symmetrical from the center of the road and were calculated as such.

Contributing drainage area for new development in WS1:

$$A = L * W = 62\text{ ft}*13.5\text{ ft} = 837\text{ ft}^2$$

Contributing drainage area for run-on for WS 1:

$$A = L * W = 62\text{ ft}*15\text{ ft} = 930\text{ ft}^2$$

The volume from new impervious area of WS 1 was found by using equation 12.12:

$$WQV_{\text{New Dev}} = 0.5\text{in}* \frac{837\text{ ft}^2}{12\frac{\text{in}}{\text{ft}}} = 35\text{ ft}^3$$

The run-on volume from the sidewalks in WS 1 was found by using equation 12.13:

$$WQV_{\text{Run-on}} = 0.5\text{ in}* \frac{930\text{ ft}^2}{12\frac{\text{in}}{\text{ft}}} = 39\text{ft}^3$$

The total WQV of WS 1 was found by taking the sum of equation 12.12 and equation 12.13:

$$35\text{ ft}^3 + 39\text{ft}^3 = 74\text{ft}^3$$

Table 10 shows the WQVs for the other sub-watersheds.

Table 10 WQV for each sub-watershed

WS 1 & 2	WS 3 & 4	WS 5 & 6	WS 7 & 8	WS 9 & 10
74 ft^3	192 ft^3	175 ft^3	213 ft^3	59 ft^3

Step 3: Specify Filter Media Type. The filter media for each bioretention cell will consist of 60% clean washed AASHTO M-6 sand, 5% fines, and 35% compost, per the EPA's guidance (Clar et al. 2004).

Step 4: Determine Necessary Media Depth. The depth will be designed for treatment of solids, metals, petroleum hydrocarbons (with the mulch), and sorbed phosphorous. Nitrogen and dissolved phosphorous will also be treated, but are not critical to design as they were not the priority pollutants in this case. For this reason there will not be a permanent saturated zone in these bioretention cells. Depth of roots must also be considered. Therefore, each cell will have 36 in. of filter media. Figure 18 shows the media profile for cell 2.

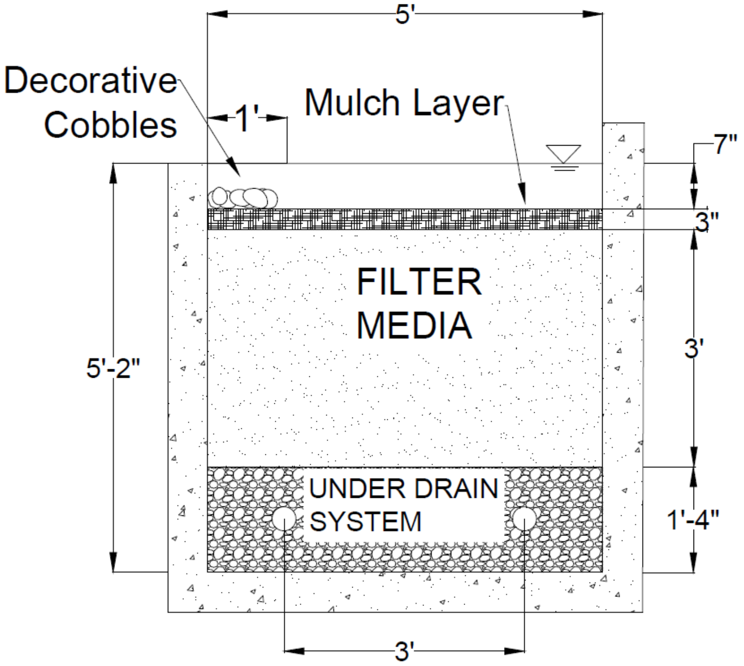

Figure 18 Cross-section A for bioretention example

Step 5: Calculate Surface Area. Equation 12.14 will be used to calculate the area of cell 1, and

Table **11** shows the results for all watersheds:

$$A_{\text{Cell 1}} = \frac{\text{WQV}}{D_{\text{Max Pond}}} = \frac{74\text{ft}^3}{0.5\text{ ft}} = 148\text{ ft}^2$$

Table 11 Required bioretention area per sub-watershed

WS 1 & 2	WS 3 & 4	WS 5 & 6	WS 7 & 8	WS 9 & 10
$148ft^2$	$385ft^2$	$350ft^2$	$426ft^2$	$119ft^2$

Step 6: Select Dimensions for Bioretention Area. The bioretention area will be limited to 5 ft wide to accommodate pedestrian traffic on the sidewalk. Each cell will run parallel to and directly adjacent to the roadway. The required length for each cell is shown in Table 12.

Figure **19** shows the plan view for cell 2.

Table 12 Required lengths for bioretention cells

Cell 1 & 2			Cell 7 & 8		
Length (ft)	Width (ft)	Area (ft^2)	Length (ft)	Width (ft)	Area (ft^2)
30.00	5	150	86.00	5	430
Cell 3 & 4			Cell 9 & 10		
Length (ft)	Width (ft)	Area (ft^2)	Length (ft)	Width (ft)	Area (ft^2)
77.00	5	385	25.00	5	125
Cell 5 & 6					
Length (ft)	Width (ft)	Area (ft^2)			
70.00	5	350			

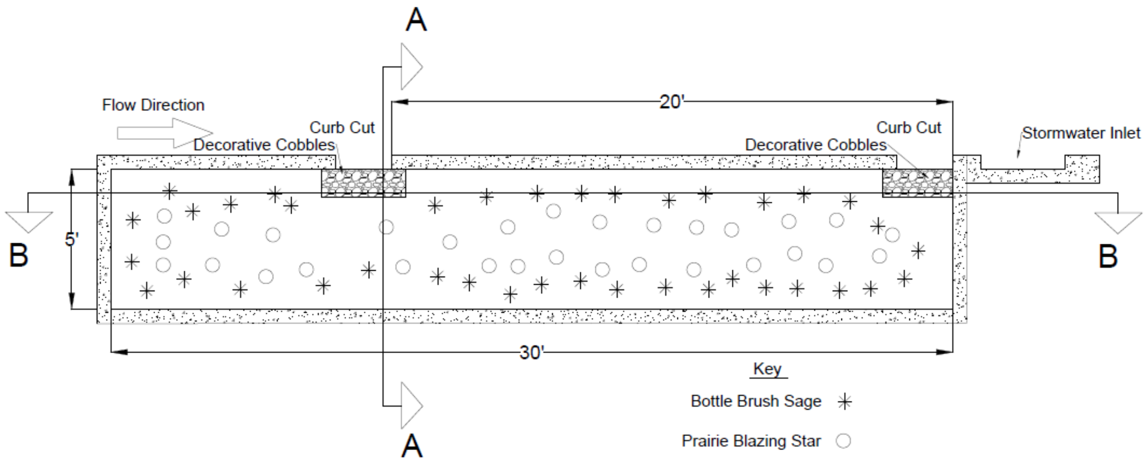

Figure 19 Plan view of Cell 2 for bioretention example

Step 7: Design Inlet System and Pretreatment. Curb cuts will be used to divert runoff to the bioretention cells. These cuts will be placed 1 ft upstream of the stormwater inlets and then every 20 ft upstream from there, as shown in

Figure **19** for cell 2. This will allow for the majority of the runoff to be captured and distributed over the length of the cell. Upon entering the cells, the runoff will be passed over decorative cobbles which will act to slow the runoff and prevent erosion. Additional removal of solids will be achieved with a 3-in. thick layer of shredded hard wood mulch spread evenly over the cells.

Step 8: Design Under-drain (if necessary). Each cell will require the installation of an under-drain. It will be 2, 4-in. diameter slotted PVC pipes spaced 3 ft apart running longitudinally down the length of the cells. Two pipes are used to ensure functionality if one clogs. The pipes will be laid in a 16-in. deep bed of gravel, with 6 in. of gravel above the pipe and 6 in. below. The pipes will have a 1% slope towards the outlet. A geotextile to the specifications presented in table 12.18 will overlay the gravel to prevent transport of the filter media into the gravel layer and outlet pipe. The pipe will discharge into the existing sewer system, as shown in Figure 20.

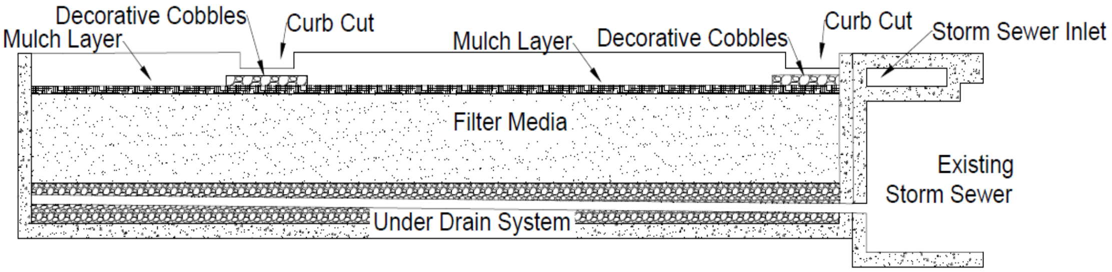

Figure 20 Cross-section B of Cell 2 for bioretention example

Step 9: Select and Size Overflow Method. This system will not incorporate an overflow system. The bottom of the curb cuts will be positioned at the same elevation as the design depth of the WQV. This orientation will allow for volumes greater than the WQV to either discharge from the curb cuts or flow by without entering the cell.

Step 10: Specify Vegetation Plan. The vegetation for each cell will be a mix of local herbaceous grasses and flowers. Bottlebrush sedge will be coordinated with prairie blazing star in each cell. These species are both well-suited for saturated conditions. Drought conditions should also be factored into plant selection. Although these plants are not drought resistant, the local business owners who are doing the day-to-day maintenance of these cells will water them between rainfall events. Areas with less intensive maintenance opportunities should put a greater emphasis on drought resistance.

12.2.4 Basin Sand Filter

12.2.4.1 Design Process

Step 1: Evaluate Applicable Location Considering Site Constraints.

Step 2: Calculate Water Quality Volume to be Treated (WQV).

Step 3: Size Sediment Basin.

Step 4: Determine Filter Media Characteristics.

Step 5: Select Filter Bed Depth.

Step 6: Calculate Filter Surface Area.

Step 7: Design Sediment Basin Outlet Riser.

Step 8: Specify Filter Inlet Characteristics.

Step 9: Design Under-drain.

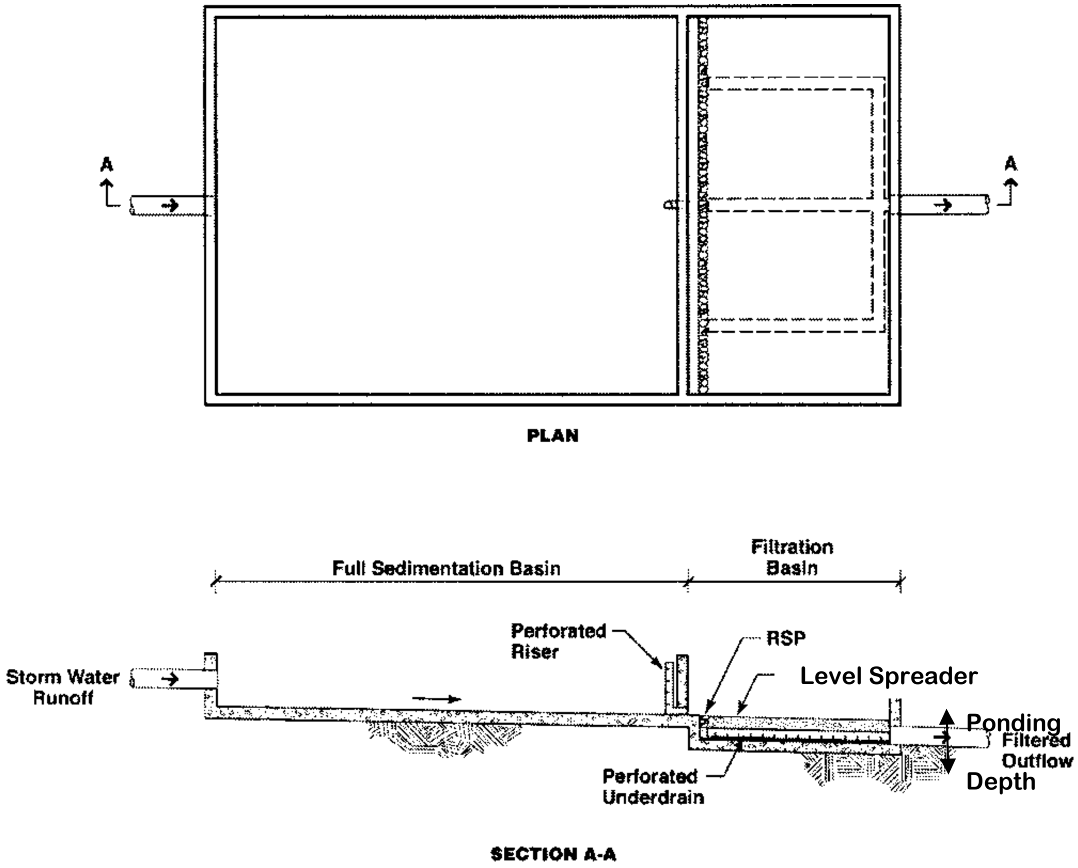

Figure 21 Sand filter design (Barrett 2003)

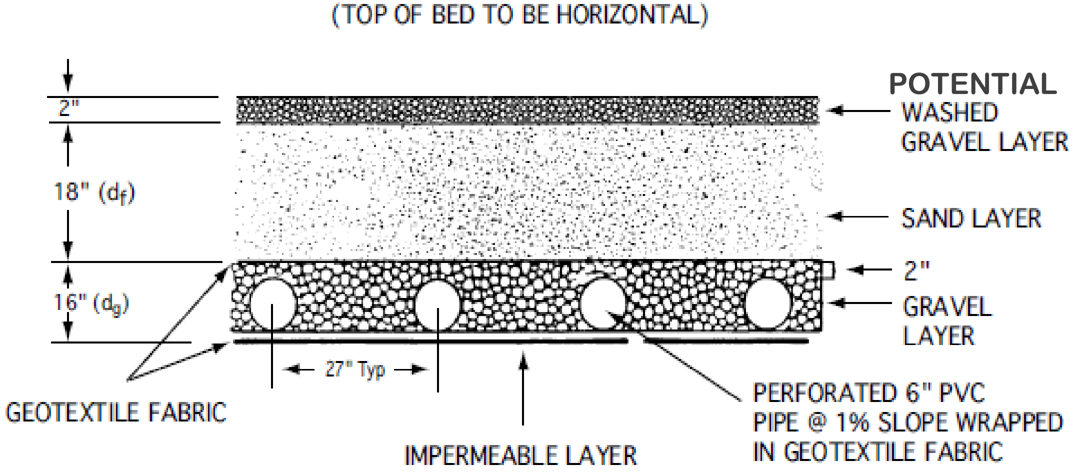

Figure 22 Filter bed cross section (NVPDC 1996)

12.2.4.2 Design Criteria

Table 13 contains the criteria to be considered while working through the design process.

Table 13 Design considerations for basin filters

Design Parameter	Minimum	Maximum
Sediment basin layout (L:W)	$2:1^{3,5,7,8}$	$4:1^{8}$
Depth of filter media	$18\ \text{inches}^{1,2,3,4,5,6,7}$	-
Infiltration rate of filter media	$3.5\ \frac{\text{in}}{\text{day}}^{2,4,5,6,7,8}$	
Diameter of under drain pipes	$6\ \text{inches}^{1,2,4,7}$	
Slope of under drain pipes	$1\%^{1,5,7,8}$	
Slope of sedimentation basin	$2\%^{1}$	
Time for filter surface drawdown	$40\ \text{hrs}^{2,6,7,8}$	
Drawdown time for sediment basin	$24\ \text{hrs}^{7}$	
1) CalTrans (2010c) 2) Landphair et al. (2000) 3) KCDNRP (2009) 4) SEMCOG (2008)	5) MSSC (2005) 6) NCDENR (2007) 7) NVPDC (1996) 8) VCSQMP (2001)	

Step 1: Evaluate Applicable Location Considering Site Constraints. Filtration systems are a viable option for ultra-urban situations due to their small footprint and layout flexibility. They can be located at stormwater inlets and discharge into the existing sewer system. Sand filters can also serve as end-of-pipe BMPs with the forebay acting as an energy dissipater.

Sand filters perform best when treating highly impervious watersheds (MSSC 2005). Impervious watersheds contribute less total suspended solids, thus limiting the amount of fines entering the system (CalTrans 2004). Treating impervious areas will extend the life of the filter as well as reduce maintenance costs.

Step 2: Calculate Water Quality Volume to be Treated (WQV). The water quality volume (WQV) is the amount of runoff requiring treatment. The water quality volume was calculated by summing the volume which comes from newly constructed impervious areas and the volume of run-on from adjacent property which comingles with run off from the new development. The volume from impervious areas can be found with equation 12.15:

$$\mathrm{WQV}_{\mathrm{New\ Dev}} = 0.5\mathrm{in} * \frac{\mathrm{Area\ Treated\ (ft^2)}}{12\frac{\mathrm{in}}{\mathrm{ft}}} \qquad (12.15)$$

The volume running off pervious areas was found with equation 12.16:

$$\mathrm{WQV}_{\mathrm{Run-On}} = \mathrm{Q\ (in)} * \frac{\mathrm{Area\ Treated\ (ft^2)}}{12\frac{\mathrm{in}}{\mathrm{ft}}} \qquad (12.16)$$

The runoff depth (Q) can be found in Table .

Step 3: Size Sediment Basin. The sediment basin should be sized to retain the entire WQV. A riser pipe will discharge into the infiltration basin. The basin geometry should have at least a 2:1 length-to-width ratio (NVPDC & ESI 1996). This ratio will facilitate the settlement of particles within the basin. The inlet and riser pipe outlet should be on opposite ends of the basin to promote residence time and to decrease the amount of dead zones within the system. Runoff should enter the basin at 3 $\frac{ft}{s}$ (0.9 $\frac{m}{s}$) or less. An energy dissipation device, such as a rip-rap apron or basin, should be used for larger velocities (NVPDC & ESI 1996).

The minimum surface area of the sediment basin was calculated using equation 12.17 (Camp Hazen Equation) (NCDENR 2007):

$$A_B = -\left(\frac{Q_o}{w}\right) * (\ln(1 - E)) \tag{12.17}$$

where:

A_B: Surface area of sedimentation basin (ft^2 or m^2)

Q_o: Outflow (cfs or cms)

E: Trap efficiency of the chamber (unitless) (E = 0.9) (KCDNRP 2009)

w: Critical settling velocity of particle $\left(\frac{ft}{s} \text{ or } \frac{m}{s}\right)$

Settling velocity is a function of particle size, and therefore, percent imperviousness of the watershed. For watersheds with ≥ 75% impervious, w = 0.0033 $\frac{ft}{s}$ and for watersheds < 75%, w = 0.0004 $\frac{ft}{s}$ (KCDNRP 2009).

Sedimentation chambers should be at least 1.5 feet (0.46 m) wide (parallel to flow) (NCDENR 2007), with an L:W ratio between 4:1 and 2:1 (Table 13) (VCSCQMP 2001). Ponding depth in the sedimentation basin should be 2 to 6 ft (0.61 to 1.8 m) (CEI & NHDES 2008).

Step 4: Determine Filter Media Characteristics. Filter media can be sand or a mixture of sand, mulch, clay, or wood fiber. Different mixes have varying hydraulic characteristics, pollutant removal capabilities, and costs. Costs are largely based upon the availability of the media in question. Regardless of the media mixture, an infiltration rate of 3.5 $\frac{in}{day}$ (8.9$\frac{cm}{day}$) must be maintained throughout the life of the system. If sand is the only media being used, it should be similar to ASTM C-33 Concrete Sand, as described in Appendix F (NVPDC &ESI 1996). The King County Surface Water Design Manual (KCDNRP 2009) suggests use of sand meeting the specifications presented in Table 14, which is based on the weight of sand which will pass standard sieves. Each of these sand specifications is ideal due to the small portions of fines they contain. Fines should be avoided in the filter media to avoid premature media clogging.

Table 14 Sand Media Specifications

U.S. Sieve Size	**Percent passing**
U.S. No. 4	95% to 100%
U.S. No. 8	70% to 100%
U.S. No. 16	40% to 90%
U.S. No. 30	25% to 75%
U.S. No. 50	2% to 25%
U.S. No. 100	Less than 4%
U.S. No. 200	Less than 2%

(KCDNRP 2009)

Step 5: Select Filter Bed Depth. As shown in Table 13, the filter depth must be at least 18 in. (45.7 cm) deep (KCDNRP 2009). The minimum is acceptable but may require more labor intensive maintenance. A deeper filter bed will allow for the top 2 in. (5 cm) where the majority of clogging occurs, (CalTrans 2004 & Hatt et al. 2010), to be removed without the immediate addition of more media.

Hydraulic requirements may limit the depth of media. The elevation change between the inlet and outlet must exceed the total depth of the water over the filter, the filter media, and the under-drain system (CalTrans 2004). Deeper media may not allow gravity flow through the system and into existing sewer systems. Pumping can be employed but increases expenses and potential problems.

Step 6: Calculate Filter Surface Area. The surface area of the filter was determined by equation 12.18, the Austin Sand Filter Equation (NCDENR 2007):

$$A_f = \frac{WQV * d_f}{K(h+d_f)t} \qquad (12.18)$$

where:

A_f = Surface area of sand bed (ft^2 or m^2)

WQV: Water Quality Volume (ft^3 or m^3)

d_f= sand bed depth (ft or m)

K = Hydraulic conductivity for sand filter (0.29 $\frac{ft}{hr}$ or 0.088 $\frac{m}{hr}$) (NCDENR 2007)

h = average depth of water above surface of sand media (ft or m); half of maximum ponding depth

t = time required for runoff volume to pass through filter media (hours)

The average filter head (h) is half of the maximum filter head. Ponding above the filter should be limited to 6 in. (15.2 cm) (SEMCOG 2008) to ensure drainage in 40 hrs.

Step 7: Design Sediment Basin Outlet Riser. The riser between the sediment basin and the filter bed should be designed to drawdown the WQV within 24 hrs (NVPDC & ESI 1996). There should be a grate around the riser which will act as a trash rack preventing debris from clogging the orifices.

Figure **23** shows a profile view of a riser pipe.

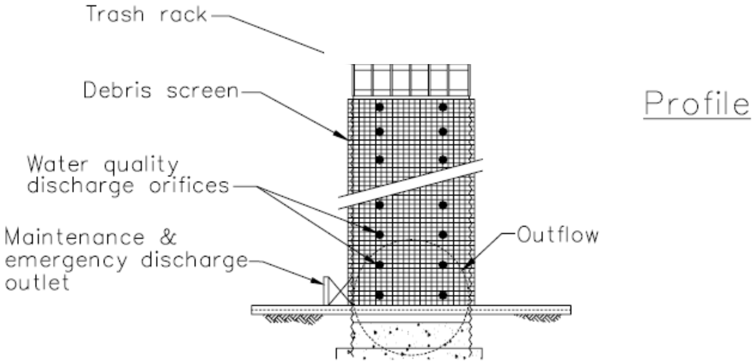

Figure 23 Profile of riser pipe (CASQA 2003)

Riser pipe design was done using equation 12.19 (CASQA 2003):

$$a_t = \frac{2A(h_{max})}{3600CT(2g[h_{max} - h_{centroid\ of\ orifices}])^{0.5}} \qquad (12.19)$$

Where:

a_t: Total area of orifices (ft^2)

A: Surface area of sedimentation basin (ft^2)

h_{max}: Maximum height from lowest orifice to highest water level (ft)

$h_{centroid\ of\ orifices}$: Height from lowest orifice to centroid of orifices (ft)

C: Orifice coefficient (0.66 for pipe material equal to or less than the diameter of the orifice or 0.8 for pipe material thicker than the diameter of the orifice) (CASQA 2003)

T: Drawdown time of full basin (hrs)

g: Gravity (32.2 $\frac{ft}{s^2}$)

In order to maintain drainage if an area of the riser is clogged, orifices should be placed on the riser in 2 even rows. These rows should be 120 degrees apart horizontally. Vertical spacing between holes should be three times the diameter of the hole (CASQA 2003). This spacing will protect against clogging of multiple holes simultaneously.

Step 8: Specify Filter Bed Inlet Characteristics. Discharge from the riser pipe must be evenly and safely distributed over the area of the filter. Concentrated flows could create scour or short circuiting of the filtration process. For this purpose energy dissipaters or flow spreaders are required at the filter bed inlet.

The King County Surface Water Design Manual suggests criteria for an effective flow spreader (KCDNRP 2009):

> “a) If the sand filter is curved or an irregular shape, a flow spreader shall be provided for a minimum of 20% of the filter perimeter.
>
> b) If the length-to-width ratio of the filter is 2:1 or greater, a flow spreader must be located on the longer side and for a minimum length of 20% of the facility perimeter.
>
> c) In other situations, use good engineering judgment in positioning the spreader.”
>
> Figure 24 demonstrates placement of flow spreaders for irregular shapes as discussed above.

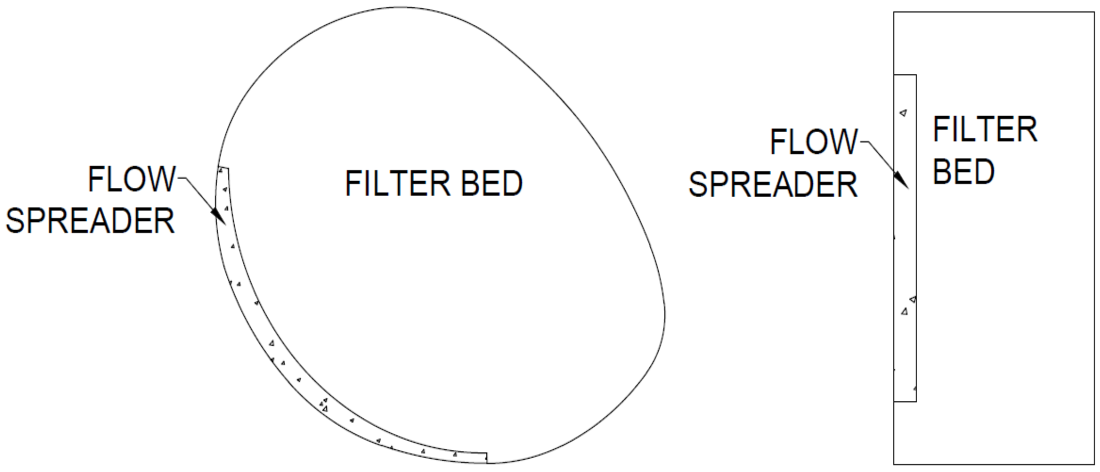

Figure 24 Flow spreader placement for irregular shaped filters

The King County Surface Water Design Manual (KCDNRP 2009) also requires 1 ft (0.3 m) of erosion protection between the flow spreader and the filter bed. The uses of weighted-down geotextile or coarse aggregates are acceptable erosion protection practices. Figure 25 shows a profile of the transition between the sediment basin and the filter bed. Level spreaders constructed from concrete must utilize weep holes so the entire WQV can drain into the filter bed.

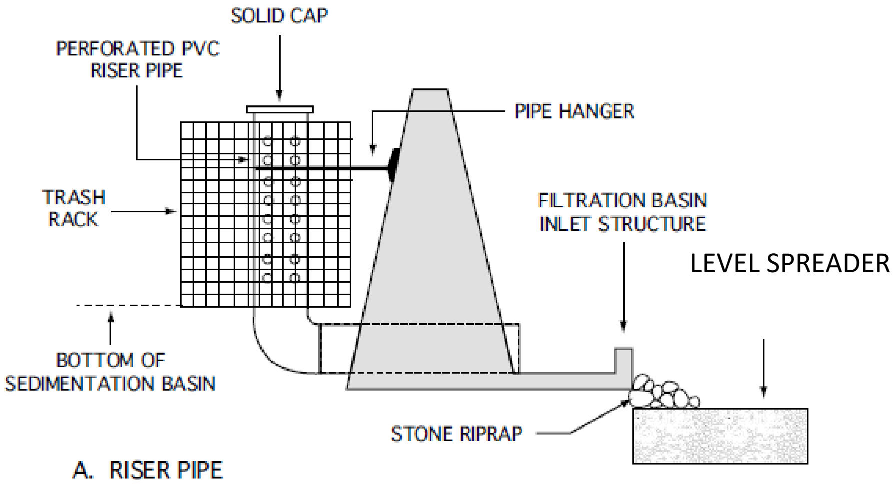

Figure 25 Transition from sedimentation basin to filter bed (NVPDC 1996)

Step 9: Design Under-drain. Once the runoff has passed through the filter media it will be collected and discharged by an under-drain system. This system will be composed of 6-in. diameter slotted PVC pipes wrapped in geotextile and set in a 16-in. thick gravel bed at a 1% down-slope to the outlet (NVPDC & ESI 1996). The gravel will over top the pipes by at least 2 in. and conform to the AASHO #3 standard as described in Appendix F (VCSQMP 2001, NVPDC & ESI 1996). The pipes will be no more than 10 ft apart (NVPDC & ESI 1996). There must also be a nonwoven geotextile layer between the filter and under-drain media. The geotextile must meet the specification presented in

Table **15**.

Table 15 Geotextile specifications (VCSQMP 2001)

Geotextile property	Specification	Test
Grab strength	90 lbs	ASTM D4632
Elongation at peak load	50%	ASTM D4632
Puncture strength	24 lbs	ASTM D3787
Permitivity	$0.7\ \mathrm{sec}^{-1}$	ASTM D4491
Burst strength	180 psi	ASTM D3786
Toughness	5500 lbs	% Elongation * Grab strength
Ultraviolet resistance (% strength after 500 Weatherometer hours)	70%	ASTM D4355

12.2.4.3 Design Example

A newly constructed section of urban highway requires treatment of runoff from 0.8 acres of impervious surface (CN 98) and 0.2 acres of adjacent grass (CN 83). The area requires the use of a BMP with a relatively small footprint, so a sand filter was selected. Figure 26 shows the plan view of the site.

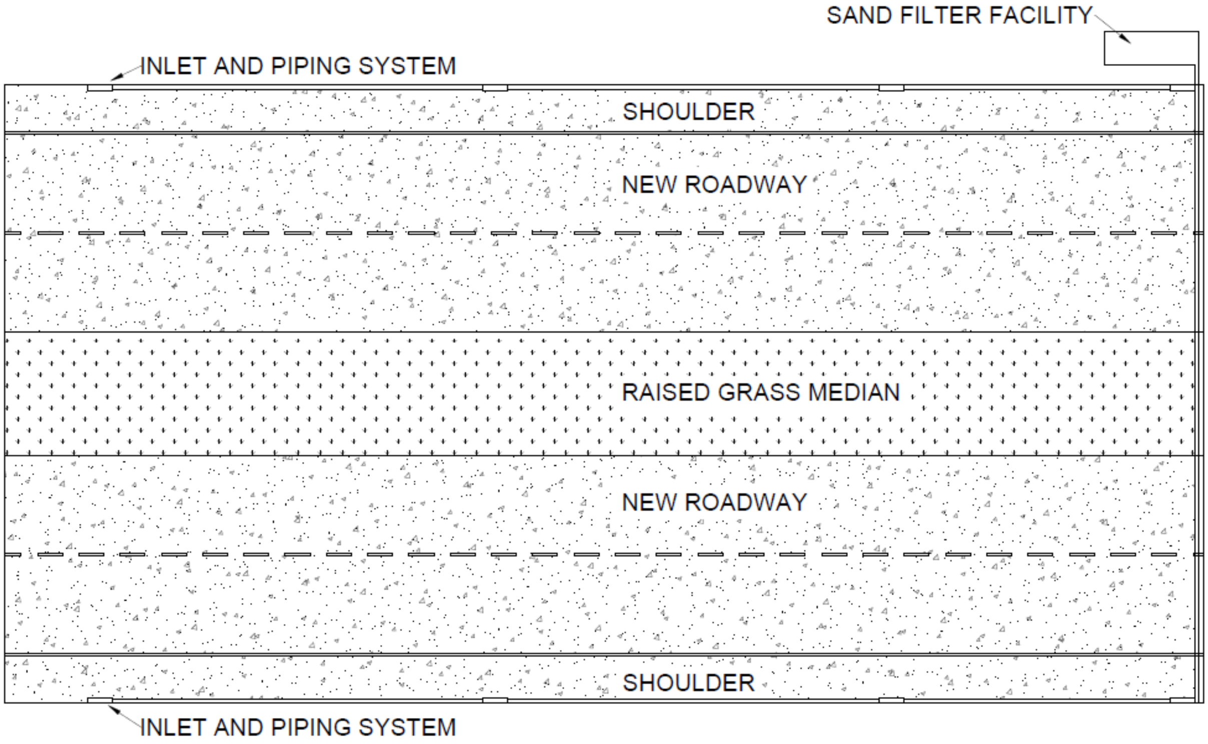

Figure 26 Example site plan view for basin sand filter

Step 1: Evaluate Applicable Location Considering Site Constraints. A sand filter was selected for this location because the watershed is highly impervious, and the available land in the right-of-way is very limited.

Step 2: Calculate Water Quality Volume to be Treated (V_{WQ}). The water quality volume can be found by summing the volumes of runoff from the pervious and impervious surfaces. This can be done through summing equations 12.15 and 12.16:

$$WQV_{New\ Dev} = 0.5in * \frac{Area\ Treated\ (ft^2)}{12\frac{in}{ft}}$$

$$WQV_{Run-On} = Q * \frac{Area\ Treated\ (ft^2)}{12\frac{in}{ft}}$$

Table shows a Q of 0.049 in. for CN = 83.

$$\mathrm{WQV_T} = 0.5\ \mathrm{in}^* \frac{0.8\ \mathrm{Ac}^*43560\frac{\mathrm{ft}^2}{\mathrm{Ac}}}{12\frac{\mathrm{in}}{\mathrm{ft}}} + 0.049\mathrm{in}\ * \frac{0.2\mathrm{Ac}^*43560\frac{\mathrm{ft}^2}{\mathrm{Ac}}}{12\frac{\mathrm{in}}{\mathrm{ft}}} = 1488\mathrm{ft}^3$$

Step 3: Size Sediment Basin. The sediment basin needs to be sized to store the WQV and to drain within 24 hrs. Equation 12.17 was used to find the minimum required area of the sediment basin:

$$A_B = -\left(\frac{Q_o}{w}\right) * (\ln(1-E))$$

$$A_B = -\left(\frac{\frac{1488ft^3}{24\ hrs} * \frac{1\ hr}{3600\ s}}{0.0033\ \frac{ft}{s}}\right) * (\ln(1-0.9)) = 12\ ft^2$$

Settling velocity (w) was $0.0033\ \frac{ft}{s}$ because the contributing watershed had greater than 75% impervious area. A $12\ ft^2$ sedimentation basin would require a depth of 124 ft, which is unacceptable. With limiting depths of 2 to 6 ft, a 33x11 ft sedimentation chamber with a depth of 4 ft will be used. This configuration allows for a 3:1 ration which provides an adequate flow path while also storing the WQV at a depth of 4 ft, a 6-in. free board will be included.

Figure **27** shows the orientation of the sedimentation basin.

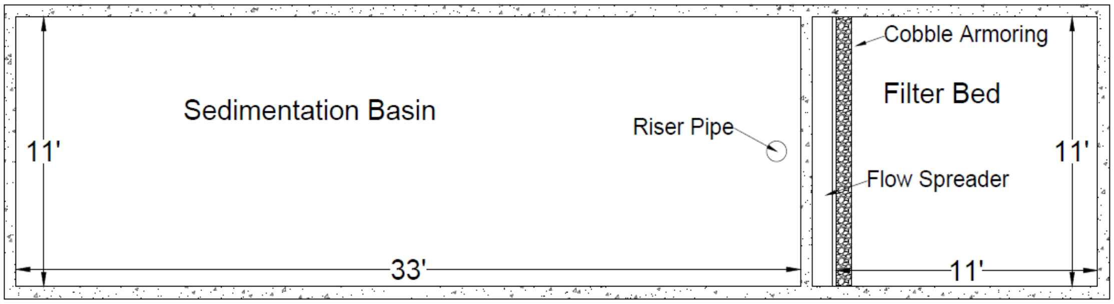

Figure 27 Plan view of example sand filter

Step 4: Determine Filter Media Characteristics. Because there are no special requirements for treatment, the filter will be composed entirely of sand which adheres to the ASTM C-33 Concrete Sand standard. Fines should be avoided as they can clog the media.

Step 5: Select Filter Bed Depth. This filter bed will be 24 in. deep initially. This will allow for maintenance crews to remove the top 3 in. 2 times before requiring additional sand be brought in to replenish the system.

Figure **28** shows the cross-section of the system.

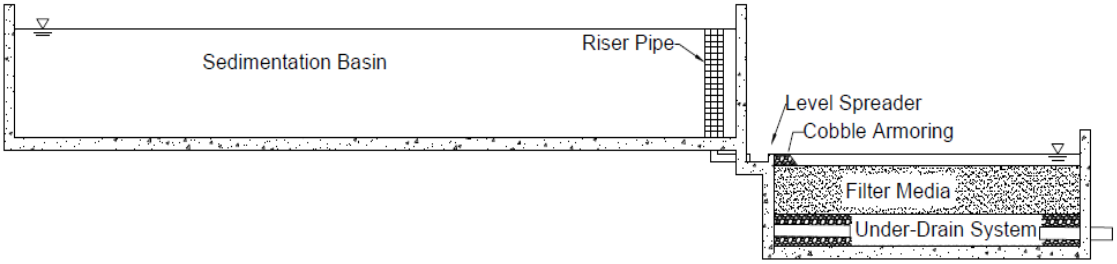

Figure 28 Profile view of example sand filter

Step 6: Calculate Filter Surface Area. Equation 12.18 was used to determine the required surface area of the filter bed.

$$A_f = \frac{WQV * d_f}{k(h + d_f)t}$$

$$A_f = \frac{(1488\, ft^3) * 2ft}{0.29\, \frac{ft}{hr}(0.25\, ft + 2\, ft) * 40\, hrs} = 114\, ft^2$$

The dimensions of the filter bed will be 11 x 11 ft, which provides 121 ft^2 of surface area. Figure **27** shows the plan view of the system.

Step 7: Design Sediment Basin Outlet Riser. The required area of holes in the riser pipe can be found with equation 12.19:

$$a_t = \frac{2A(h_{max})}{3600CT(2g[h_{max}\text{-}h_{centroid\ orifices}])^{0.5}} = \frac{2(363)(4)}{3600*0.66*24(2(32.2)[4\text{-}2])^{0.5}} =$$

$$0.0045\ \text{ft}^2 = 0.65\ \text{in}^2$$

Using the geometry of the sedimentation basin to determine a maximum depth of 4 ft and using a riser height of 4 ft, an area requirement of 0.65 in^2 is found.

An orifice diameter of 0.25 in. was selected for this riser. This diameter requires 13 orifices to account for the total required orifice area. These orifices will be positioned in two parallel columns 120 degrees apart from each. They will be vertically spaced 7 in. apart beginning 6 in. above the bottom of the sedimentation basin. Figure 29 shows the orifice spacing.

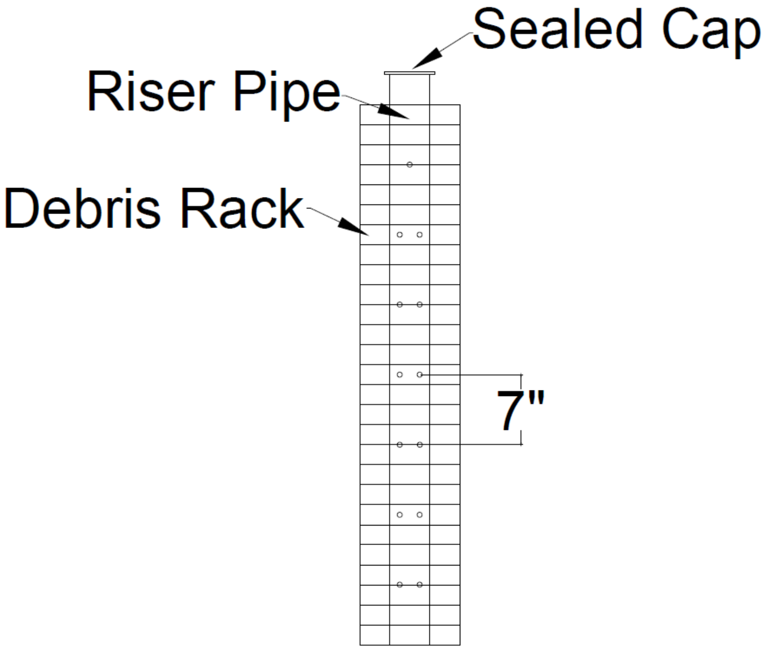

Figure 29 Detail of example riser pipe

Step 8: Specify Filter Inlet Characteristics. The level spreader in the filter bed will be a 4-in. deep, 1-ft wide reinforced concrete trench. The trench will run against the wall the filter vault shares with the sedimentation basin. Water will discharge over a one-foot wide strip of coarse gravel as it enters the sand filter.

Step 9: Design Under-drain. The under-drain will consist of a 16-in. deep coarse aggregate layer which has 2, 6-in. diameter perforated PVC pipes which run the width of the chamber and slope down to the outlet at 1%. The pipes will be 3 ft from the outside walls. The uphill end of the PVC will be 5 in. beneath the top of the gravel layer. That depth will increase as the pipes slope downward. The two pipes will feed into a 6-in. collector pipe at the downhill edge of the filter chamber which will be discharged through a single outlet. There will be a geotextile between the sand layer and the gravel layer, as well as around the pipes, which conforms to the requirements set out in

Table **15**. Figure 30 shows the layout of the under-drain system.

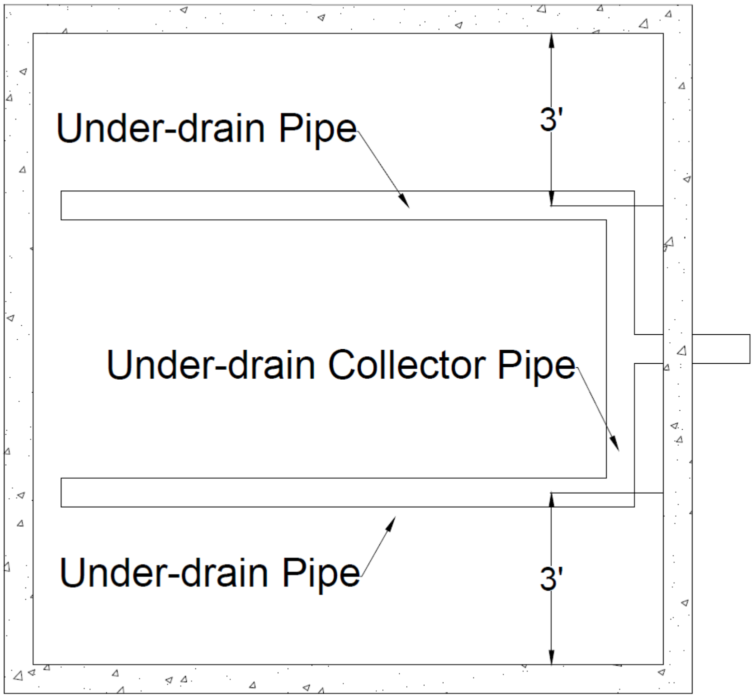

Figure 30 Under-drain layout for example sand filter

12.2.5 Horizontal Filter Trench

12.2.5.1 Design Process

Step 1: Evaluate Applicable Location Considering Site Constraints.

Step 2: Calculate Water Quality Volume to be Treated (WQV).

Step 3: Select Filter Media Specifications.

Step 4: Select Armoring Specifications.

Step 5: Calculate Trench Dimensions.

Step 6: Verify Armoring Size by Checking Scour Potential.

Step 7: Select Pretreatment.

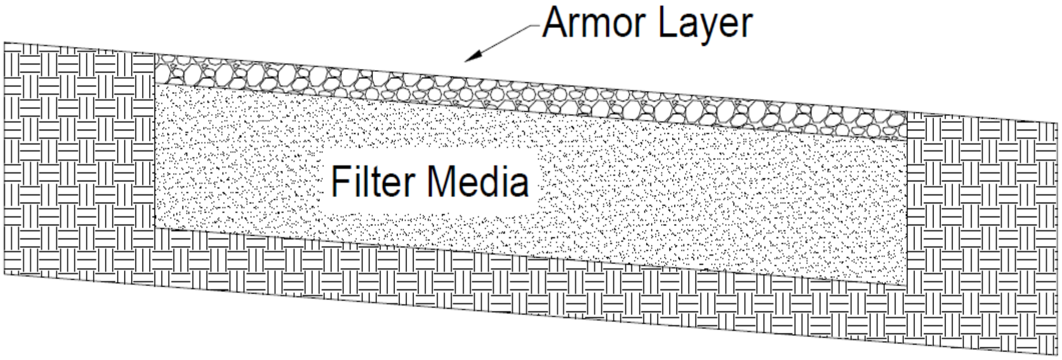

Figure 31 Profile of filter trench length

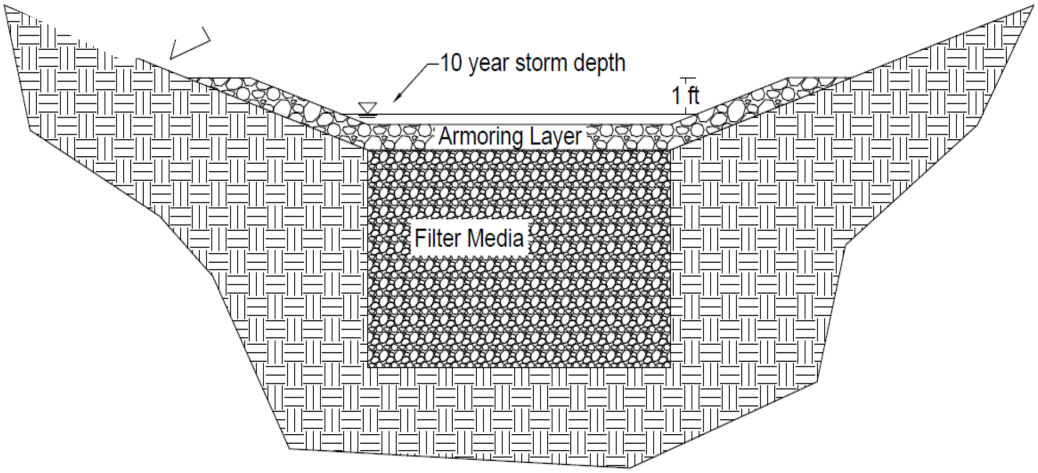

Figure 32 Profile of filter trench width

Step 1: Evaluate Applicable Location Considering Site Constraints. Horizontal filter trenches are best suited for linear applications. Prime siting areas are in ditches or swales along roadways and as end of pipe treatment systems. There is high retrofit potential for horizontal filter trenches in existing roadside drainage ditches. Although some existing infrastructure adds water quality benefits already, a horizontal filter trench can be placed in the bottom of drainage ditches if existing vegetation is insufficient to treat the runoff, or if expected flows will damage vegetated systems. Filter trenches may not be cost effective if the existing ditch is wide with gentle side sloped due to armoring requirements. However, these types of channels may already act as vegetated filter strip/vegetated swale systems, or they could with minor modifications. The trench can be placed the entire length of the treated roadway or downstream from the treated area, depending on site constraints such as availability of land in the right-of-way and slope adjacent to the roadway. Roadside vegetation on the slope leading to the bottom of the ditch may also act as a pretreatment for solids removal.

Horizontal filters are designed for use on sloped surfaces. If there is no slope, the filter will not be able to discharge and will act as an infiltration trench. Horizontal filter trenches should not be used as infiltration trenches in the following situations:

However, the following situations will require the use of an under-drain:

- Inadequately drained subgrades (hydraulic conductivity $\leq 0.50\ {}^{\text{in}}/_{\text{hr}}$ ($1.3\ {}^{\text{cm}}/_{\text{hr}}$)),
- Infiltration that is harmful to surrounding structures (e.g., possible damage to foundations),
- The seasonal high groundwater table is within 3 ft (0.9 m) of the bottom of the bioretention cell (MSSC 2005),
- Treating a pollutant hot spot (e.g., gas station) where groundwater contamination is probable.

Step 2: Calculate Water Quality Volume to be Treated (WQV). The water quality volume (WQV) is the amount of runoff requiring treatment. The water quality volume was calculated by summing the volume which comes from newly constructed impervious areas and the volume of run-on from adjacent property which comingles with run-off from the new development. The

WQV can be found by summing equation 12.20, which calculates the volume coming off newly developed areas, and equation 12.21, which calculates the volume of run-on:

$$\mathrm{WQV}_{\mathrm{New\ Dev}} = 0.5\mathrm{in} * \frac{\mathrm{Area\ Treated\ (ft^2)}}{12\frac{\mathrm{in}}{\mathrm{ft}}} \qquad (12.20)$$

$$\mathrm{WQV}_{\mathrm{Run-On}} = \mathrm{Q\ (in)} * \frac{\mathrm{Contributing\ Area\ (ft^2)}}{12\frac{\mathrm{in}}{\mathrm{ft}}} \qquad (12.21)$$

Q is the runoff depth found in Table .

Step 3: Select Filter Media Specifications. The filter media should be $^3/_8 - ^3/_4$ in. (0.95–1.9 cm) clean washed media. There should be very few fines to avoid clogging and to prolong the life of the BMP. Potential media constituents include pea gravel, shredded tires, or a mixture of the two. A porosity of 0.3 will be used in calculations for the filter media.

Shredded tires, if being considered, should conform to the same sizing criteria as pea gravel. If shredded tires are the only media, filter depth should not be greater than 3.3 ft (1 m) (Humphrey 1999) or self-heating may be a problem. Guidelines to avoid self-heating were established by an Ad Hoc Civil Engineering Committee of government and industry entities (AHCEC 1997) and published as ASTM D6270-98 (ASTM 1998). These guidelines for avoiding self-heating of scrap tires for depths of 3.3 to 10 ft (1 to 3 m) follow:

- Tire shreds shall be free of contaminants such as oil, grease, gasoline, diesel fuel, etc., that could create a fire hazard
- In no case shall the tire shreds contain the remains of tires that have been subjected to a fire

- Tire shreds shall have a maximum of 25% (by weight) passing 1½-in. sieve
- Tire shreds shall have a maximum of 1% (by weight) passing no. 4 (4.75-mm) sieve
- Tire shreds shall be free from fragments of wood, wood chips, and other fibrous organic matter
- Tire shreds shall have less than 1% (by weight) of metal fragments that are not at least partially encased in rubber
- Metal fragments that are partially encased in rubber shall protrude no more than 1 in. from the cut edge of the tire shred on 75% of the pieces and no more than 2 in. on 100% of the pieces
- Infiltration of water into the tire shred fill shall be minimized (see below)
- Infiltration of air into the tire shred fill shall be minimized
- No direct contact between tire shreds and soil containing organic matter, such as topsoil
- Tire shreds should be separated from the surround soil using a geotextile
- Use of drainage features located at the bottom of the fill that could provide free access to air should be avoided

For the purposes of the horizontal filter trench, water and air will need to infiltrate into the tire media. Self-heating can be avoided by mixing the tire with granular media (Edil et al. 2004) or by keeping the depth below 3.3 ft (1 m). Shredded tires have been shown to avoid self-heating when used in depths less than 3.3 ft (1 m) in several landfill drainage applications (Edil et al. 2004; Humphrey 1999).

Step 4: Select Armoring Specifications. Armoring for the trench should adhere to the Federal Highway Administrations definition of cobbles by having a diameter of 2.5 to 5 in. (6.4 to 13 cm) (Kilgore and Cotton 2005). They are generally alluvial, uniformly graded, and rounded. Armoring depth should be at least 1.75 times the diameter of stone for which 50%, by weight, of gradation is finer (d_{50}) (OES & WWE 2000).

Armoring must be placed over the filter trench and up the side slopes. The armoring should reach 1 ft (0.3 m) above the water surface for the 10-yr, scour check storm (OES & WWE 2000). A geotextile is required between the armoring and both the trench and the adjacent soil. The geotextile facilitates maintenance and prevents mobilization of the underlying media into the cobbles.

Step 5: Calculate Trench Dimensions. The trench must be sized so the WQV can be stored in its pore space. The armoring will store and slow runoff but will not be considered to add directly to the treatment, so pore space in the armoring will not be counted towards the WQV storage.

The required trench size to accommodate the WQV was found with equation 12.22:

$$\frac{\mathrm{WQV}}{\mathrm{p}} = \mathrm{L} * \mathrm{W} * \mathrm{D} \tag{12.22}$$

Where:

L: Trench length (ft or m)

W: Trench width (ft or m)

D: Media depth (ft or m)

p: Media porosity

The available width and length of the trench will be site-specific based on the geometry of the existing drainage ditches, available right-of-way, and existing grade. The depth of media should not be less than 1 ft (0.3 m). Trenches should not be deeper than 5 ft (1.5 m) due to the added costs of a protective system required at that depth (NIOSH 2011). The bottom of the trench should not be within 2 ft (0.61 m) of the seasonal high groundwater table.

Step 6: Verify Armoring Size by Checking Scour Potential. Armoring must be able to withstand scouring effects of the peak flows. Peak scour flow rates were determined by using the 10-yr design storm with a type II NRCS 24 hr distribution and equation 12.23 (NRCS 1986).

$$\mathrm{q_p = q_u A_m Q F_p} \qquad (12.23)$$

Where:

q_p: Peak discharge (cfs)

q_u: Unit peak discharge $\left(\frac{cfs}{mi^2 * in}\right)$ (Figure or Table)

A_m: Drainage area (mi^2)

Q: Runoff corresponding to 24-hr rainfall (in) (Table)

F_p: Pond or swamp adjustment factor (1.0 for Nebraska)

When considering a watershed with both impervious and pervious ground cover, the area can either be considered completely impervious, or a weighted flow may be calculated as described in the Hydrology Section of this work. Assuming total imperviousness would result in larger than actual flows and, therefore, oversized BMPs. For this reason the weighted flow method is recommended.

Once the peak flow is found, the roadside ditch geometry (channel’s bottom width, side slopes, and longitudinal slope) must be determined. Side slopes should be no greater than 3:1 (horizontal:vertical) to avoid slope damage from channelization and to facilitate mowing.

Once the shape of the swale is determined, equation 12.24 (Manning's Equation) can be applied to determine flow depth (NRCS 1986).

$$\mathrm{Q}_{10-\mathrm{yr}} = \frac{\mathrm{k}}{\mathrm{n}}\mathrm{AR}^{2/3}\mathrm{S}^{1/2} \qquad (12.24)$$

Where:

Q_{10-yr}: Flow from10-yr storm (cfs or cms)

S: Slope in direction of flow $\left(\frac{\mathrm{ft}}{\mathrm{ft}} \text{ or } \frac{\mathrm{m}}{\mathrm{m}}\right)$

R: Hydraulic Radius $\left(R = \frac{A}{P_w}\right)$

A: Cross sectional area of flow (ft^2 or m^2)

P_w: Wetted Perimeter (ft or m)

n: Manning's coefficient

k: constant (1 for Metric Units; 1.486 for English Units)

The equations for the elements of trapezoidal cross-sections can be found in

Table **16**.

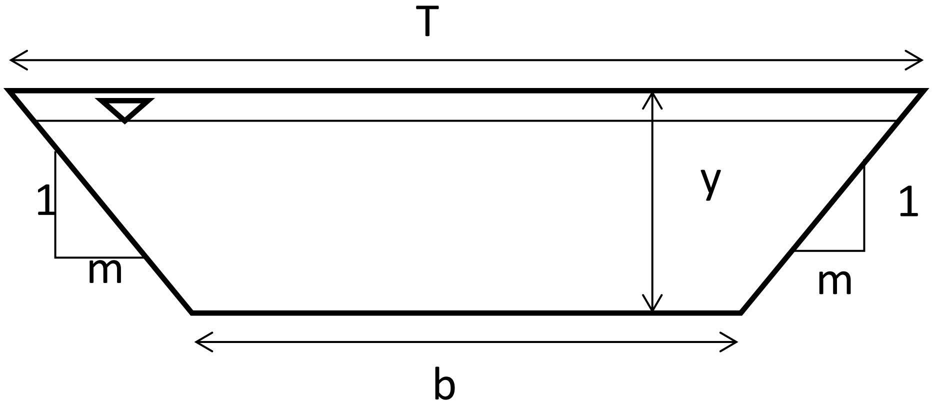

Figure 33 Reference shape for

Table **16**

Table 16 Geometric elements of trapezoidal cross section (Adapted from WSDOT 2010)

Area of flow (A) (ft^2 or m^2)	$(b+my)y$
Wetted perimeter (P_w) (ft or m)	$b+2y\sqrt{1+m^2}$
Hydraulic radius (R) (ft or m)	$\frac{(b+my)y}{b+2y\sqrt{1+m^2}}$

Inserting these geometric elements into the Manning's equation results in equation 12.25, which was then used to solve for the depth of flow (y) by trial and error.

$$Q_p = \left(\frac{k}{n}\right) * (b+my)y * \left[\frac{(b+my)y}{b+2y\sqrt{1+m^2}}\right]^{2/3} * S^{1/2} \qquad (12.25)$$

Manning's coefficient (n) can be calculated for rock-lined channels using equation 12.26 (OES & WWE 2000).

$$n = 0.0395(d_{50})^{1/6} \qquad (12.26)$$

Where:

n: Manning's coefficient

d_{50}: Diameter of stone for which 50%, by weight, of gradation is finer (ft)

The velocity of the flow through the BMP can be determined with equation 12.27 using the peak flow rate and area of flow. The cross-sectional area of flow can be found using

Table 16.

$$v = \frac{Q_p}{A} \tag{12.27}$$

The scour velocity found with equation 12.27 must be less than or equal to $7\,\frac{ft}{s}$ $(2.1\,\frac{m}{s})$ (Caltrans 2003). If the velocity found with equation 12.27 is greater than $7\,\frac{ft}{s}$ $(2.1\,\frac{m}{s})$ corrective action must be taken. Corrective action can consist of resizing the channel, selecting larger cobbles, or incorporating check dams.

Check dams for horizontal filter strips should not be earthen. Earthen check dams could leach fines which would contribute to clogging of the filter media. Rip-rap check dams are best suited for use with horizontal filters. The large void spaces associated with rip-rap check dams are not a problem in this situation as temporary ponding is not essential to the functionality of the trench. The check dams simply act to slow the flows, thereby preventing scour. The Vegetated Swale Design Guide section of this work describes sizing and spacing requirements for check dams.

Step 7: Select Pretreatment. Pretreatment for horizontal filter trenches should be designed to remove solids and, if receiving concentrated flows, act as an energy dissipater. Pretreatment can extend the life of the trench dramatically by preventing clogging and scour.

When retrofitting an existing ditch, vegetation on the side slopes of the ditch can serve as vegetated filter strips. This pretreatment will remove solids but may not adequately attenuate velocities. However, runoff directly from the roadway will likely be in the form of sheet flow and will not require pretreatment for velocity. Vegetated filter strip design considerations can be found in the Vegetated Filter Strip Design Guide section of this work. If the filter trench does not run the entire length of the roadway it is treating, the ditch up stream of the filter trench may also act as pretreatment. The drainage ditch should be designed to the specifications in the Vegetated Swale Design Guide section of this work. If existing vegetation is not dense enough it may require refurbishing.

Shallow forebays at the initial point of the channel can be employed as treatment for solids and as energy dissipaters. Rip-rap forebays are well suited as pretreatment for horizontal filters. The volume of the forebay should be 0.05 in. (0.13 cm) multiplied by the impervious acres of the drainage area (Clar et al. 2004). Rip-rap is suggested as lining for the forebay because it will drain readily and will resist being washed away during times of high flow. Figure 34 shows a properly designed forebay.

Figure 34 Rip-rap forebay (NCDENR 2007)

12.2.5.2 Design Example

A 500 ft section of a 6-lane divided highway is being redeveloped. Figure 35 shows the layout of the 6-lane divided highway. Each direction features 3, 12-ft wide lanes with 6 ft shoulder on each side. The watershed will need to be broken into 3 sub-watersheds to accommodate drainage from each side of the highway. There is also a 25-ft wide median and two

25-ft wide drainage ditches which run the length of the roadway. Horizontal filter trenches will be placed in the bottom of each ditch to treat the runoff, with a third filter trench in the median of roughly double the size. The median and ditches have a 3:1 side slopes with a 7 ft bottom width. The longitudinal slope is 5%. The vegetated areas have a CN of 80. There is no run-on from neighboring properties.

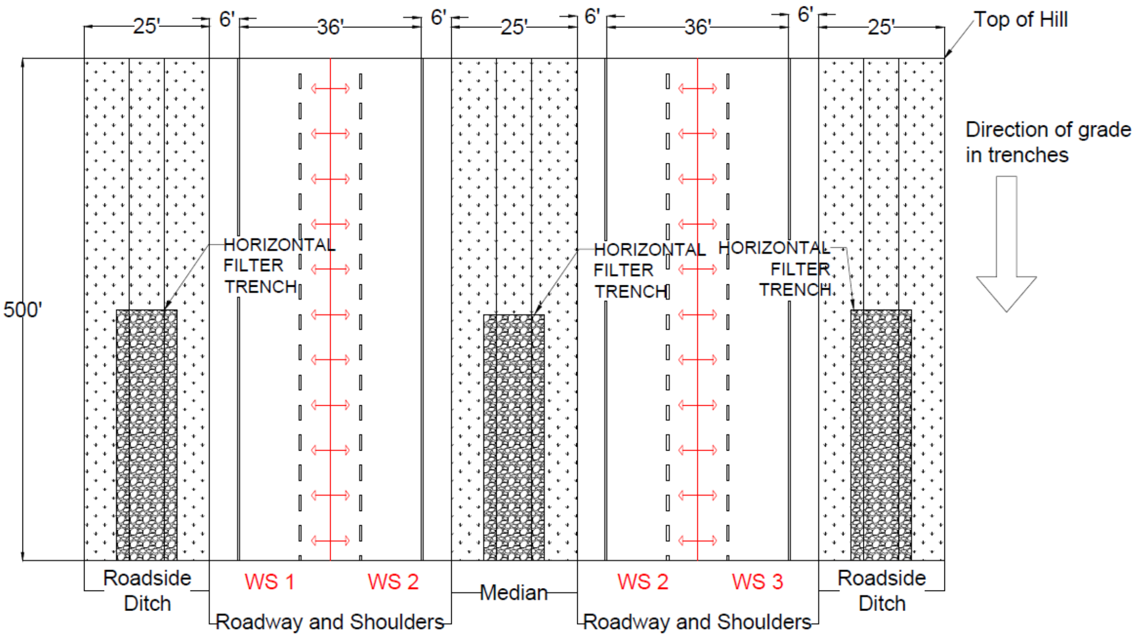

Figure 35 Site plan for horizontal filter example

Step 1: Evaluate Applicable Location Considering Site Constraints. This is an ideal site for horizontal filters due to the existing median and drainage ditches. They are well suited to accommodate the filter trenches.

Step 2: Calculate Water Quality Volume to be Treated (WQV). The WQV for the contributing area for each subwatershed must be calculated. Example calculations for WS 1 will be performed. Equation 12.20 will be used to find the runoff from the newly developed roadway for WS 1:

$$\text{WQV}_{\text{New Dev WS1}} = 0.5\text{in}^{*} \ \frac{(18\text{ ft} + 6\text{ ft}) * 500\text{ ft}}{12\frac{\text{in}}{\text{ft}}} = 500\text{ft}^{3}$$

The volume of run-on for WS 1 (i.e., the runoff from the grassy areas in WS 1) was found with equation 12.21, Q for a CN of 80 was found to be 0.023 in from Table :

$$\text{WQV}_{\text{Run-On}} = 0.023\text{ in}^{*} \ \frac{25\text{ft} * 500\text{ ft}}{12\frac{\text{in}}{\text{ft}}} = 24\text{ft}^{3}$$

The entire ditch and median area were considered for run-on because it was unknown at this stage what the dimensions of the horizontal filter trench were.

The total WQV was then the sum of the runoff from the new development and the run-on volume:

$$500\text{ ft}^{3} + 24\text{ ft}^{3} = 524\text{ ft}^{3}$$

Table **17** shows the calculated WQVs for each subbasin.

Table 17 Subbasin WQVs

Sub Basin	WQV (ft^3)

WS 1	524
WS 2	1024
WS 3	524

Step 3: Select Filter Media Specifications. The filter media will be clean washed pea gravel ranging in size from ${}^{3}/_{8} - {}^{3}/_{4}$ in.

Step 4: Select Armoring Specifications. Cobbles with an average diameter by weight (d_{50}) of 3 in. will be initially selected for design, because it is readily available from a local quarry. If this selection proves to be insufficient at preventing erosion, the design process will revert to this step and select a larger cobble size which prevents scour.

Step 5: Calculate Trench Dimensions. The width of each trench will coincide with the 7 ft bottom width of the drainage ditches. The length and depth of the trench required for the filter media are dependent variables when considering the WQV. The required length and depth for WS 1 was calculated by using equation 12.22. For example, the WQV for WS 1 is 524ft^3which requires a trench volume of 1747 ft^3 assuming a porosity of 0.3.

$$\frac{WQV_{WS1}}{\text{p}} = \text{L} * \text{W} * \text{D} = \frac{524\text{ft}^3}{0.3} = 1747 \text{ ft}^3$$

Several length and depth relationships were checked, the dimensions decided upon are shown in Table 18.

Table 18 Dimensions of filter media in trench

Sub Basin	Width (ft)	Depth (ft)	Length (ft)	Volume

WS 1	7	1	250	1750
WS 2	7	2	245	3430
WS 3	7	1	250	1750

A constant length was selected which required the center sub basin (WS 2) to have a 2 ft depth. Had a constant 1 ft depth been used the WS 2 trench would have been 490 ft. A shorter trench was selected to be more cost effective for WS 2. Figure 36 shows the longitudinal profile of the horizontal filter trench for WS 2, and Figure 37 shows the width cross-section for WS 2.

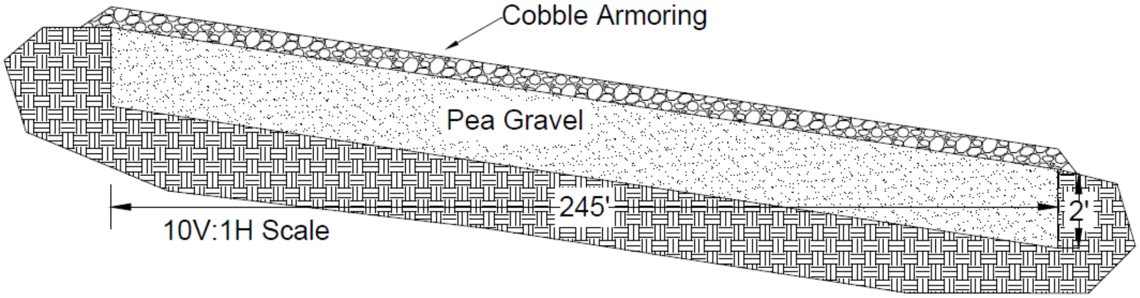

Figure 36 Longitudinal cross-section for example horizontal filter trench in WS 2

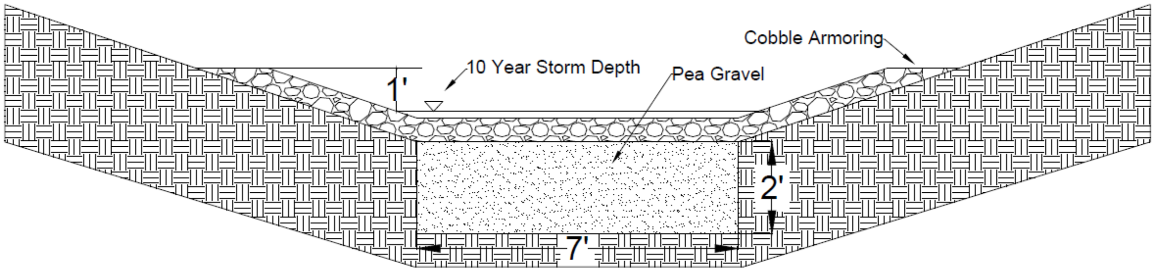

Figure 37 Width cross-section for example horizontal filter trench in WS 2

Step 6: Verify Armoring Size by Checking Scour Potential. Sample calculations will be performed for WS 1. First the peak flows were calculated for the new development and run-on using equation 12.23. Then the flows from the two contributing areas were summed to find the total:

For new development:

$$q_p = q_u A_m Q F_p = 1100 \frac{\text{cfs}}{\text{mi}^2\text{in}} * 0.00043\text{mi}^2 * 4.76\text{in} * 1 = 2.25\text{cfs}$$

For run-on:

$$q_p = q_u A_m Q F_p = 1000 \frac{\text{cfs}}{\text{mi}^2\text{in}} * 0.00045\text{mi}^2 * 2.89\text{in} * 1 = 1.3\text{cfs}$$

Total peak flow:

$$2.25\text{cfs} + 1.3\text{cfs} = 3.55\ \text{cfs}$$

The peak flow (q_p) was then used to determine the flow depth and area, which led to the scour velocity. Unit peak discharge (q_u) was found in Figure , and the runoff depth (Q) was found in Table . The swamp adjustment factor (F_s) for Nebraska is 1.

Equation 12.24 (Manning's Equation) was then used to find the depth of flow, which will be used in flow velocity calculations. The geometric elements of a trapezoid, from

Table 16, were inserted into equation 12.24 transforming it into equation 12.25:

$$Q_p = \left(\frac{k}{n}\right) * (B+my)y * \left[\frac{(B+my)y}{B+2y\sqrt{1+m^2}}\right]^{2/3} * S^{1/2}$$

Equation 12.26 was used to find Manning’s coefficient (n) for rock lined channels:

$$n = 0.0395(d_{50})^{1/6} = 0.0395(0.25)^{1/6} = 0.031$$

$$3.55 \text{ cfs} = \left(\frac{1.468}{0.031}\right) * (7\text{ft}+3y)y * \left[\frac{(7\text{ft}+3y)y}{7 \text{ ft}+2y\sqrt{1+3^2}}\right]^{2/3} * 0.05^{1/2}$$

Flow depth (y) was found by trial and error to be 1.92 in. (0.16 ft). This depth was then used to calculate the area of flow with the equation in

Table **16**:

$$A = (B+my)y = (7ft+3*0.16ft)0.16ft = 1.2ft^2$$

Flow velocity was then found with equation 12.27:

$$v = \frac{Q_p}{A} = \frac{3.55\frac{ft^3}{s}}{1.2ft^2} = 3\frac{ft}{s}$$

The resulting velocity is less than $7\frac{ft}{s}$; therefore, it is satisfactory.

Step 7: Select Pretreatment. Pretreatment for these trenches will be provided by the vegetated slopes. They must be maintained to specifications presented in the Vegetated Filter Strip Fact Sheet portion of this work. If they are not initially to those standards, the slope must be refurbished before installation of the trenches.

Chapter 13 Volume II Conclusions

Several conclusions can be drawn for each BMP which can be used to remediate run-off from highways and protect receiving waters.

- Vegetated filter strips are a viable option for pollutant removal. Existing roadside vegetation may already be acting as a BMP or may be easily retrofit to do so. The length, vegetation density, and slope are the primary design elements affecting performance of vegetated filter strips.
- Vegetated swales have shown to be a viable treatment option as stand-alone BMPs in some cases, as well as within a treatment train. They show high retrofit potential in existing drainage ditches which, when coupled with existing vegetated filter strips, may already be satisfying pollution removal requirements. Check dams may be required to protect vegetated swales from flow velocities which would damage, or limit their functionality.
- Bioretention is a flexible BMP which can add great aesthetic appeal. Bioretention is a very flexible BMP in regards to siting, targeting specific pollutants, vegetation, and infiltration capacity. Maintenance of bioretention facilities is generally higher than other BMPs, particularly early in the life of the BMP when plants are getting established.
- Sand filters have a track record as an effective BMP. Pollutant removal with sand filters has been shown to be very high. Although the initial construction cost of sand filters is substantial, maintenance is not overly burdensome or costly. The major component to the longevity of sand filters is the prevention of fine sediment reaching the filter, which can be done by stabilizing the watershed and incorporating a sedimentation basin.
- Horizontal filter trenches require more research, but their simplicity, applicability for

roadside scenarios, and low maintenance burden suggest they are a strong candidate for remediating roadway runoff. The primary concern for horizontal filter trenches is preventing fine sediment from clogging the system.

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Appendix A Additional Regulations Related to Stormwater Pollution

A.1 The National Environmental Policy Act

The main objective of National Environmental Policy Act (NEPA) was to create a way for considering environmental impacts consistent with other national needs, such as economic development. To accomplish this objective, NEPA established a policy that obligates all Administered Federal Programs to become more environmentally efficient, imposing environmental responsibilities on all agencies of the Federal Government. As part of NEPA, Federal Agencies are required to conduct a preliminary impact analysis in the early planning process and in all cases prior to the undertaking of any project or action.

A.2 The Coastal Zone Management Act

The Coastal Zone Management Act of 1972 (CZMA), was approved by Congress to "preserve, protect, develop, and where possible to restore and enhance the resources of the Nation's coastal zone for this and succeeding generations" (CZMA, 1972). This act encourages states and territories to develop comprehensive programs to protect and manage coastal resources, including the Great Lakes. In 1990, the Coastal Zone Act Reauthorization Amendments (CZARA) mandate to all State Coastal Programs and state nonpoint sources programs (including highway programs) to work in the solution of nonpoint source pollution that may affect coastal water quality.

A.3 The Safe Drinking Water Act

The Safe Drinking Water Act of 1974 (SDWA) and its 1984 amendment has as its main objective the "protection of the Nation's sources of drinking water and the protection of public health to the maximum extent possible, using proper water treatment techniques" (SDWA, 1974). In the SDWA, underground sources of drinking water and aquifers were included to be protected. Therefore, in order to be in compliance with SDWA, highway projects required additional planning and analysis, and possible permitting if the project is located in the recharge area of a drinking water aquifer.

Appendix B Modeling Runoff with HEC-HMS

The HEC-HMS model was used to estimate runoff from the subbasins at the study site. The HEC-HMS incorporates several sub-models to compute runoff. To determine the cumulative losses, HEC-HMS offers the following sub-models: Initial and constant rate loss, deficit and constant-rate, SCS curve number (CN), and Green and Ampt loss. For Unit-Hydrograph (UH) models, HEC-HMS offers the following sub-models: Snyder’s UH model, SCS UH model, Clark’s UH model, ModClark model, and Kinematic-wave model. HEC-HMS also offers sub-models to determine the baseflow; however, in this model baseflow was estimated to be insignificant compared to runoff-event flows.

For this study, the model uses the SCS curve number method to determine the cumulative losses. This model was chosen because it requires a single variable (curve number) that is characterized by data that were available for the site; i.e., the type of soil, soil cover and antecedent moisture. Additionally, as this model was developed for agricultural watersheds in the Midwest, it is applicable for conditions in this study.

The unit-hydrograph model used for this basin model was the SCS UH model. This model was chosen because it requires a single variable (lag time) which could be estimated from the physical characteristics of the site and from the measured hydrographs from the site subbasins. Additionally, some of the parameters required for other models are difficult to determine at a site like the NDOR site because the area is very heterogeneous and does not have open channels with constant characteristics.

The HEC-HMS model requires specification of the area for each subbasin. Areas for each subbasin were determined using drawings of the drainage system, highway vertical alignments, and section profiles of the site. Additionally, aerial photos obtained at the Geographic

Information Systems for Douglas County website (DOGIS, 2009) were used to measure distances and areas inside of the NDOR site.

The model requires specification of the curve number (CN) which was used to determine the losses for each subbasin. The CN was obtained from table 2.2a, 2.2b, 2.2c, and 2.2d contained in the Urban Hydrology for Small Watersheds, Technical Release 55 (U.S. Department of Agriculture, 1986). In addition, HEC-HMS requires specifications of the percentage of impervious area for each subbasin. The percent impervious area for each subbasin was assumed to be 0.0. No impervious area was used because all of the surfaces (including pavement) have some permeability. Table B.1 shows the CN for each subbasin in the HEC-HMS basin model.

Table B.1 Curve numbers for each subbasin

Subbasin	Description	CN
Subbasin-1A	Hard packed driveway in the construction debris lot.	98
Subbasin-1B	Construction material piles and soil in the construction debris lot.	60
Subbasin-2	Area along the fence, interstate shoulder, and some of I-80 Eastbound/D entrance lane.	69
Subbasin-3	Tributary area for the west pipe inlet. This area includes the two outside lanes of the I-80 East bound and interstate shoulder.	99
Subbasin-4	Grass and shoulder area that runoff directly into the detention basin and some pavement from the center lanes.	80
Subbasin-5A	Tributary area for the east pipe inlet. This area includes the two inside lanes of the I-80 Eastbound and the four lanes of the I-80 Westbound.	98
Subbasin-5B	Tributary area for the East pipe inlet. This area includes a segment of the two outside lanes of the I-80 Westbound and the grass area between I-80 Westbound and the Exit 445 ramp.	89
Subbasin-6	Detention basin.	69

As shown in table B.1, a value between 95 and 98 was assigned to the hard packed driveway in the construction debris lot and the highway pavement. The CN for subbasin 1B was more difficult to determine because there are different kinds of material piles with characteristics that differ from one to the other. As it was seen in the field, water running off the material piles was minimal comparing with water coming from the driveway. As a result, a small CN was assigned to this subbasin (representing the material piles) because most of the water infiltrates into the piles.

For subbasin 2 a CN of 69 was assigned, to reflect the low runoff from this subbasin. It was discovered late in the project (fall of 2010) that significant amounts of flows in the ditch were leaking through the berm separating the ditch site from the construction debris lot. Therefore, direct flow measurements (i.e., from the weir at sample site B) were not reliable. Consequently, flows from the ditch/shoulder site were estimated by the HEC-HMS model using typical input values for the properties at that site.

For subbasin 4 and 5A, the CN values correspond to typical values for an area covered with grass. The CN for subbasin 5B is smaller compared to the one for subbasin 4 because the area of subbasin 5B is not covered completely with grass and is apparently more compacted (based on runoff measurements).

For subbasin 6, a CN of 69 was assigned. During the sample period, negligible flow was observed coming from the detention basin floor (i.e., away from the central channel). This is likely because of the small slope in the detention basin, the permeability of the surface, and the low intensity storms.

The SCS unit hydrograph requires the lag time as an input. To calculate the initial lag time values, the time of concentration t_c was used as it was suggested by the NRCS (NRCS, 1986). Equation B.1 shows the relation between lag time and time of concentration.

$$t_{lag} = 0.6t_c \tag{B.1}$$

In the Revised Technical Release (NRCS,1986), the NRCS published that water moves though a watershed sequentially as sheet flow, shallow concentrated flow, open channel flow, or a combination of the three before reaching an inlet of the sewer system (as cited in Gupta, 2001). Time of concentration was calculated as the sum of the travel time for sheet flow, shallow concentrated flow, and open channel flow. Equation B.2 was used to calculate the travel time for sheet flow.

$$t_{sheet} = \frac{0.42(nL)^{0.8}}{(P_2)^{0.5}S^{0.4}} \tag{B.2}$$

Where: t_{sheet} = travel time for sheet flow, s.

n = Manning's roughness coefficient.

L = flow length, ft.

P_2 = 2-yr 24-hr rainfall, in. (3.2 in for Omaha)

S = land slope.

Equation B.3 was used to calculate the travel time for shallow concentrated flow.

$$t_{shallow} = \frac{L}{V} \times \frac{1}{60} \tag{B.3}$$

Where: $t_{shallow}$ = travel time for shallow concentrated flow, s.

L = flow length, ft.

V = average velocity, ft/s

The average velocity can be obtained from standard references such as the Technical Release 55 (NRCS, 1986) and Hydrology and Hydraulic Systems (Gupta, 2008). The time in open channel flow can be calculated using Manning's equation. Equation B.4 shows Manning's equation for open channels.

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad \text{(B.4)}$$

Where: V = mean velocity of flow in an open channel, ft/s.

R = hydraulic radius, ft.

S = slope of energy line. Equal to channel bottom for uniform flow.

n = Manning's roughness coefficient.

The lag times obtained from these equations were the initial values used in the HEC-HMS basin model. Once data from the rain events were obtained, these values were adjusted to make the model match the measured data. Table B.2 shows the lag times used for the different subbasins in the HEC-HMS basin model. Lag times for subbasin 2 and 6 could not be adjusted due to lack of runoff measurements from these sites during the rain events. Therefore, the values used for these subbasins are the lag time obtained using the above equations (SCS method).

In the HEC-HMS basin model, junctions were created as the discharge point from one or more basins. The model included five junctions (see fig. 4.6). Table B.3 shows the junctions with their connections upstream and downstream.

Table B.2 Lag times for each subbasin in the HEC-HMS model

Subbasin	Surface Flow Material/ Condition	Length of Flow Path (ft)	Lag Time (min)
Subbasin-1A	Hard packed driveway	560	8.1
Subbasin-1B	Mixed construction material piles	270	37
Subbasin-2	Grass-fair condition	370	29.5
Subbasin-3	Pavement	550	5
Subbasin-4	Grass-fair condition	260	10.51
Subbasin-5A	Grass-fair condition	600	4.5
Subbasin-5B	Pavement	540	25
Subbasin-6	Grass-poor condition	290	20.7

Table B.3 Junctions and connections in the HEC-HMS model

Junction	Upstream Connections	Downstream Connection
Junction 1	Subbasin 1A Subbasin 1B	Junction 3
Junction 2	Subbasin 3	Reach 2
Junction 3	Subbasin 2 Reach 2	Reach 3
Junction 4	Subbasin 5A Subbasin 5B	Reach 5
Junction 5	Reach 3 Reach 4 Reach 5 Reach 6	None

Junction 1 represented the point where water coming from Site A entered the basin. Junction 2 represented where the west pipe entered the basin. Junction 4 represented the beginning of the channel that conducted water from the west side of the detention basin to the outlet pipe. Junction 5 represented the east pipe. Junction 5 represented the outlet pipe, where water from the detention basin, I-80 runoff, and construction debris lot were collected and discharge into the creek.

Reaches were created to connect subbasins with junctions or to connect one junction with another junction. The routing method used in the reaches was the lag method. This method was used because it required the lag times in the reach, and these had been calculated as described above. Other routing methods (e.g. kinematic wave, Muskingum, Modified puls) were not selected because the reaches had different types of surfaces, and the channels are not well defined. The initial lag times were calculated based on the topography and the distance between the subbasin or junction connected by the reach. Elevations and distances were obtained from DOGIS (DOGIS, 2009). Lag times values were adjusted to match as close as possible the data obtained from the measured hydrographs during rain events.

The model was calibrated using the rain and flow measurements obtained from an ISCO rain gauge and the flow measurement devices installed in each of the sampling sites respectively.

Appendix C Complete List of Analytes

Table C.1 List of inorganics and anions tested at the University of Nebraska-Lincoln Water Sciences Laboratory

Inorganics	Detection Limit (DL)	Method Used	Anions	Detection Limit (DL)	Method Used
Calcium (Ca)	0.1 mg/L	AA Spectrophotometry	Bromide	0.10 mg/L	EPA 300.0
Magnesium (Mg)	0.1 mg/L	AA Spectrophotometry			
Potassium (K)	0.1 mg/L	AA Spectrophotometry			
Sodium (Na)	0.1 mg/L	AA Spectrophotometry	Chloride	0.10 mg/L	EPA 300.0
Cadmium (Cd)	1.0 µg/L	ICPMS 6020A			
Chromium (Cr)	1.0 µg/L	ICPMS 6020A			
Copper (Cu)	0.5 µg/L	ICPMS 6020A	Fluoride	0.10 mg/L	EPA 300.0
Iron (Fe)	1.0 µg/L	ICPMS 6020A			
Lead (Pb)	0.2 µg/L	ICPMS 6020A			
Mercury (Hg)	0.1 µg/L	ICPMS 6020A			
Nickel (Ni)	0.1 µg/L	ICPMS 6020A	Nitrate	0.10 mg/L	EPA 300.0
Zinc (Zn)	0.1 µg/L	ICPMS 6020A			
Silica (SiO_2)	0.20 mg/L	EPA 370.1			
Soluble phosphate	0.02 mg/L	EPA 365.1			
Alkalinity as $CaCO_3$	10.0 mg/L	SM 2320			
BOD – 5 day	0.5 mg/L	SM 5210	Nitrite	0.10 mg/L	EPA 300.0
COD	5 mg/L	SM 5220			
Total phosphorus (TP)	0.02 mgP/L	EPA 365.1			
Total Kjeldahl Nitrogen (TKN)	0.20 mg/L	SM 4500			
Total Solids (TS)	10 mg/L	SM 2540B	Phosphate	0.10 mg/L	EPA 300.0
Total Suspended Solids (TSS)	10 mg/L	SM 2540D			
Total Volatile Solids (TVS)	10 mg/L	SM 2540G			
Volatile Dissolved Solids (VDS)	10 mg/L	SM 2540C	Sulfate	0.10 mg/L	EPA 300.0
Volatile Suspended Solids (VSS)	10 mg/L	SM 2540E			
Oil and Grease	5.0 mg/L	EPA 1664			

Table C.2 List of VOCs and SVOCs tested at the University of Nebraska–Lincoln Water Sciences Laboratory

Volatile Organic Compounds (VOCs)	Detection Limit (DL)	Method Used	Semi-volatile Organic Compounds (SVOCs)	Detection Limit (DL)	Method Used
Benzene	0.05 µg/L	EPA 8260	1,2,4-Trichlorobenzene	0.10 µg/L	EPA 8270
Toluene	0.05 µg/L	EPA 8260	1,2-Dichlorobenzene	0.10 µg/L	EPA 8270
o-Xylene	0.05 µg/L	EPA 8260	1,3-Dichlorobenzene	0.10 µg/L	EPA 8270
m-Xylene + p-Xylene	0.20 µg/L	EPA 8260	1,4-Dichlorobenzene	0.10 µg/L	EPA 8270
Ethylbenzene	0.05 µg/L	EPA 8260	2-Chloronaphthalene	0.10 µg/L	EPA 8270
Isopropylbenzene	0.05 µg/L	EPA 8260	2-Methylnaphthalene	0.10 µg/L	EPA 8270
Propylbenzene	0.05 µg/L	EPA 8260	4-Chloroaniline	0.10 µg/L	EPA 8270
Butylbenzene	0.05 µg/L	EPA 8260	Acenaphthene	0.10 µg/L	EPA 8270
sec-Butylbenzene	0.05 µg/L	EPA 8260	Acenaphthylene	0.10 µg/L	EPA 8270
p-Isopropyltoluene	0.05 µg/L	EPA 8260	Anthracene	0.10 µg/L	EPA 8270
1,2,4-Trimethylbenzene	0.05 µg/L	EPA 8260	Benz[a]anthracene	0.10 µg/L	EPA 8270
Chlorobenzene	0.05 µg/L	EPA 8260	Benzo[a]pyrene	0.10 µg/L	EPA 8270
2-Chlorotoluene	0.05 µg/L	EPA 8260	Benzo[b]fluoranthene	0.10 µg/L	EPA 8270
4-Chlorotoluene	0.05 µg/L	EPA 8260	Benzo[ghi]perylene	0.10 µg/L	EPA 8270
1,2-Dichlorobenzene	0.05 µg/L	EPA 8260	Benzo[k]fluoranthene	0.10 µg/L	EPA 8270
1,3-Dichlorobenzene	0.05 µg/L	EPA 8260	Bis-(2-chloroethoxy) methane	0.10 µg/L	EPA 8270
1,4-Dichlorobenzene	0.05 µg/L	EPA 8260	Carbazole	0.40 µg/L	EPA 8270
1,2,3-Trichlorobenzene	0.05 µg/L	EPA 8260	Chrysene	0.10 µg/L	EPA 8270
1,2,4-Trichlorobenzene	0.05 µg/L	EPA 8260	Dibenz[a,h]anthracene	0.10 µg/L	EPA 8270
Bromobenzene	0.05 µg/L	EPA 8260	Dibenzofuran	0.10 µg/L	EPA 8270
Naphthalene	0.05 µg/L	EPA 8260	Fluoranthene	0.10 µg/L	EPA 8270
Styrene	0.05 µg/L	EPA 8260	Fluorene	0.10 µg/L	EPA 8270
Chloroform	0.05 µg/L	EPA 8260	Hexachloro-1,3-butadiene	0.10 µg/L	EPA 8270
1,1,1,2-Tetrachloroethane	0.05 µg/L	EPA 8260	Hexachlorobenzene	0.10 µg/L	EPA 8270
			Hexachlorocyclopentadiene	0.10 µg/L	EPA 8270
1,2,3-Trichloropropane	0.05 µg/L	EPA 8260	Hexachloroethane	0.10 µg/L	EPA 8270
			Indeno[1,2,3-cd]pyrene	0.10 µg/L	EPA 8270
cis-1,3-Dichloropropene	0.05 µg/L	EPA 8260	Isophorone	0.10 µg/L	EPA 8270
			Naphthalene	0.10 µg/L	EPA 8270
trans-1,3-Dichloropropene	0.05 µg/L	EPA 8260	Nitrobenzene	0.10 µg/L	EPA 8270
Hexachloro-1,3-butadiene	0.05 µg/L	EPA 8260	Phenanthrene	0.10 µg/L	EPA 8270
			Pyrene	0.10 µg/L	EPA 8270

Table C.3 List of inorganics tested at Midwest Laboratories

Inorganics	Detection Limit (DL)	Method Used
Antimony	0.0001 mg/L	EPA 200.8
Arsenic	0.001 mg/L	EPA 200.8
Beryllium	0.0005 mg/L	EPA 200.7
Cadmium (Cd)	0.002 mg/L	EPA 200.7
Calcium	0.01 mg/L	EPA 200.7
Chloride	50 mg/L	SM 4500-CL E
Chromium (Cr)	0.01 mg/L	EPA 200.7
Copper (Cu)	0.01 mg/L	EPA 200.7
Lead (Pb)	0.0005 mg/L	EPA 200.8
Magnesium (Mg)	0.01 mg/L	EPA 200.7
Mercury	0.0004 mg/L	EPA 245.1
Nitrate/Nitrite Nitrogen	0.2 mg/L	EPA 353.2
Nickel	0.01 mg/L	EPA 200.7
Phosphorus (dissolved ortho)	0.05 mg/L	SM 4500-P G
Phosphorus (total)	0.05 mg/L	SM 4500-P H
Sodium (Na)	0.01 mg/L	EPA 200.7
Selenium	0.001 mg/L	EPA 200.8
Silver	0.01 mg/L	EPA 200.7
Thallium	0.0005 mg/L	EPA 200.8
Zinc (Zn)	0.01 mg/L	EPA 200.7
Total Kjeldahl Nitrogen (TKN)	0.50 mg/L	PAI - DK 02
BOD	2 mg/L	SM 5210B
COD	5 mg/L	ASTM D 1252-95-B
Total dissolved solids	10 mg/L	SM 2540C
Total suspended solids	4 mg/L	SM 2540D

Table C.4 List of VOCs and SVOCs tested at Midwest Laboratories

Compound	Detection Limit (DL)	Method Used	Compound	Detection Limit (DL)	Method Used
Benzene	1 µg/L	OA-1	bis(2-Chloroethyl) Ether	10 µg/L	EPA 625
Toluene	1 µg/L	OA-1	1,3-Dichlorobenzene	10 µg/L	EPA 625
Ethylbenzene	1 µg/L	OA-1	1,4-Dichlorobenzene	10 µg/L	EPA 625
Napthalene	1 µg/L	OA-1	1,2-Dichlorobenzene	10 µg/L	EPA 625
Total Xylenes	1 µg/L	OA-1	bis (2-Chloroisopropyl) Ether	10 µg/L	EPA 625
Total Purgeable Hydrocarbons	10 µg/L	OA-1	N-Nitrosodimethylamine	10 µg/L	EPA 625
TEH as Gasoline	50 µg/L	OA-2	N-Nitroso-di-n-propylamine	10 µg/L	EPA 625
TEH as Diesel	50 µg/L	OA-2	Hexachloroethane	10 µg/L	EPA 625
TEH as Waste Oil	50 µg/L	OA-2	Dibenz (a,h) Anthracene	10 µg/L	EPA 625
Isophorone	10 µg/L	EPA 625	1,2-Diphenylhydrazine	10 µg/L	EPA 625
bis (2-Chloroethoxy) Methane	10 µg/L	EPA 625	Hexachlorocyclopentadiene	10 µg/L	EPA 625
1,2,4-Trichlorobenzene	10 µg/L	EPA 625	Phenol	10 µg/L	EPA 625
Naphtalene	10 µg/L	EPA 625	2-Chlorophenol	10 µg/L	EPA 625
Diethyl Phthalate	10 µg/L	EPA 625	2-Nitrophenol	10 µg/L	EPA 625
N-Nitrosodiphenylamine	10 µg/L	EPA 625	2,4-Dichlorophenol	10 µg/L	EPA 625
4-Bromophenyl Phenyl Ether	10 µg/L	EPA 625	2,4-Dimethylphenol	10 µg/L	EPA 625
Hexachlorobenzene	10 µg/L	EPA 625	4-Chloro-3-methylphenol	10 µg/L	EPA 625
3,3'-Dicholorobenzidine	20 µg/L	EPA 625	2,4,6-Trichlorophenol	10 µg/L	EPA 625
Chrysene	10 µg/L	EPA 625	Benzo (g,h,i) Perylene	10 µg/L	EPA 625
Benzo (a) Anthracene	10 µg/L	EPA 625	Hexachlorobutadiene	10 µg/L	EPA 625
Benzo (k) Fluoranthene	10 µg/L	EPA 625	2-Chloronapthalene	10 µg/L	EPA 625
Indeno (1,2,3,-cd) Pyrene	10 µg/L	EPA 625	Dimethyl Phtalate	10 µg/L	EPA 625
Benzidine	50 µg/L	EPA 625	Acenaphthylene	10 µg/L	EPA 625
Acenaphthene	10 µg/L	EPA 625	2,6-Dinitrotoluene	10 µg/L	EPA 625
2,4-Dinitrotoluene	10 µg/L	EPA 625	2,4-Dinitrophenol	50 µg/L	EPA 625
Benzo (b) Fluoranthene	10 µg/L	EPA 625	4-Nitrophenol	10 µg/L	EPA 625
Benzo (a) Pyrene	10 µg/L	EPA 625	4,6-Dinitro-2-methylphenol	25 µg/L	EPA 625
4-Chlorophenyl Phenyl Ether	10 µg/L	EPA 625	Pentacholorophenol	10 µg/L	EPA 625
Fluorene	10 µg/L	EPA 625	Di-n-octyl Phthalate	10 µg/L	EPA 625
Nitrobenzene	10 µg/L	EPA 625	n-Hexane	1 µg/L	EPA 625
Butyl Benzyl Phthalate	10 µg/L	EPA 625	Methyl t-Butyl Ether	1 µg/L	EPA 625
Bis (2-ethylhexyl) Phthalate	10 µg/L	EPA 625	Anthracene	10 µg/L	EPA 625
Pyrene	10 µg/L	EPA 625	Di-n-butyl Phthalate	10 µg/L	EPA 625
Phenanthrene	10 µg/L	EPA 625	Fluoranthene	10 µg/L	EPA 625

Appendix D Results from Each Sampled Site

Table D.1 Concentrations for Site A

Analyte[1]	**Unit**	**Rainfall Event**											
		11/10/2008		**5/12/2009**		**7/3/2009**		**7/31/2009**		**9/3/2009**		**3/27/2010**	
		First Flush	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**
Antimony (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.0008	0.0008
Arsenic (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.005	0.005
Beryllium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.0005	<0.0005
Cadmium (dissolved)	(mg/L)	0.0032	0.0038	<0.001	<0.001	<0.001	<0.001	0.0013	<0.001	<0.001	<0.001	<0.002	<0.002
Cadmium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.002	<0.002
Calcium (dissolved)	(mg/L)	123	50.7	8.5	6	13.6	13.6	256	202	58.6	8.4	N/A	N/A
Calcium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	59.6	59.6
Chromium (dissolved)	(mg/L)	<0.001	0.0032	0.0161	0.0121	0.002	0.0026	0.0472	0.0165	0.0064	0.0042	<0.01	<0.01
Chromium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.01	<0.01
Copper (dissolved)	(mg/L)	0.0096	0.013	0.0119	0.0085	0.0074	0.0313	0.0221	0.0055	0.0038	0.0027	<0.01	<0.01
Copper (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.01	<0.01
Iron (dissolved)	(mg/L)	0.016	0.122	0.004	<0.001	0.0015	<0.001	0.00298	0.002134	0.0178	0.0023	N/A	N/A
Lead (dissolved)	(mg/L)	0.0023	0.0053	0.0004	0.0004	0.0002	0.0002	0.0211	0.0203	0.0003	0.0002	0.0011865	0.001187
Lead (total)	(mg/L)	-	-	-	-	-	-	-	-	-	-	0.0015	0.0015
Magnesium (dissolved)	(mg/L)	67.2	17.2	7.9	6.3	2.5	3.5	22.2	19.6	7.2	3.7	N/A	N/A
Magnesium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	37.3	37.3
Mercury (dissolved)	(mg/L)	N/A	N/A	0.0027	0.0027	0.0538	0.0102	0.007	0.0052	0.0026	0.0025	N/A	N/A
Mercury (total)	(mg/L)	N/A	N/A	-	-	-	-	-	-	-	-	<0.0004	<0.0004
Nickel (dissolved)	(mg/L)	N/A	N/A	0.0076	0.0044	0.0022	0.0013	0.0442	0.0117	0.0016	0.001	<0.01	<0.01
Nickel (total)	(mg/L)	N/A	N/A	-	-	-	-	-	-	-	-	<0.01	<0.01
Potassium (dissolved)	(mg/L)	25.4	20.5	0.4	43	6.3	12.1	91.4	85.4	40	21.3	N/A	N/A
Selenium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.002	0.002
Silver (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.01	<0.01
Sodium (dissolved)	(mg/L)	301	183	16.2	353	67.3	100	561	491	252	140	N/A	N/A
Sodium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	292	292
Thallium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.0005	<0.0005
Zinc (dissolved)	(mg/L)	0.0018	0.016	<0.0001	<0.0001	<0.0001	<0.0001	0.0852	0.0439	<0.0001	<0.0001		
Zinc (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.01	<0.01
Silica	(mg/L)	30.9	21.4	7.57	8.2	14	8.3	23.8	28.5	N/A	N/A	N/A	N/A
Bromide	(mg/L)	<0.10	<0.10	0.1	0.1	0.1	0.1	0.31	0.54	0.1	0.1	N/A	N/A

N/A: No data available.

Table D.1 Concentrations for Site A (cont.)

Analyte	Unit	Rainfall Event											
		11/10/2008		5/12/2009		7/3/2009		7/31/2009		9/3/2009		3/27/2010	
		First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC
Chloride	(mg/L)	438.1	456.7	675.2	700.6	111.8	120.7	386.2	634.2	255	150.8	239	239
Fluoride	(mg/L)	<0.10	<0.10	0.25	0.31	0.13	0.15	0.1	0.12	0.1	0.1	N/A	N/A
Nitrate	(mg/L)	<0.10	0.82	1.57	1.79	0.24	0.52	1.37	2.67	1.21	1.36	N/A	N/A
Nitrite	(mg/L)	<0.10	<0.10	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	N/A	N/A
Nitrate/Nitrite Nitrogen	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.2	<0.2
Phosphate	(mg/L)	<0.10	<0.10	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	N/A	N/A
Sulfate	(mg/L)	158.4	170.2	274.6	273.8	82.2	72.1	98.3	201	101.2	58.3	N/A	N/A
Soluble Phosphate	(mg P/L)	<0.02	<0.02	0.02	0.02	0.02	0.03	0.02	0.415	0.706	0.02	N/A	N/A
Phosphorus (dissolved ortho)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.05	<0.05
Total Phosphorus	(mg N/L)	0.08	0.13	<0.02	<0.02	0.03	0.06	<0.02	0.067	0.021	<0.020	0.1	0.1
Total Kjeldahl Nitrogen	(mg/L)	N/A	N/A	2.79	3.18	1.89	1.37	8.71	5.32	1.34	<0.020	1.42	1.42
Total Dissolved Solids	(mg/L)	N/A	N/A	1,510	1,998	1,406	210	1,830	1,380	758	468	1178	1178
Total Suspended Solids	(mg/L)	N/A	N/A	732	638	1,994	380	1,758	1,352	228	196	35	35
Total Solids	(mg/L)	2506	1422	2,242	2,636	3,400	590	3,588	2,732	986	664	N/A	N/A
Volatile Dissolved Solids	(mg/L)	244	213	50	110	12	16	298	396	196	42	N/A	N/A
Volatile Suspended Solids	(mg/L)	136	51	174	156	342	64	40	54	30	22	N/A	N/A
Total Volatile Solids	(mg/L)	380	264	224	266	354	80	338	450	226	64	N/A	N/A
Alkalinity as $CaCO_3$	(mg/L)	241.9	177.6	146.1	146.3	95.5	51.5	137.1	142.2	90.2	64.8	N/A	N/A
Oil and Grease	(mg/L)	N/A	N/A	25.9	28.5	<5.0	<5.0	<5.0	<5.0	<5.0	<5.0	N/A	N/A
TEH as Diesel	(µg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	340	340
BOD	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	5	5
COD	(mg/L)	126.7	83.6	107.8	90.6	55.7	31.2	94.6	79.6	76	5	189	189
n-Hexane	(µg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<1	<1
Methyl t-Butyl Ether	(µg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<1	<1
Benzene	(µg/L)	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<1	<1
Toluene	(µg/L)	<0.05	<0.05	<0.05	0.08	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<1	<1
Ethylbenzene	(µg/L)	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<1	<1
Napthalene	(µg/L)	0.11	0.11	0.11	<0.05	0.09	<0.05	0.09	<0.05	0.17	0.09	<1	<1

N/A: No data available.

Table D.1 Concentrations for Site A (cont.)

Analyte	Unit	Rainfall Event							
		5/7/2010		5/20/2010		7/4/2010		9/13/2010	
		First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC
Antimony (total)	(mg/L)	0.0006	0.0008	First flush sample could not be collected because a previous low intensity showers wash off the pollutants from the highway	0.0014	<0.0010	<0.0010	0.0015	0.0015
Arsenic (total)	(mg/L)	0.006	0.006		0.008	0.008	0.005	0.013	0.013
Beryllium (total)	(mg/L)	<0.0005	<0.0005		<0.0005	0.001	0.001	0.001	0.001
Cadmium (dissolved)	(mg/L)	<0.002	<0.002		<0.002	<0.002	<0.002	<0.002	<0.002
Cadmium (total)	(mg/L)	<0.002	<0.002		<0.002	<0.002	<0.002	<0.002	<0.002
Calcium (dissolved)	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Calcium (total)	(mg/L)	83.7	65.5		N/A	53.4	33.8	54.30	54.30
Chromium (dissolved)	(mg/L)	<0.01	0.00982		0.02946	0.02946	0.01964	0.02	0.02
Chromium (total)	(mg/L)	<0.01	0.01		0.03	0.03	0.02	0.02	0.02
Copper (dissolved)	(mg/L)	<0.01	<0.01		0.0096	0.0192	0.0096	0.019	0.019
Copper (total)	(mg/L)	<0.01	<0.01		0.01	0.02	0.01	0.02	0.02
Iron (dissolved)	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Lead (dissolved)	(mg/L)	0.0041923	0.00356		0.006486	0.0092547	0.008147	0.0058	0.0058
Lead (total)	(mg/L)	0.0053	0.0045		0.0082	0.0117	0.0103	0.0073	0.0073
Magnesium (dissolved)	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Magnesium (total)	(mg/L)	36.9	29.5		15.1	7.26	4.99	19	19
Mercury (dissolved)	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Mercury (total)	(mg/L)	<0.0004	<0.0004		<0.0004	<0.0004	<0.0004	<0.0004	<0.0004
Nickel (dissolved)	(mg/L)	<0.01	<0.01		0.00998	0.00998	0.00998	0.01	0.01
Nickel (total)	(mg/L)	<0.01	<0.01		0.01	0.01	0.01	0.01	0.01
Potassium (dissolved)	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Selenium (total)	(mg/L)	0.001	<0.001		0.001	0.002	0.001	0.002	0.002
Silver (total)	(mg/L)	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01
Sodium (dissolved)	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Sodium (total)	(mg/L)	278	353		298	205	73.2	439	439.00
Thallium (total)	(mg/L)	<0.0005	<0.0005		<0.0005	<0.0005	<0.0005	<0.0005	<0.0005
Zinc (dissolved)	(mg/L)	0.02	0.02		0.029	0.029	0.029	0.039	0.039
Zinc (total)	(mg/L)	0.02	0.02		0.03	0.03	0.03	0.04	0.04
Silica	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Bromide	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Chloride	(mg/L)	227	328		374	190	62	403	403
Fluoride	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Nitrate	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Nitrite	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A

N/A: No data available.

Table D.1 Concentrations for Site A (cont.)

Analyte	Unit	Rainfall Event							
		5/7/2010		5/20/2010		7/4/2010		9/13/2010	
		First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC
Nitrate/Nitrite Nitrogen	(mg/L)	<0.2	1	First flush sample could not be collected because a previous low intensity showers wash off the pollutants from the highway	1.2	1.3	0.5	1.6	1.6
Phosphate	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Sulfate	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Soluble Phosphate	(mg P/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Phosphorus (dissolved ortho)	(mg/L)	<0.05	<0.05		<0.05	<0.05	0.06	<0.05	<0.05
Total Phosphorus	(mg N/L)	0.1	0.13		0.16	0.32	0.28	0.46	0.46
Total Kjeldahl Nitrogen	(mg/L)	1.58	1.67		1.75	2.43	1.55	4.81	4.81
Total Dissolved Solids	(mg/L)	1246	1398		1246	668	228	1554	1554
Total Suspended Solids	(mg/L)	114	142		157	377	251	226	226
Total Solids	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Volatile Dissolved Solids	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Volatile Suspended Solids	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Total Volatile Solids	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Alkalinity as $CaCO_3$	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Oil and Grease	(mg/L)	N/A	N/A		N/A	N/A	N/A	N/A	N/A
TEH as Diesel	(µg/L)	130	120		<50	155	50	462	462
BOD	(mg/L)	7	9		8	13	8	22	22
COD	(mg/L)	146	244		107	80	46	302	302
n-Hexane	(µg/L)	<1	<1		<1	<2	1	<1	<1
Methyl t-Butyl Ether	(µg/L)	<1	<1		<1	<1	<1	<1	<1
Benzene	(µg/L)	<1	<1		<1	<1	<1	<1	<1
Toluene	(µg/L)	<1	<1		<1	<1	<1	<1	<1
Ethylbenzene	(µg/L)	<1	<1		<1	<1	<1	<1	<1
Napthalene	(µg/L)	<1	<1		<1	<1	<1	<10	<10

N/A: No data available.

Table D.2 Concentrations for Site B

Analyte	Unit	Rainfall Event											
		4/15/2011		4/19/2011		5/12/2011		5/25/2011		6/25/2011		8/22/2011	
		First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC
Antimony (total)	(mg/L)	0.001	0.0007	N/A	0.0012	N/A	0.0008	N/A	0.0005	N/A	<0.0005	N/A	<0.0005
Arsenic	(mg/L)	0.002	0.003	N/A	0.003	N/A	0.004	N/A	0.003	N/A	0.002	N/A	0.003
Copper	(mg/L)	0.01	0.01	N/A	0.01	N/A	0.001	N/A	<0.01	N/A	0.01	N/A	0.01
Lead	(mg/L)	0.0032	0.027	N/A	0.0041	N/A	0.0029	N/A	0.0014	N/A	0.0033	N/A	0.003
Zinc	(mg/L)	0.06	0.04	N/A	0.04	N/A	0.03	N/A	0.2	N/A	0.02	N/A	0.04
Phosphorus (dissolved ortho)	(mg/L)	0.93	0.28	N/A	0.19	N/A	0.29	N/A	0.21	N/A	< 0.0005	N/A	0.32
Phosphorus (Total)	(mg/L)	N/A	0.34	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.12	N/A	N/A
Nitrate/Nitrite Nitrogen	(mg/L)	0.7	0.4	N/A	0.3	N/A	0.2	N/A	0.2	N/A	N/A	N/A	1
Total Kjeldahl Nitrogen	(mg/L)	2.75	1.22	N/A	1.04	N/A	1.97	N/A	2.26	N/A	0.5	N/A	2.4
Magnesium(Total)	(mg/L)	2.09	3.1	N/A	2.62	N/A	2.38	N/A	3.22	N/A	1.27	N/A	2.55
Bichemical Oxygen Demand	(mg/L)	N.D	5	N/A	5	N/A	8	N/A	8	N/A	2.24	N/A	8
Total Suspended Solids	(mg/L)	31	20	N/A	38	N/A	53	N/A	11	N/A	4	N/A	61
Calcium (Total)	(mg/L)	11.4	24.4	N/A	15.5	N/A	18.5	N/A	38.6	N/A	46	N/A	14.2
Chemical Oxygen Demand	(mg/L)	68	49	N/A	39	N/A	69	N/A	76	N/A	12.9	N/A	63
Sodium (total)	(mg/L)	54	136	N/A	94.5	N/A	74.4	N/A	105	N/A	46	N/A	30.3
Chloride	(mg/L)	38	186	N/A	129	N/A	66	N/A	147	N/A	39.5	N/A	18
Total Dissolved Solids	(mg/L)	288	536	N/A	448	N/A	344	N/A	524	N/A	29	N/A	236
Silver	(mg/L)	<0.01	< 0.01	N/A	< 0.01	N/A	< 0.01	N/A	< 0.01	N/A	276	N/A	< 0.01
Selenium	(mg/L)	<0.001	< 0.001	N/A	< 0.001	N/A	< 0.001	N/A	< 0.001	N/A	<0.01	N/A	< 0.001
Mercury	(mg/L)	<0.0004	< 0.0004	N/A	< 0.0004	N/A	< 0.0004	N/A	< 0.0004	N/A	<0.0004	N/A	< 0.0004
Thallium	(mg/L)	< 0.0005	< 0.0005	N/A	< 0.0005	N/A	< 0.0005	N/A	< 0.0005	N/A	0.0033	N/A	< 0.0005
Nickel	(mg/L)	< 0.01	< 0.01	N/A	< 0.01	N/A	< 0.01	N/A	< 0.01	N/A	< 0.005	N/A	< 0.01
2-ChloroN/Apthalene	(µg/L)	< 10	< 10	N/A	< 10	N/A	< 10	N/A	< 10	0.01	N/A	N/A	< 10
Beryllium	(mg/L)	<0.0005	<0.0005	N/A	0.001	N/A	<0.0005	N/A	<0.0005	N/A	N/A	N/A	<0.0005
Cadmium	(mg/L)	<0.002	<0.002	N/A	<0.002	N/A	<0.002	N/A	<0.002	N/A	0.001	N/A	<0.002
Phenol	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<0.002	N/A	<10

N/A: No data available.

Table D.2 Concentrations for Site B (cont.)

Analyte	Unit	Rainfall Event											
		4/15/2011		4/19/2011		5/12/2011		5/25/2011		6/25/2011		8/22/2011	
		First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC
bis(2-Chloroethyl) Ether	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
2-Chlorophenol	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
1,3-Dichlorobenzene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
1,4-Dichlorobenzene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
1,2-Dichlorobenzene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
2 - Methylphenol	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
bis(2-Chloroisopropyl) Ether	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
4 - Methylephenol	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
N-Nitroso-di-n-propylamine	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Hexachloroethane	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Nitrobenzene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Isophorone	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
2-Nitrophenol	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
2,4-Dichlorophenol	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
bis (2- Chloroethoxy) Methane	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
2,4 - Dichlorophenol	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
1,2,4-Trichlorobenzene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
N/Apthalene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
4- Chloroaniline	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Hexachlorobutadiene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
4-Chloro-3-methylphenol	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
2- methylN/Aphthalene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Dibenzofarun	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
2,4-Dinitrotoluene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Anthracene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10

N.A: No data available

Table D.2 Concentrations for Site B (cont.)

Analyte	Unit	Rainfall Event											
		4/15/2011		4/19/2011		5/12/2011		5/25/2011		6/25/2011		8/22/2011	
		First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC
Di-n-butyl Phthalate	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Fluoranthene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Pyrene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Butyl Benzyl Phthalate	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
3,3'-Dicholorobenzidine	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Benzo (a) Anthracene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Hexachlorocyclopentadiene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	< 25	N/A	<10
2,4,6 -Trichlorophenol	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	< 10	N/A	<10
Dimethyl Phtalate	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	< 10	N/A	<10
AceN/Aphthylene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
AceN/Aphthene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Bis (2-ethylhexyl) Phthalate	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Chrysene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Di -n-octyl Phthalate	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Benzo (b) Fluoranthene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Benzo (k) Fluoranthene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Benzo (a) Pyrene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Indeno (1,2,3,-cd) Pyrene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Dibenz (a,h) Anthracene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Benzo (g,h,i)Perylene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Total Purgeable Hydrocarbons	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<25	N/A	<10
2,4-Dinitrophenol	(µg/L)	<25	<25	N/A	<25	N/A	<25	N/A	<25	N/A	<25	N/A	<25
4-Nitrophenol	(µg/L)	<25	<25	N/A	<25	N/A	<25	N/A	<25	N/A	<25	N/A	<25
4- Nitroaniline	(µg/L)	<25	<25	N/A	<25	N/A	<25	N/A	<25	N/A	<10	N/A	<25
Hexachlorocyclopentadiene	(µg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<10	N/A	N/A

N.A: No data available

Table D.2 Concentrations for Site B (cont.)

Analyte	**Unit**	**Rainfall Event**											
		4/15/2011		**4/19/2011**		**5/12/2011**		**5/25/2011**		**6/2/2011**		**8/22/2011**	
		First Flush	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**
Chromium	(mg/L)	<0.01	<0.01	N/A	0.01	N/A	<0.01	N/A	<0.01	N/A	<0.01	N/A	<0.01
n-Hexane	(µg/L)	<1	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1
Methyl t-Butyl Ether	(µg/L)	<1	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1
Benzene	(µg/L)	<1	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1
Toluene	(µg/L)	<1	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1
Ethylbenzene	(µg/L)	<1	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1
N/Aphtalene	(µg/L)	<1	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1
Total Xylenes	(µg/L)	<1	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1	N/A	<1
2,6-Dinitrotoluene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Diethyl Phthalate	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
4-Chlorophenyl Phenyl Ether	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Fluorene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
N-Nitrosodiphenylamine	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
4 - Bromophenyl Phenyl Ether	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Hexachlorobenzene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
PheN/Anthrene	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
Crabazole	(µg/L)	<10	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10	N/A	<10
4,6 Dintro -2- methylphnol	(µg/L)	<25	<25	N/A	<25	N/A	<25	N/A	<25	N/A	<25	N/A	<25
Pentacholorophenol	(µg/L)	<25	<25	N/A	<25	N/A	<25	N/A	<25	N/A	<25	N/A	<25
2,4,5 -Trichlorophenol	(µg/L)	<25	<25	N/A	<25	N/A	<25	N/A	<25	N/A	< 25	N/A	<25
2- Nitroaniline	(µg/L)	<25	<25	N/A	<25	N/A	<25	N/A	<25	N/A	<25	N/A	<25
3 - Nitroaniline	(µg/L)	<25	<25	N/A	<25	N/A	<25	N/A	<25	N/A	< 25	N/A	<25
TEH as Gasoline	(µg/L)	<50	<50	N/A	<50	N/A	<50	N/A	<50	146	N/A	N/A	<50
TEH as Waste Oil	(µg/L)	<50	<50	N/A	<50	N/A	<50	N/A	<50	N/A	< 50	N/A	<50
TEH as Diesel	(µg/L)	172	135	N/A	175	N/A	64	N/A	170	N/A	< 50	N/A	<50
1,2-Diphenylhydrazine	(µg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

N.A: No data available

Table D.3 Concentrations for Site C

Analyte	**Unit**	**Rainfall Event**											
		11/10/2008		**5/12/2009**		**6/1/2009**		**7/3/2009**		**7/31/2009**		**9/3/2009**	
		First Flush	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**
Antimony (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Arsenic (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Beryllium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Cadmium (dissolved)	(mg/L)	0.0033	0.0018	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001
Cadmium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Calcium (dissolved)	(mg/L)	31.8	24.9	17.2	15.8	21.400	21.400	37.9	8.2	21.7	15.4	16.7	11.3
Calcium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Chromium (dissolved)	(mg/L)	0.0022	0.0017	0.0048	0.003	0.00260	0.00260	0.014	0.0031	0.0052	0.0058	0.0011	0.0013
Chromium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Copper (dissolved)	(mg/L)	0.014	0.024	0.0196	0.0142	0.005	0.005	0.0312	0.005	0.0183	0.014	0.0059	0.0053
Copper (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Iron (dissolved)	(mg/L)	0.116	0.121	0.0044	0.0028	0.1250	0.1250	0.026	<0.001	1.322	0.0795	0.0073	0.0057
Lead (dissolved)	(mg/L)	0.005	0.0024	0.0003	0.0003	0.0004	0.0004	0.0003	0.0002	0.00108	0.006	0.0003	0.0003
Lead (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Magnesium (dissolved)	(mg/L)	3.6	2.9	0.8	0.9	1.500	1.500	2.1	0.3	0.8	0.3	0.8	0.4
Magnesium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Mercury (dissolved)	(mg/L)	N/A	N/A	0.0026	0.0025	0.003900	0.003900	0.0063	0.0055	0.0045	0.0042	0.0024	0.0023
Mercury (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Nickel (dissolved)	(mg/L)	N/A	N/A	0.0025	0.0014	0.006000	0.006000	0.0056	0.0006	0.0089	0.009	0.001	0.0007
Nickel (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Potassium (dissolved)	(mg/L)	5.9	5.3	2.5	2.4	6.5	6.5	7.9	0.8	<0.1	0.2	2.5	1.4
Selenium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Silver (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Sodium (dissolved)	(mg/L)	50.5	36.1	81.7	87.7	61.8	61.8	85.9	17.1	15	18.4	22.9	12.6
Sodium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Thallium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Zinc (dissolved)	(mg/L)	0.016	0.021	<0.0001	<0.0001	0.0045	0.0045	0.0163	<0.0001	0.168	0.162	0.003	<0.0001
Zinc (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Silica	(mg/L)	7.4	6.9	2.51	3.44	1.280	1.280	8.3	1.8	1.6	2.6	N/A	N/A
Bromide	(mg/L)	<0.10	<0.10	< 0.10	< 0.10	< 0.10	< 0.10	< 0.10	< 0.10	<0.10	<0.10	0.1	0.1
Chloride	(mg/L)	125.1	48.4	112.2	111.6	93.000	93.000	134.9	10.2	17	21	255	150.8
Fluoride	(mg/L)	0.58	0.66	0.51	0.51	0.230	0.230	0.99	0.23	0.35	0.32	0.1	0.1
Nitrate	(mg/L)	<0.10	<0.10	0.6	0.69	<0.10	<0.10	<0.10	0.36	1.19	1.11	1.21	1.36
Nitrite	(mg/L)	<0.10	<0.10	0	0	0.000	0.000	3.3	0	0	0	0.1	0.1

N/A: No data available.

Table D.3 Concentrations for Site C (cont.)

Analyte	**Unit**	**Rainfall Event**											
		11/10/2008		**5/12/2009**		**6/1/2009**		**7/3/2009**		**7/31/2009**		**9/3/2009**	
		First Flush	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**
Nitrate/Nitrite Nitrogen	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Phosphate	(mg/L)	<0.10	<0.10	0.45	0.3	0.830	0.830	1	0.35	0.41	0.38	0.49	0.3
Sulfate	(mg/L)	57.1	20.2	30.2	41.7	84.100	84.100	82.9	7.71	13.1	16.6	17.2	6.52
Soluble Phosphate	(mg P/L)	0.18	0.13	0.07	0.11	0.170	0.170	0.27	0.07	0.093	0.811	0.116	0.087
Phosphorus (dissolved ortho)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Total Phosphorus	(mg N/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Total Kjeldahl Nitrogen	(mg/L)	N/A	N/A	1.73	1.64	9.040	9.040	6.6	0.71	2.41	1.6	7.54	1.8
Total Dissolved Solids	(mg/L)	1.01	89	236	272	106.000	106.000	172	78	86	90	64	122
Total Suspended Solids	(mg/L)	N/A	N/A	198	116	1040.000	1040.000	380	124	246	116	552	118
Total Solids	(mg/L)	744	92	434	388	1146.000	1146.000	552	202	332	206	616	240
Volatile Dissolved Solids	(mg/L)	101	89	<10.0	<10.0	<10.0	<10.0	66	24	32	22	134	16
Volatile Suspended Solids	(mg/L)	167	51	104	54	324.000	324.000	40	34	32	30	38	114
Total Volatile Solids	(mg/L)	268	140	76	56	260.000	260.000	106	58	64	52	172	130
Alkalinity as $CaCO_3$	(mg/L)	131.7	63.1	71	68.2	84.800	84.800	106	25.1	46	39.8	40.5	27.4
Oil and Grease	(mg/L)	N/A	N/A	6.8	<5.0	14	14	<5.0	<5.0	<5.0	<5.0	<5.0	<5.0
TEH as Diesel	(µg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
BOD	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
COD	(mg/L)	101.1	35.4	5.6	23.5	80.7	80.7	84	88.4	14	14.6	<5.0	<5.0
n-Hexane	(µg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Methyl t-Butyl Ether	(µg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Benzene	(µg/L)	<0.05	<0.05	<0.05	<0.05	0.12	0.12	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Toluene	(µg/L)	<0.05	<0.05	<0.05	<0.05	< 0.05	< 0.05	<0.05	<0.05	<0.05	0.06	<0.05	<0.05
Ethylbenzene	(µg/L)	<0.05	<0.05	<0.05	<0.05	< 0.05	< 0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Napthalene	(µg/L)	0.11	0.11	<0.05	<0.05	0.08	0.08	<0.05	<0.05	<0.05	<0.05	0.08	0.06

N/A: No data available.

Table D.3 Concentrations for Site C (cont.)

Analyte	**Unit**	**Rainfall Event**									
		3/27/2010		**5/7/2010**		**5/20/2010**		**7/4/2010**		**9/13/2010**	
		First Flush	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**
Antimony (total)	(mg/L)	0.0108	0.0098	0.0031	0.0018	First flush sample could not be collected because a previous low intensity showers wash off the pollutants from the highway	0.0037	0.0019	<0.0010	0.0027	0.0027
Arsenic (total)	(mg/L)	0.011	0.008	0.002	0.002		<0.001	0.002	<0.001	<0.001	<0.001
Beryllium (total)	(mg/L)	0.001	0.001	<0.0005	<0.0005		<0.0005	<0.0005	<0.0005	<0.005	<0.005
Cadmium (dissolved)	(mg/L)	0.0038	0.0028	N/A	N/A		N/A	N/A	N/A	<0.002	<0.002
Cadmium (total)	(mg/L)	0.004	0.003	<0.002	<0.002		<0.002	<0.002	<0.002	<0.002	<0.002
Calcium (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Calcium (total)	(mg/L)	153	97.2	56.6	186		18.2	37.6	8.18	16.20	16.20
Chromium (dissolved)	(mg/L)	0.08838	0.05892	<0.01	0.00982		0.02946	0.02946	0.01964	<0.01	<0.01
Chromium (total)	(mg/L)	0.09	0.06	0.03	0.03		0.02	0.02	<0.01	<0.01	<0.01
Copper (dissolved)	(mg/L)	0.1824	0.1152	<0.01	<0.01		0.0096	0.0192	0.0096	0.019	0.019
Copper (total)	(mg/L)	0.19	0.12	0.05	0.03		0.03	0.06	0.01	0.02	0.02
Iron (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Lead (dissolved)	(mg/L)	0.0667604	0.047144	0.004192	0.00356		0.006486	0.009255	0.008147	0.0025	0.0025
Lead (total)	(mg/L)	0.0844	0.0596	0.0136	0.0071		0.0059	0.0176	0.0037	0.0032	0.0032
Magnesium (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Magnesium (total)	(mg/L)	19.6	13	4	5.2		15.1	2.7	0.79	1.05	1.05
Mercury (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Mercury (total)	(mg/L)	<0.0004	<0.0004	<0.0004	<0.0004		<0.0004	<0.0004	<0.0004	<0.0004	<0.0004
Nickel (dissolved)	(mg/L)	0.0499	0.02994	<0.01	<0.01		0.00998	0.00998	0.00998	<0.01	<0.01
Nickel (total)	(mg/L)	0.05	0.03	<0.01	<0.01		<0.01	0.01	<0.01	<0.01	<0.01
Potassium (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Selenium (total)	(mg/L)	0.001	<0.001	0.002	0.001		<0.001	0.001	<0.001	<0.001	<0.001
Silver (total)	(mg/L)	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01
Sodium (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Sodium (total)	(mg/L)	159	170	190	949		36	13.5	6.67	17.8	17.8
Thallium (total)	(mg/L)	<0.0005	<0.0005	<0.0005	<0.0005		<0.0005	<0.0005	<0.0005	<0.0005	<0.0005
Zinc (dissolved)	(mg/L)	1.22	0.80	0.02	0.02		0.029	0.029	0.029	0.068	0.068
Zinc (total)	(mg/L)	1.25	0.82	0.18	0.13		0.1	0.29	0.04	0.07	0.07
Silica	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Bromide	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Chloride	(mg/L)	166	206	332	1709		49	18	6	13.0	13.0
Fluoride	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Nitrate	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Nitrite	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A

N/A: No data available.

Table D.3 Concentrations for Site C (cont.)

Analyte	Unit	Rainfall Event									
		3/27/2010		5/7/2010		5/20/2010		7/4/2010		9/13/2010	
		First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC
Nitrate/Nitrite Nitrogen	(mg/L)	0.4	0.5	<0.2	1	First flush sample could not be collected because a previous low intensity showers wash off the pollutants from the highway	1.2	1.3	0.5	1.0	1.0
Phosphate	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Sulfate	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Soluble Phosphate	(mg P/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Phosphorus (dissolved ortho)	(mg/L)	0.1	0.09	<0.05	<0.05		<0.05	<0.05	0.06	0.11	0.11
Total Phosphorus	(mg N/L)	1.5	0.81	0.1	0.13		0.16	0.32	0.28	0.28	0.28
Total Kjeldahl Nitrogen	(mg/L)	4.97	3	1.58	1.67		1.75	2.43	1.55	1.27	1.27
Total Dissolved Solids	(mg/L)	428	490	748	3236		156	22	<10	42	42
Total Suspended Solids	(mg/L)	1273	596	262	110		47	216	160	71	71
Total Solids	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Volatile Dissolved Solids	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Volatile Suspended Solids	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Total Volatile Solids	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Alkalinity as CaCO$_3$	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Oil and Grease	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
TEH as Diesel	(µg/L)	880	520	1780	650		390	231	85	407	407
BOD	(mg/L)	28	14	88	25		11	14	7	9	9
COD	(mg/L)	394	271	246	276		47	35	24	44	44
n-Hexane	(µg/L)	<1	<1	<1	<1		<1	<2	1	<1	<1
Methyl t-Butyl Ether	(µg/L)	<1	<1	<1	<1		<1	<1	<1	<1	<1
Benzene	(µg/L)	<1	<1	<1	<1		<1	<1	<1	<1	<1
Toluene	(µg/L)	<1	<1	<1	<1		<1	<1	<1	<1	<1
Ethylbenzene	(µg/L)	<1	<1	<1	<1		<1	<1	<1	<1	<1
Napthalene	(µg/L)	<1	<1	<1	<1		<1	<1	<1	<10	<10

N/A: No data available.

Table D.4 Concentrations for Site D

Analyte	Unit	Rainfall Event											
		11/10/2008		5/12/2009		6/1/2009		7/3/2009		7/31/2009		9/3/2009	
		First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC
Antimony (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Arsenic (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Beryllium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Cadmium (dissolved)	(mg/L)	0.0017	0.0016	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001
Cadmium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Calcium (dissolved)	(mg/L)	22.2	16.9	19.4	18.9	18.900	18.900	40.8	7.9	12.6	12.6	21.9	13.4
Calcium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Chromium (dissolved)	(mg/L)	0.0017	0.002	0.0041	0.0023	0.00110	0.00110	0.0024	0.0025	0.0108	0.0073	0.003	0.003
Chromium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Copper (dissolved)	(mg/L)	0.013	0.013	0.02	0.0167	0.00510	0.00510	0.0119	0.0057	0.0221	0.0175	0.004	0.0054
Copper (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Iron (dissolved)	(mg/L)	0.083	0.106	0.0093	0.0091	0.0029	0.0029	0.0046	0.0032	0.0027	0.0012	0.0037	0.0059
Lead (dissolved)	(mg/L)	0.0024	0.0025	0.0003	0.0003	0.0002	0.0002	0.0002	0.0003	0.0092	0.0087	0.0003	0.0003
Lead (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Magnesium (dissolved)	(mg/L)	1.6	1.4	3.7	4.1	1.000	1.000	1.7	0.3	1	0.6	1.4	1
Magnesium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Mercury (dissolved)	(mg/L)	N/A	N/A	0.0025	0.0023	0.003000	0.003000	0.0053	0.005	0.0041	0.004	0.0023	0.0022
Mercury (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Nickel (dissolved)	(mg/L)	N/A	N/A	0.0032	0.0021	0.002400	0.002400	0.0022	0.0008	0.0278	0.0139	0.0006	0.0006
Nickel (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Potassium (dissolved)	(mg/L)	2.2	1.9	7.7	8.2	3.400	3.400	7.5	1.3	4.6	0.7	1	1.3
Selenium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Silver (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Sodium (dissolved)	(mg/L)	25.7	19.5	121	150	75.700	75.700	184	34.8	43.3	41.5	8	17.8
Sodium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Thallium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Zinc (dissolved)	(mg/L)	0.019	0.014	<0.0001	<0.0001	<0.0001	<0.0001	0.001	0.0007	0.197	0.186	0.0023	<0.0001
Zinc (total)	(mg/L)	N/A	N/A	-	-	-	-	-	-	-	-	-	-
Silica	(mg/L)	8.4	7.6	1.97	2.9	2.090	2.090	8.8	2.3	1.6	2.1	-	-
Bromide	(mg/L)	<0.10	<0.10	< 0.10	< 0.10	< 0.10	< 0.10	< 0.10	< 0.10	<0.10	<0.10	<0.10	<0.10
Chloride	(mg/L)	41.2	28.5	221.5	215.3	156.500	156.500	330.3	39.3	53.3	38.6	7.72	18.6
Fluoride	(mg/L)	0.61	0.5	0.58	0.47	0.170	0.170	0.82	0.21	0.3	0.27	0.17	0.12
Nitrate	(mg/L)	0.38	0.31	0.63	0.71	0.420	0.420	0.15	0.23	0.64	0.65	0.25	0.26
Nitrite	(mg/L)	<0.10	<0.10	<0.10	<0.10	<0.10	<0.10	1.2	<0.10	<0.10	<0.10	<0.10	<0.10

N/A: No data available.

Table D.4 Concentrations for Site D (cont.)

Analyte	**Unit**	**Rainfall Event**											
		11/10/2008		**5/12/2009**		**6/1/2009**		**7/3/2009**		**7/31/2009**		**9/3/2009**	
		First Flush	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**
Nitrate/Nitrite Nitrogen	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Phosphate	(mg/L)	<0.10	<0.10	0.5	0.21	0.310	0.310	0.35	0.2	0.66	0.51	0.28	0.23
Sulfate	(mg/L)	10.5	8.74	40.7	38.1	69.800	69.800	36.7	8.83	17.9	11	4.83	7.36
Soluble Phosphate	(mg P/L)	0.09	0.1	0.05	0.07	0.060	0.060	0.11	0.09	0.188	0.517	0.043	0.081
Phosphorus (dissolved ortho)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Total Phosphorus	(mg N/L)	0.36	0.34	0.05	0.09	0.070	0.070	0.29	0.13	0.253	0.191	0.107	0.107
Total Kjeldahl Nitrogen	(mg/L)			1.26	1.04	3.860	3.860	2.85	0.49	2.63	2.2	<0.20	1.2
Total Dissolved Solids	(mg/L)	48	58	460	512	360.000	360.000	678	146	120	124	136	42
Total Suspended Solids	(mg/L)			154	<10.0	278.000	278.000	82	46	138	104	262	80
Total Solids	(mg/L)	506	904	614	512	638.000	638.000	760	192	258	228	398	122
Volatile Dissolved Solids	(mg/L)	48	58	<10.0	<10.0	10.000	10.000	26	156	58	16	94	<10.0
Volatile Suspended Solids	(mg/L)	114	100	74	72	128.000	128.000	24	40	16	36	52	60
Total Volatile Solids	(mg/L)	162	158	52	46	138.000	138.000	50	196	74	52	146	64
Alkalinity as $CaCO_3$	(mg/L)	51.5	41.2	60.1	50.2	76.800	76.800	86	28	38.1	37.5	36	30
Oil and Grease	(mg/L)	N/A	N/A	9.8	<5.0	9.6	9.6	<5.0	<5.0	<5.0	<5.0	<5.0	<5.0
TEH as Diesel	(μg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
BOD	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
COD	(mg/L)	29.2	26.1	24.9	27.8	32.1	32.1	50.1	20.1	21.8	20.1	9.6	<5.0
n-Hexane	(μg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Methyl t-Butyl Ether	(μg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Benzene	(μg/L)	<0.05	<0.05	<0.05	<0.05	0.07	0.07	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Toluene	(μg/L)	<0.05	<0.05	<0.05	0.05	0.06	0.06	<0.05	<0.05	0.08	0.1	<0.05	<0.05
Ethylbenzene	(μg/L)	<0.05	<0.05	<0.05	<0.05	< 0.05	< 0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Napthalene	(μg/L)	0.11	0.11	<0.05	<0.05	< 0.05	< 0.05	<0.05	<0.05	<0.05	<0.05	<0.05	0.06

N/A: No data available.

Table D.4 Concentrations for Site D (cont.)

Analyte	Unit	Rainfall Event									
		3/27/2010		5/7/2010		5/20/2010		7/4/2010		9/13/2010	
		First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC
Antimony (total)	(mg/L)	0.0076	0.0068	0.0024	0.0017	First flush sample could not be collected because a previous low intensity showers wash off the pollutants from the highway	0.0028	<0.0010	<0.0010	0.003	0.003
Arsenic (total)	(mg/L)	0.007	0.006	0.003	0.001		<0.001	<0.001	0.002	<0.001	<0.001
Beryllium (total)	(mg/L)	0.001	0.001	<0.0005	<0.0005		<0.0005	<0.0005	<0.0005	<0.0005	<0.0005
Cadmium (dissolved)	(mg/L)	0.0028	0.0028	<0.002	<0.002		<0.002	<0.002	<0.002	<0.002	<0.002
Cadmium (total)	(mg/L)	0.003	0.003	<0.002	<0.002		<0.002	<0.002	<0.002	<0.002	<0.002
Calcium (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Calcium (total)	(mg/L)	94.2	87.1	93.7	76.4		16.6	8.92	10.5	13.00	13.00
Chromium (dissolved)	(mg/L)	0.0491	0.03928	0.01964	0.01964		0.01964	<0.01	<0.01	<0.01	<0.01
Chromium (total)	(mg/L)	0.05	0.04	0.02	0.02		0.02	<0.01	<0.01	<0.01	<0.01
Copper (dissolved)	(mg/L)	0.1824	0.1152	0.048	0.0288		0.0288	0.0576	0.0096	0.0096	0.0096
Copper (total)	(mg/L)	0.1	0.08	0.04	0.02		0.02	0.01	<0.01	0.01	0.01
Iron (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Lead (dissolved)	(mg/L)	0.0394709	0.03077	0.010758	0.0053		0.004113	0.003639	0.003006	0.0024	0.0024
Lead (total)	(mg/L)	0.0499	0.0389	0.0136	0.0067		0.0052	0.0046	0.0038	0.003	0.003
Magnesium (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Magnesium (total)	(mg/L)	10.9	9.02	4.03	2.49		1.06	0.89	1.83	0.83	0.83
Mercury (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Mercury (total)	(mg/L)	<0.0004	<0.0004	<0.0004	<0.0004		<0.0004	<0.0004	<0.0004	<0.0004	<0.0004
Nickel (dissolved)	(mg/L)	0.02994	0.01996	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01
Nickel (total)	(mg/L)	0.03	0.02	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01
Potassium (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Selenium (total)	(mg/L)	0.003	<0.001	0.002	<0.001		<0.001	<0.001	<0.001	<0.001	<0.001
Silver (total)	(mg/L)	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01
Sodium (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Sodium (total)	(mg/L)	106	209	399	386		47.4	11	24.5	15.30	15.30
Thallium (total)	(mg/L)	<0.0005	<0.0005	<0.0005	<0.0005		<0.0005	<0.0005	<0.0005	<0.0005	<0.0005
Zinc (dissolved)	(mg/L)	0.83	0.65	0.29	0.14		0.108	0.078	0.029	0.078	0.078
Zinc (total)	(mg/L)	0.85	0.66	0.3	0.14		0.11	0.08	0.03	0.08	0.08
Silica	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Bromide	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Chloride	(mg/L)	116	254	986	690		66	5	15	11.0	11.0
Fluoride	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Nitrate	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Nitrite	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A

N/A: No data available.

Table D.4 Concentrations for Site D (cont.)

Analyte	Unit	Rainfall Event									
		3/27/2010		5/7/2010		5/20/2010		7/4/2010		9/13/2010	
		First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC	First Flush	EMC
Nitrate/Nitrite Nitrogen	(mg/L)	0.5	0.7	2.6	1.8	First flush sample could not be collected because a previous low intensity showers wash off the pollutants from the highway	0.4	0.2	<0.2	0.8	0.8
Phosphate	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Sulfate	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Soluble Phosphate	(mg P/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Phosphorus (dissolved ortho)	(mg/L)	0.1	0.09	0.14	0.09		0.09	0.08	0.11	0.06	0.06
Total Phosphorus	(mg N/L)	0.66	0.51	0.51	0.36		0.16	0.18	0.25	0.16	0.16
Total Kjeldahl Nitrogen	(mg/L)	2.44	1.91	4.32	3.53		0.82	1.07	0.83	1.24	1.24
Total Dissolved Solids	(mg/L)	296	588	1884	1334		168	<10	34	10	10
Total Suspended Solids	(mg/L)	622	419	241	122		32	71	59	51	51
Total Solids	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Volatile Dissolved Solids	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Volatile Suspended Solids	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Total Volatile Solids	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Alkalinity as $CaCO_3$	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Oil and Grease	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
TEH as Diesel	(μg/L)	260	570	1400	710		270	172	54	370	370
BOD	(mg/L)	18	14	34	24		9	7	6	9	9.0
COD	(mg/L)	172	200	292	80		46	23	29	40	40.0
n-Hexane	(μg/L)	<1	<1	1	1		<1	<1	<1	<1	<1
Methyl t-Butyl Ether	(μg/L)	<1	<1	<1	<1		<1	<1	<1	<1	<1
Benzene	(μg/L)	<1	<1	<1	<1		1	<1	<1	<1	<1
Toluene	(μg/L)	<1	<1	<1	<1		<1	<1	<1	<1	<1
Ethylbenzene	(μg/L)	<1	<1	<1	<1		<1	<1	<1	<1	<1
Napthalene	(μg/L)	<1	<1	<1	<1		<1	<1	<1	<1	<1

N/A: No data available.

Table D.5 Concentrations for Site E

Analyte	**Unit**	**Rainfall Event**											
		11/10/2008		**5/12/2009**		**6/1/2009**		**7/3/2009**		**7/31/2009**		**9/3/2009**	
		First Flush	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**
Antimony (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Arsenic (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Beryllium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Cadmium (dissolved)	(mg/L)	0.002	0.0023	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001
Cadmium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Calcium (dissolved)	(mg/L)	45.8	50.5	19.4	18.9	23.9	23.9	42.2	12.1	12.1	21.7	22.8	20.2
Calcium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Chromium (dissolved)	(mg/L)	0.0015	0.0017	0.0018	0.0016	0.002	0.002	0.0012	0.0032	0.0082	0.0078	0.0033	0.0036
Chromium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Copper (dissolved)	(mg/L)	0.02	0.014	0.0116	0.0088	0.0083	0.0083	0.0103	0.0057	0.0133	0.016	0.0063	0.0044
Copper (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Iron (dissolved)	(mg/L)	0.104	0.049	0.0016	0.0052	0.0173	0.0173	0.0021	0.0027	0.0874	0.0915	0.0019	0.0042
Lead (dissolved)	(mg/L)	0.0024	0.0023	0.0003	0.0003	0.0003	0.0003	0.0002	0.0003	0.0048	0.0054	0.0003	0.0003
Lead (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Magnesium (dissolved)	(mg/L)	8.5	20.5	3.7	4.1	1.40	1.40	6.3	1.4	0.6	2.4	2.6	5
Magnesium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Mercury (dissolved)	(mg/L)	N/A	N/A	0.0023	0.0024	0.0027	0.0027	0.0047	0.0044	0.0039	0.0039	0.0022	0.0022
Mercury (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Nickel (dissolved)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Nickel (total)	(mg/L)	N/A	N/A	0.0025	0.0022	0.0028	0.0028	0.0032	0.0012	0.0085	0.014	0.0013	0.0013
Potassium (dissolved)	(mg/L)	9.9	17.9	7.7	8.2	5.100	5.100	16.4	4.5	4.9	8.8	4	16.9
Selenium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Silver (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Sodium (dissolved)	(mg/L)	162	232	121	150	107	107	326	62.4	34.6	86	34.6	104
Sodium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Thallium (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Zinc (dissolved)	(mg/L)	0.029	0.0045	<0.10	<0.10	0.0193	19.3	<0.0001	<0.0001	0.265	0.164	0.0006	<0.0001
Zinc (total)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Silica	(mg/L)	14.5	15.3	2.34	4.28	2.0	2.0	15.9	5.1	1.5	4.7	N/A	N/A
Bromide	(mg/L)	<0.10	<0.10	< 0.10	< 0.10	< 0.10	< 0.10	< 0.10	< 0.10	<0.10	<0.10	<0.10	<0.10
Chloride	(mg/L)	275.5	400.1	242.6	293.8	196.1	196.1	604	64.1	41.5	109.5	114.8	123.7
Fluoride	(mg/L)	0.7	0.33	0.51	0.51	0.180	0.180	0.49	0.17	0.27	0.26	0.2	0.12
Nitrate	(mg/L)	<0.10	0.23	0.32	0.57	0.690	0.690	0.47	0.32	0.67	0.87	0.94	0.66
Nitrite	(mg/L)	<0.10	<0.10	<0.10	<0.10	0.370	0.370	<0.10	<0.10	<0.10	<0.10	<0.10	<0.10

N/A: No data available.

Table D.5 Concentrations for Site E (cont.)

Analyte	**Unit**	**Rainfall Event**											
		11/10/2008		**5/12/2009**		**6/1/2009**		**7/3/2009**		**7/31/2009**		**9/3/2009**	
		First Flush	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**
Nitrate/Nitrite Nitrogen	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Phosphate	(mg/L)	<0.10	<0.10	0.4	0.32	0.120	0.120	<0.10	0.23	0.4	0.41	0.35	0.23
Sulfate	(mg/L)	41.8	104.4	90.6	81.7	31.3	31.3	129.1	32.6	12.2	33.3	28.1	47.9
Soluble Phosphate	(mg P/L)	0.07	0.03	0.09	0.08	0.040	0.040	0.03	0.08	0.824	0.134	0.068	0.034
Phosphorus (dissolved ortho)	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Total Phosphorus	(mg N/L)	0.38	0.26	0.04	0.08	0.070	0.070	0.21	0.14	0.086	0.125	0.098	0.03
Total Kjeldahl Nitrogen	(mg/L)	N/A	N/A	2.33	2.03	2.79	2.79	1.89	0.55	0.66	0.6	1.64	0.9
Total Dissolved Solids	(mg/L)	N/A	N/A	650	674	424	424	1,168	548	118	184	50	470
Total Suspended Solids	(mg/L)	N/A	N/A	158	200	338	338	300	758	128	190	634	306
Total Solids	(mg/L)	1262	966	808	874	762	762	1,468	1,306	246	374	684	776
Volatile Dissolved Solids	(mg/L)	123	<10	<10.0	<10.0	<10.0	<10.0	50	200	40	44	26	52
Volatile Suspended Solids	(mg/L)	165	169	100	86	106	106	58	24	56	12	68	18
Total Volatile Solids	(mg/L)	288	170	84	84	98.0	98.0	108	224	96	56	94	70
Alkalinity as $CaCO_3$	(mg/L)	118	158	107	88.5	69.5	69.5	121.1	68	32.6	55	55	66.8
Oil and Grease	(mg/L)	N/A	N/A	13.1	7.8	9.8	9.8	<5.0	<5.0	<5.0	<5.0	<5.0	<5.0
TEH as Diesel	(µg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
BOD	(mg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
COD	(mg/L)	47.3	64.2	18.5	14.9	54.9	54.9	51.8	49.6	15.7	29	<5.0	47.4
n-Hexane	(µg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Methyl t-Butyl Ether	(µg/L)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Benzene	(µg/L)	<0.05	<0.05	0.06	0.06	< 0.05	< 0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Toluene	(µg/L)	<0.05	<0.05	<0.05	<0.05	0.06	0.06	<0.05	<0.05	0.17	0.1	0.1	0.08
Ethylbenzene	(µg/L)	<0.05	<0.05	<0.05	<0.05	< 0.05	< 0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Napthalene	(µg/L)	0.11	0.11	<0.05	<0.05	< 0.05	< 0.05	<0.05	<0.05	<0.05	<0.05	0.1	0.05

N/A: No data available.

Table D.5 Concentrations for Site E (cont.)

Analyte	**Unit**	**Rainfall Event**									
		3/27/2010		**5/7/2010**		**5/20/2010**		**7/4/2010**		**9/13/2010**	
		First Flush	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**
Antimony (total)	(mg/L)	0.003	0.003	0.0013	0.0024	First flush sample could not be collected because a previous low intensity showers wash off the pollutants from the highway	0.0027	<0.0010	<0.0010	0.0021	0.0021
Arsenic (total)	(mg/L)	0.003	0.003	0.004	0.002		0.003	<0.001	0.003	0.003	0.003
Beryllium (total)	(mg/L)	<0.0005	<0.0005	<0.0005	<0.0005		<0.0005	<0.0005	<0.0005	<0.0005	<0.0005
Cadmium (dissolved)	(mg/L)	<0.002	<0.002	<0.002	<0.002		<0.002	<0.002	<0.002	<0.002	<0.002
Cadmium (total)	(mg/L)	<0.002	<0.002	<0.002	<0.002		<0.002	<0.002	<0.002	<0.002	<0.002
Calcium (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Calcium (total)	(mg/L)	85.7	78.9	114	33.2		32.3	11.4	21.3	28.4	28.4
Chromium (dissolved)	(mg/L)	0.0196	0.01964	0.0196	0.0196		0.0196	<0.01	0.00982	<0.01	<0.01
Chromium (total)	(mg/L)	0.02	0.02	0.02	0.02		0.02	<0.01	0.01	<0.01	<0.01
Copper (dissolved)	(mg/L)	0.0288	0.0288	0.0192	0.0288		0.0192	0.0096	0.0096	0.0096	0.0096
Copper (total)	(mg/L)	0.03	0.03	0.02	0.03		0.02	0.01	0.01	0.01	0.010
Iron (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Lead (dissolved)	(mg/L)	0.00973	0.00831	0.00419	0.00625		0.003718	0.002373	0.005537	0.0018	0.00182
Lead (total)	(mg/L)	0.0123	0.0105	0.0053	0.0079		0.0047	0.003	0.007	0.0023	0.00230
Magnesium (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Magnesium (total)	(mg/L)	16.9	15.1	7.84	2.61		5.29	1.74	3.25	8.98	8.98
Mercury (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Mercury (total)	(mg/L)	<0.0004	<0.0004	<0.0004	<0.0004		<0.0004	<0.0004	<0.0004	<0.0004	<0.0004
Nickel (dissolved)	(mg/L)	0.00998	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01
Nickel (total)	(mg/L)	0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01
Potassium (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Selenium (total)	(mg/L)	<0.001	<0.001	0.002	<0.001		<0.001	<0.001	0.003	<0.001	<0.001
Silver (total)	(mg/L)	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01
Sodium (dissolved)	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Sodium (total)	(mg/L)	380	389	621	107		150	30.6	40.4	111	111
Thallium (total)	(mg/L)	<0.0005	<0.0005	<0.0005	<0.0005		<0.0005	<0.0005	<0.0005	<0.0005	<0.0005
Zinc (dissolved)	(mg/L)			0.07	0.09		0.068	0.049	0.029	0.039	0.039
Zinc (total)	(mg/L)	0.2	0.17	0.07	0.09		0.07	0.05	0.03	0.040	0.040
Silica	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Bromide	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Chloride	(mg/L)	499	510	1092	149		185	25	39	118	118
Fluoride	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Nitrate	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Nitrite	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A

N/A: No data available.

Table D.5 Concentrations for Site E (cont.)

Analyte	Unit	Rainfall Event									
		3/27/2010		5/7/2010		5/20/2010		7/4/2010		9/13/2010	
		First Flush	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**	**First Flush**	**EMC**
Nitrate/Nitrite Nitrogen	(mg/L)	0.4	0.4	1.6	1.8	First flush sample could not be collected because a previous low intensity showers wash off the pollutants from the highway	0.6	0.2	0.3	0.900	0.900
Phosphate	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Sulfate	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Soluble Phosphate	(mg P/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Phosphorus (dissolved ortho)	(mg/L)	<0.05	<0.05	0.05	0.08		<0.05	0.06	0.13	0.07	0.070
Total Phosphorus	(mg N/L)	0.25	0.25	0.21	0.34		0.17	0.15	0.3	0.21	0.21
Total Kjeldahl Nitrogen	(mg/L)	1.51	1.19	2.49	3.27		0.95	1.00	1.19	1.67	1.67
Total Dissolved Solids	(mg/L)	1262	1330	2054	372		520	16	128	512	512
Total Suspended Solids	(mg/L)	187	134	129	135		68	41	108	46	46.0
Total Solids	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Volatile Dissolved Solids	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Volatile Suspended Solids	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Total Volatile Solids	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Alkalinity as $CaCO_3$	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Oil and Grease	(mg/L)	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
TEH as Diesel	(µg/L)	200	290	610	1010		260	121	71	597	597
BOD	(mg/L)	11	13	16	26		9	6	7	15	15.0
COD	(mg/L)	166	194	157	138		60	23	43	101	101
n-Hexane	(µg/L)	<1	<1	<1	1		<1	<1	2	N/A	N/A
Methyl t-Butyl Ether	(µg/L)	<1	<1	<1	<1		<1	<1	<1	N/A	N/A
Benzene	(µg/L)	<1	<1	<1	<1		<1	<1	<1	N/A	N/A
Toluene	(µg/L)	<1	<1	<1	<1		<1	<1	<1	N/A	N/A
Ethylbenzene	(µg/L)	<1	<1	<1	<1		<1	<1	<1	N/A	N/A
Napthalene	(µg/L)	<1	<1	<1	<1		<1	<1	<1	N/A	N/A

N/A: No data available.

Table D.6 Pollutant loads for Site A

Analyte	Pollutant Load (Kg/event)									
	11/10/2008	**5/12/2009**	**7/3/2009**	**7/31/2009**	**9/3/2009**	**3/27/2010**	**5/7/2010**	**5/20/2010**	**7/4/2010**	**9/13/2010**
Antimony (total)	N/A	N/A	N/A	N/A	N/A	5.78E-06	1.59E-05	5.17E-11	N.D	1.54E-04
Arsenic (total)	N/A	N/A	N/A	N/A	N/A	3.61E-05	1.19E-04	2.95E-10	1.04E-09	0.00134
Beryllium (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	2.08E-10	1.03E-04
Cadmium (dissolved)	4.24E-05	N.D	N.D	N.D	N.D	N/A	N/A	N/A	N/A	N.D
Cadmium (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Calcium (dissolved)	0.565	0.0749	0.964	6.74	14.15	N/A	N/A	N/A	N/A	N/A
Calcium (total)	N/A	N/A	N/A	N/A	N/A	0.431	1.30	N/A	7.03E-06	5.59
Chromium (dissolved)	3.57E-05	1.51E-04	1.84E-04	5.51E-04	0.00707	N.D	1.95E-04	1.09E-09	4.09E-09	0.00202
Chromium (total)	N/A	N/A	N/A	N/A	N/A	N.D	1.99E-04	1.11E-09	4.16E-09	0.00206
Copper (dissolved)	1.45E-04	1.06E-04	0.00222	1.84E-04	0.00455	N.D	N.D	3.55E-10	2E-09	0.00198
Copper (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	3.69E-10	2.08E-09	0.00206
Iron (dissolved)	0.00136	N.D	N.D	0.0714	0.00387	N/A	N/A	N/A	N/A	N/A
Lead (dissolved)	5.91E-05	4.99E-06	1.42E-05	6.78E-04	3.37E-04	8.58E-06	7.07E-05	2.4E-10	1.7E-09	5.95E-04
Lead (total)	N/A	N/A	N/A	N/A	N/A	1.08E-05	8.94E-05	3.03E-10	2.14E-09	7.52E-04
Magnesium (dissolved)	0.192	0.0786	0.248	0.654	6.23	N/A	N/A	N/A	N/A	N/A
Magnesium (total)	N/A	N/A	N/A	N/A	N/A	0.270	0.586	5.58E-07	1.04E-06	1.96
Mercury (dissolved)	N/A	3.37E-05	7.23E-04	1.74E-04	0.00421	N/A	N/A	N/A	N/A	N/A
Mercury (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Nickel (dissolved)	N/A	5.49E-05	9.22E-05	3.91E-04	0.00168	N.D	N.D	3.69E-10	2.08E-09	0.00103
Nickel (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	3.69E-10	2.08E-09	0.00103
Potassium (dissolved)	0.229	0.537	0.858	2.85	35.9	N/A	N/A	N/A	N/A	N/A
Selenium (total)	N/A	N/A	N/A	N/A	N/A	1.45E-05	N.D	3.69E-11	2.08E-10	2.06E-04
Silver (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Sodium (dissolved)	2.04	4.41	7.09	16.4	236	N/A	N/A	N/A	N/A	N/A
Sodium (total)	N/A	N/A	N/A	N/A	N/A	2.11	7.01	1.10E-05	1.52E-05	45.2
Thallium (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Zinc (dissolved)	1.78E-04	1.25E-06	7.09E-06	0.00147	N.D	N.D	3.88E-04	1.08E-09	6.11E-09	0.00403
Zinc (total)	N/A	N/A	N/A	N/A	N/A	N.D	3.97E-04	1.11E-09	6.24E-09	0.00412
Silica	0.239	0.102	0.588	0.952	N.D	N/A	N/A	N/A	N/A	N/A
Bromide	N.D	0.00125	0.00709	0.0180	0.168	N/A	N/A	N/A	N/A	N/A
Chloride	5.09	8.75	8.56	21.2	254	1.73	6.51	1.38E-05	1.29E-05	41.5
Fluoride	N.D	0.00387	0.0106	0.00401	0.168	N/A	N/A	N/A	N/A	N/A
Nitrate	0.00914	0.0223	0.0369	0.0892	2.29	N/A	N/A	N/A	N/A	N/A
Nitrite	N.D	0.00125	0.00709	0.00334	0.168	N/A	N/A	N/A	N/A	N/A

N/A: No data available.

N.D: Non-detected

Table D.6 Pollutant loads for Site A (cont.)

Analyte	**Pollutant Load (Kg/event)**									
	11/10/2008	**5/12/2009**	**7/3/2009**	**7/31/2009**	**9/3/2009**	**3/27/2010**	**5/7/2010**	**5/20/2010**	**7/4/2010**	**9/13/2010**
Nitrate/Nitrite Nitrogen	N/A	N/A	N/A	N/A	N/A	N.D	0.0199	4.43E-08	1.04E-07	0.165
Phosphate	N.D	0.00125	0.00709	0.00334	0.168	N/A	N/A	N/A	N/A	N/A
Sulfate	1.90	3.42	5.11	6.71	98.2	N/A	N/A	N/A	N/A	N/A
Soluble Phosphate	N.D	2.50E-04	0.00213	0.0139	0.0337	N/A	N/A	N/A	N/A	N/A
Phosphorus (dissolved ortho)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	1.25E-08	N.D
Total Phosphorus	0.00145	N.D	0.00425	0.00224	N.D	7.23E-04	0.00258	5.91E-09	5.83E-08	0.0474
Total Kjeldahl Nitrogen	N/A	0.0397	0.0971	0.178	N.D	0.0103	0.0332	6.46E-08	3.23E-07	0.495
Total Dissolved Solids	N/A	24.9	14.9	46.1	788	8.51	27.8	4.60E-05	4.75E-05	160
Total Suspended Solids	N/A	7.96	26.9	45.1	330	0.253	2.82	5.80E-06	5.22E-05	23.3
Total Solids	15.9	32.9	41.8	91.2	1118	N/A	N/A	N/A	N/A	N/A
Volatile Dissolved Solids	2.38	1.37	1.13	13.2	70.7	N/A	N/A	N/A	N/A	N/A
Volatile Suspended Solids	0.569	1.95	4.54	1.80	37.1	N/A	N/A	N/A	N/A	N/A
Total Volatile Solids	2.94	3.32	5.67	15.0	108	N/A	N/A	N/A	N/A	N/A
Alkalinity as $CaCO_3$	1.98	1.83	3.65	4.75	109	N/A	N/A	N/A	N/A	N/A
Oil and Grease	N/A	0.356	N.D	N.D	N.D	N/A	N/A	N/A	N/A	N/A
TEH as Diesel	N/A	N/A	N/A	N/A	N/A	2.46	2.38	N.D	1.04E-05	47.6
BOD	0.00781	N/A	N/A	N/A	N/A	0.0361	0.179	2.95E-07	1.66E-06	2.27
COD	9.32E-04	1.131	2.21	2.66	8.42	1.37	4.85	3.95E-06	9.57E-06	31.1
n-Hexane	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	2.08E-07	N.D
Methyl t-Butyl Ether	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D
Benzene	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D
Toluene	N.D	9.99E-07	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D
Ethylbenzene	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D
Napthalene	N.D	N.D	N.D	N.D	1.52E-04	N.D	N.D	N.D	N.D	N.D

N/A: No data available.

N.D: Non-detected

Table D.7 Pollutant loads for Site C

Analyte	**Pollutant Load (Kg/event)**									
	11/10/2008	**5/12/2009**	**7/3/2009**	**7/31/2009**	**9/3/2009**	**3/27/2010**	**5/7/2010**	**5/20/2010**	**7/4/2010**	**9/13/2010**
Antimony (total)	N/A	N/A	N/A	N/A	N/A	6.00E-05	2.46E-05	7.8E-11	N.D	4.02E-05
Arsenic (total)	N/A	N/A	N/A	N/A	N/A	4.89E-05	2.73E-05	N.D	N.D	N.D
Beryllium (total)	N/A	N/A	N/A	N/A	N/A	6.12E-06	N.D	N.D	N.D	N.D
Cadmium (dissolved)	1.51E-05	N.D	N.D	N.D	N.D	1.73E-05	N/A	N/A	N/A	N.D
Cadmium (total)	N/A	N/A	N/A	N/A	N/A	1.84E-05	N.D	N.D	N.D	N.D
Calcium (dissolved)	0.209	0.145	0.306	0.313	5.38	N/A	N/A	N/A	N/A	N/A
Calcium (total)	N/A	N/A	N/A	N/A	N/A	0.595	2.54	3.84E-07	7.84E-07	0.241
Chromium (dissolved)	1.43E-05	2.76E-05	1.16E-04	1.18E-04	6.19E-04	3.60E-04	4.02E-04	4.14E-10	N.D	N.D
Chromium (total)	N/A	N/A	N/A	N/A	N/A	3.67E-04	4.09E-04	4.22E-10	N.D	N.D
Copper (dissolved)	2.02E-04	1.31E-04	1.86E-04	2.85E-04	0.00252	7.05E-04	3.93E-04	6.07E-10	9.20E-10	2.86E-04
Copper (total)	N/A	N/A	N/A	N/A	N/A	7.34E-04	4.09E-04	6.33E-10	9.59E-10	2.98E-04
Iron (dissolved)	0.00102	2.58E-05	<0.001	0.00162	0.00271	N/A	N/A	N/A	N/A	N/A
Lead (dissolved)	2.02E-05	2.76E-06	7.46E-06	1.22E-04	1.43E-04	2.88E-04	7.66E-05	9.84E-11	2.81E-10	3.77E-05
Lead (total)	N/A	N/A	N/A	N/A	N/A	3.65E-04	9.69E-05	1.24E-10	3.55E-10	4.76E-05
Magnesium (dissolved)	0.0244	0.00828	0.0112	0.00611	0.190	N/A	N/A	N/A	N/A	N/A
Magnesium (total)	N/A	N/A	N/A	N/A	N/A	0.0795	0.071	2.51E-08	7.57E-08	0.0156
Mercury (dissolved)	N.D	2.30E-05	2.05E-04	8.55E-05	0.00109	N/A	N/A	N/A	N/A	N/A
Mercury (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Nickel (dissolved)	0	1.29E-05	2.24E-05	1.83E-04	3.33E-04	1.83E-04	N.D	N.D	N.D	N.D
Nickel (total)	N/A	N/A	N/A	N/A	N/A	1.84E-04	N.D	N.D	N.D	N.D
Potassium (dissolved)	0.0446	0.0221	0.0298	0.00407	0.666	N/A	N/A	N/A	N/A	N/A
Selenium (total)	N/A	N/A	N/A	N/A	N/A	N.D	1.36E-05	N.D	N.D	N.D
Silver (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Sodium (dissolved)	0.304	0.807	0.638	0.374	6.00	N/A	N/A	N/A	N/A	N/A
Sodium (total)	N/A	N/A	N/A	N/A	N/A	1.04	12.9	7.59E-07	6.40E-07	0.265
Thallium (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Zinc (dissolved)	1.77E-04	N.D	N.D	0.00330	N.D	0.00491	0.00173	2.06E-09	3.75E-09	0.00102
Zinc (total)	N/A	N/A	N/A	N/A	N/A	0.00502	0.00177	2.11E-09	3.84E-09	0.00104
Silica	0.0580	0.0317	0.0671	0.0529	N.D	N/A	N/A	N/A	N/A	N/A
Bromide	N.D	N.D	N.D	N.D	0.0476	N/A	N/A	N/A	N/A	N/A
Chloride	0.4	1.03	0.380	0.427	6.47	1.26	23.3	1.03E-06	5.75E-07	0.193
Fluoride	0.00555	0.00469	0.00858	0.00651	0.0714	N/A	N/A	N/A	N/A	N/A
Nitrate	N.D	0.00635	0.0134	0.0226	0.200	N/A	N/A	N/A	N/A	N/A
Nitrite	N.D	N.D	N.D	N.D	N.D	N/A	N/A	N/A	N/A	N/A

N/A: No data available.
N.D: Non-detected

Table D.7 Pollutant loads for Site C (cont.)

Analyte	**Pollutant Load (Kg/event)**									
	11/10/2008	**5/12/2009**	**7/3/2009**	**7/31/2009**	**9/3/2009**	**3/27/2010**	**5/7/2010**	**5/20/2010**	**7/4/2010**	**9/13/2010**
Nitrate/Nitrite Nitrogen	N/A	N/A	N/A	N/A	N/A	0.00306	0.0327	1.05E-08	1.92E-08	0.0149
Phosphate	N.D	0.00276	0.0131	0.00773	0.143	N/A	N/A	N/A	N/A	N/A
Sulfate	0.170	0.384	0.287	0.338	3.10	N/A	N/A	N/A	N/A	N/A
Soluble Phosphate	0.00109	0.00101	0.00261	0.0165	0.0414	N/A	N/A	N/A	N/A	N/A
Phosphorus (dissolved ortho)	N/A	N/A	N/A	N/A	N/A	5.51E-04	0.00123	1.9E-09	1.05E-08	0.00164
Total Phosphorus	0.00303	4.60E-04	0.00410	0.00218	0.0600	0.00496	0.00409	5.06E-09	2.11E-08	0.00417
Total Kjeldahl Nitrogen	N/A	0.0151	0.0265	0.0326	0.875	0.0184	0.0423	2.51E-08	6.14E-08	0.0189
Total Dissolved Solids	N/A	2.50	2.91	1.83	58.0	3.00	44.0	3.29E-06	N.D	0.625
Total Suspended Solids	N/A	1.07	4.62	2.36	56.1	3.65	1.50	9.91E-07	5.75E-06	1.06
Total Solids	N/A	3.57	7.53	4.19	114	N/A	N/A	N/A	N/A	N/A
Volatile Dissolved Solids	0.749	N.D	0.895	0.448	7.61	N/A	N/A	N/A	N/A	N/A
Volatile Suspended Solids	0.429	0.497	1.27	0.611	54.2	N/A	N/A	N/A	N/A	N/A
Total Volatile Solids	1.18	0.515	2.16	1.06	61.9	N/A	N/A	N/A	N/A	N/A
Alkalinity as $CaCO_3$	0.531	0.628	0.94	0.81	13.0	N/A	N/A	N/A	N/A	N/A
Oil and Grease	N.D	N.D	N.D	N.D	N.D	N/A	N/A	N/A	N/A	N/A
TEH as Diesel	N/A	N/A	N/A	N/A	N/A	0.00318	0.0089	8.23E-09	8.15E-09	6.05
BOD	0.0446	N/A	N/A	N/A	N/A	0.0857	0.341	2.32E-07	6.71E-07	0.134
COD	0.298	0.216	3.30	0.297	N.D	1.66	3.77	9.91E-07	2.30E-06	0.655
n-Hexane	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	9.59E-11	N.D
Methyl t-Butyl Ether	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D
Benzene	N.D	N.D	N.D	N.D	N.D	N.D	N.D	2.11E-11	N.D	N.D
Toluene	N.D	N.D	N.D	1.22E-06	N.D	N.D	N.D	N.D	N.D	N.D
Ethylbenzene	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D
Napthalene	N.D	N.D	N.D	N.D	2.85E-05	N.D	N.D	N.D	N.D	N.D

N/A: No data available.
N.D: Non-detected

Table D.8 Pollutant loads for Site D

Analyte	Pollutant Load (Kg/event)									
	11/10/2008	**5/12/2009**	**7/3/2009**	**7/31/2009**	**9/3/2009**	**3/27/2010**	**5/7/2010**	**5/20/2010**	**7/4/2010**	**9/13/2010**
Antimony (total)	N/A	N/A	N/A	N/A	N/A	4.16E-05	2.32E-05	5.91E-11	<0.0010	4.46E-05
Arsenic (total)	N/A	N/A	N/A	N/A	N/A	3.67E-05	1.36E-05	N.D	1.92E-10	N.D
Beryllium (total)	N/A	N/A	N/A	N/A	N/A	6.12E-06	N.D	N.D	N.D	N.D
Cadmium (dissolved)	2.32E-05	N.D	N.D	N.D	N.D	1.73E-05	N/A	N/A	N/A	N.D
Cadmium (total)	N/A	N/A	N/A	N/A	N/A	1.84E-05	N.D	N.D	N.D	N.D
Calcium (dissolved)	0.246	0.168	0.295	0.256	6.38	N/A	N/A	N/A	N/A	N/A
Calcium (total)	N/A	N/A	N/A	N/A	N/A	0.533	1.04	3.5E-07	1.01E-06	0.19
Chromium (dissolved)	2.91E-05	2.12E-05	9.32E-05	1.49E-04	0.00143	2.40E-04	2.68E-04	4.14E-10	N.D	N.D
Chromium (total)	N/A	N/A	N/A	N/A	N/A	2.45E-04	2.73E-04	4.22E-10	N.D	N.D
Copper (dissolved)	1.89E-04	1.54E-04	2.13E-04	3.56E-04	0.00257	4.70E-04	2.62E-04	4.05E-10	N.D	1.43E-04
Copper (total)	N/A	N/A	N/A	N/A	N/A	4.89E-04	2.73E-04	4.22E-10	N.D	1.49E-04
Iron (dissolved)	0.00154	8.37E-05	1.19E-04	0.025	0.00281	N/A	N/A	N/A	N/A	N/A
Lead (dissolved)	3.63E-05	2.76E-06	1.12E-05	1.77E-04	1.43E-04	1.88E-04	7.23E-05	8.67E-11	2.88E-10	3.53E-05
Lead (total)	N/A	N/A	N/A	N/A	N/A	2.38E-04	9.14E-05	1.1E-10	3.64E-10	4.46E-05
Magnesium (dissolved)	0.0203	0.0110	0.0112	0.0122	0.476	N/A	N/A	N/A	N/A	N/A
Magnesium (total)	N/A	N/A	N/A	N/A	N/A	0.06	0.0340	2.24E-08	1.75E-07	0.0123
Mercury (dissolved)	N/A	2.12E-05	1.86E-04	8.14E-05	0.00105	N/A	N/A	N/A	N/A	N/A
Mercury (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Nickel (dissolved)	N/A	1.93E-05	2.98E-05	0.0003	2.85E-04	1.22E-04	N.D	N.D	N.D	N.D
Nickel (total)	N/A	N/A	N/A	N/A	N/A	1.22E-04	N.D	N.D	N.D	N.D
Potassium (dissolved)	0.0276	0.0258	0.0485	0.0142	0.619	N/A	N/A	N/A	N/A	N/A
Selenium (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Silver (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Sodium (dissolved)	0.283	1.04	1.30	0.845	8.47	N/A	N/A	N/A	N/A	N/A
Sodium (total)	N/A	N/A	N/A	N/A	N/A	1.28	5.27	1.00E-06	0.00	0.228
Thallium (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Zinc (dissolved)	2.03E-04	N.D	N.D	N.D	N.D	0.00395	0.00187	2.27E-09	2.81E-09	1.16E-03
Zinc (total)	N/A	N/A	N/A	N/A	N/A	0.00404	0.00191	2.32E-09	2.88E-09	1.19E-03
Silica	0.110	0.0267	0.086	0.0427	N.D	N/A	N/A	N/A	N/A	N/A
Bromide	N.D	N.D	N.D	N.D	N.D	N/A	N/A	N/A	N/A	N/A
Chloride	0.414	1.98	1.47	0.786	8.85	1.55	9.41	1.39E-06	1.44E-06	1.64E-01
Fluoride	0.00727	0.00432	0.00783	0.00550	0.0571	N/A	N/A	N/A	N/A	N/A
Nitrate	0.00450	0.00653	0.00858	0.0132	0.124	N/A	N/A	N/A	N/A	N/A
Nitrite	N.D	N.D	N.D	N.D	N.D	N/A	N/A	N/A	N/A	N/A

N/A: No data available.
N.D: Non-detected

Table D.8 Pollutant loads for Site D (cont.)

Analyte	**Pollutant Load (Kg/event)**									
	11/10/2008	**5/12/2009**	**7/3/2009**	**7/31/2009**	**9/3/2009**	**3/27/2010**	**5/7/2010**	**5/20/2010**	**7/4/2010**	**9/13/2010**
Nitrate/Nitrite Nitrogen	N/A	N/A	N/A	N/A	N/A	0.00428	0.0246	8.44E-09	N.D	1.19E-02
Phosphate	N.D	0.00193	0.00746	0.0104	0.109	N/A	N/A	N/A	N/A	N/A
Sulfate	0.127	0.351	0.329	0.22	3.50	N/A	N/A	N/A	N/A	N/A
Soluble Phosphate	0.00145	6.44E-04	0.00336	0.0105	0.0385	N/A	N/A	N/A	N/A	N/A
Phosphorus (dissolved ortho)	N/A	N/A	N/A	N/A	N/A	5.51E-04	0.00109	N.D	1.15E-08	8.93E-04
Total Phosphorus	0.00494	8.28E-04	0.00485	0.00389	0.0509	0.00312	0.00491	3.37E-09	2.40E-08	0.00238
Total Kjeldahl Nitrogen	N/A	0.00957	0.0183	0.0442	0.561	0.0117	0.0482	1.73E-08	7.96E-08	0.0184
Total Dissolved Solids	N/A	4.71	5.44	2.52	20.0	3.60	18.2	3.54E-06	3.26E-06	0.149
Total Suspended Solids	N/A	0.0920	1.72	2.12	38.1	2.56	1.66	6.75E-07	5.66E-06	7.59E-01
Total Solids	13.1	4.71	7.16	4.64	58.0	N/A	N/A	N/A	N/A	N/A
Volatile Dissolved Solids	0.843	N.D	5.82	0.326	<10.0	N/A	N/A	N/A	N/A	N/A
Volatile Suspended Solids	1.45	0.663	1.49	0.733	28.5	N/A	N/A	N/A	N/A	N/A
Total Volatile Solids	2.30	0.423	7.31	1.06	30.5	N/A	N/A	N/A	N/A	N/A
Alkalinity as $CaCO_3$	0.599	0.462	1.04	0.763	14.3	N/A	N/A	N/A	N/A	N/A
Oil and Grease	0	N.D	N.D	N.D	N.D	N/A	N/A	N/A	N/A	N/A
TEH as Diesel	N/A	N/A	N/A	N/A	N/A	0.00349	0.0097	5.69E-09	5.18E-09	5.50
BOD	0.0509	N/A	N/A	N/A	N/A	0.0857	0.327	1.90E-07	5.75E-07	1.34E-01
COD	0.379	0.256	0.749	0.409	N.D	1.22	1.09	9.70E-07	2.78E-06	5.95E-01
n-Hexane	N.D	N.D	N.D	N.D	N.D	N.D	1.36E-05	N.D	N.D	N.D
Methyl t-Butyl Ether	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D
Benzene	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D
Toluene	N.D	4.60E-07	N.D	2.04E-06	N.D	N.D	N.D	N.D	N.D	N.D
Ethylbenzene	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D
Napthalene	N.D	N.D	N.D	N.D	2.85E-05	N.D	N.D	N.D	N.D	N.D

N/A: No data available.
N.D: Non-detected

Table D.9 Pollutant loads for Site E

Analyte	Pollutant Load (Kg/event)									
	11/10/2008	**5/12/2009**	**7/3/2009**	**7/31/2009**	**9/3/2009**	**3/27/2010**	**5/7/2010**	**5/20/2010**	**7/4/2010**	**9/13/2010**
Antimony (total)	N/A	N/A	N/A	N/A	N/A	6.81E-06	1.24E-04	3.37E-04	N.D	3.12E-05
Arsenic (total)	N/A	N/A	N/A	N/A	N/A	6.81E-06	1.03E-04	3.74E-04	0.00228	4.46E-05
Beryllium (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Cadmium (dissolved)	7.66E-05	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D
Cadmium (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Calcium (dissolved)	1.68	0.681	2.42	1.86	103	N/A	N/A	N/A	N/A	N/A
Calcium (total)	N/A	N/A	N/A	N/A	N/A	0.179	1.72	4.03	16.2	0.422
Chromium (dissolved)	5.66E-05	5.76E-05	6.39E-04	6.68E-04	0.0184	4.46E-05	0.00101	0.00245	0.00747	N.D
Chromium (total)	N/A	N/A	N/A	N/A	N/A	4.54E-05	0.00103	0.00249	0.00761	N.D
Copper (dissolved)	4.66E-04	3.17E-04	1.14E-03	0.00137	0.0225	6.54E-05	0.00149	0.00239	0.00730	1.43E-04
Copper (total)	N/A	N/A	N/A	N/A	N/A	6.81E-05	0.00155	0.00249	0.00761	1.49E-04
Iron (dissolved)	0.00163	1.87E-04	5.39E-04	0.0783	0.0215	N/A	N/A	N/A	N/A	N/A
Lead (dissolved)	7.66E-05	1.08E-05	5.99E-05	4.62E-04	0.00153	1.89E-05	3.23E-04	4.64E-04	0.00421	2.71E-05
Lead (total)	N/A	N/A	N/A	N/A	N/A	2.38E-05	4.08E-04	5.86E-04	0.00533	3.42E-05
Magnesium (dissolved)	0.682	0.148	0.280	0.205	25.6	N/A	N/A	N/A	N/A	N/A
Magnesium (total)	N/A	N/A	N/A	N/A	N/A	0.0343	0.135	0.660	2.47	0.134
Mercury (dissolved)	N/A	8.64E-05	8.79E-04	3.34E-04	0.0112	N/A	N/A	N/A	N/A	N/A
Mercury (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Nickel (dissolved)	N/A	7.92E-05	2.40E-04	0.00120	0.00665	N.D	N.D	N.D	N.D	N.D
Nickel (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Potassium (dissolved)	0.596	0.295	0.899	0.753	86.4	N/A	N/A	N/A	N/A	N/A
Selenium (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Silver (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Sodium (dissolved)	7.72	5.40	12.5	7.36	532	N/A	N/A	N/A	N/A	N/A
Sodium (total)	N/A	N/A	N/A	N/A	N/A	0.883	5.53	18.7	30.7	1.65
Thallium (total)	N/A	N/A	N/A	N/A	N/A	N.D	N.D	N.D	N.D	N.D
Zinc (dissolved)	1.50E-04	N.D	N.D	0.0140	N.D	3.77E-04	0.00455	0.00854	0.0223	5.82E-04
Zinc (total)	N/A	N/A	N/A	N/A	N/A	3.86E-04	0.00465	0.00873	0.0228	5.95E-04
Silica	0.509	0.154	1.02	0.40	N.D	N/A	N/A	N/A	N/A	N/A
Bromide	N.D	N.D	N.D	N.D	N.D	N/A	N/A	N/A	N/A	N/A
Chloride	13.3	10.6	12.8	9.37	632	1.16	7.70	23.1	29.7	1.76
Fluoride	0.0110	0.0184	0.0339	0.0223	0.613	N/A	N/A	N/A	N/A	N/A
Nitrate	0.00766	0.0205	0.0639	0.0745	3.37	N/A	N/A	N/A	N/A	N/A
Nitrite	N.D	N.D	N.D	N.D	N.D	N/A	N/A	N/A	N/A	N/A

N/A: No data available.
N.D: Non-detected

Table D.9 Pollutant loads for Site E (cont.)

Analyte	Pollutant Load (Kg/event)									
	11/10/2008	5/12/2009	7/3/2009	7/31/2009	9/3/2009	3/27/2010	5/7/2010	5/20/2010	7/4/2010	9/13/2010
Nitrate/Nitrite Nitrogen	N/A	N/A	N/A	N/A	N/A	9.08E-04	0.0930	0.0748	0.228	0.0134
Phosphate	N.D	0.0115	0.0459	0.0351	1.18	N/A	N/A	N/A	N/A	N/A
Sulfate	3.48	2.94	6.51	2.85	245	N/A	N/A	N/A	N/A	N/A
Soluble Phosphate	9.99E-04	0.00288	0.0160	0.0115	0.174	N/A	N/A	N/A	N/A	N/A
Phosphorus (dissolved ortho)	N/A	N/A	N/A	N/A	N/A	N.D	0.00413	N.D	0.099	0.00104
Total Phosphorus	0.00866	0.00288	0.0280	0.0107	0.153	5.67E-04	0.0176	0.0212	0.228	0.00312
Total Kjeldahl Nitrogen	N/A	0.0731	0.110	0.0513	4.6	0.0027	0.169	0.118	0.905	0.0248
Total Dissolved Solids	N/A	24.3	109	15.7	2403	3.0	19.2	64.9	97.4	7.62
Total Suspended Solids	N/A	7.20	151	16.3	1564	0.30	6.98	8.48	82.2	0.684
Total Solids	32.2	31.5	261	32.0	3967	N/A	N/A	N/A	N/A	N/A
Volatile Dissolved Solids	N.D	N.D	39.9	3.77	266	N/A	N/A	N/A	N/A	N/A
Volatile Suspended Solids	5.63	3.10	4.79	1.03	92.0	N/A	N/A	N/A	N/A	N/A
Total Volatile Solids	5.66	3.02	44.7	4.79	358	N/A	N/A	N/A	N/A	N/A
Alkalinity as $CaCO_3$	5.26	3.186	13.6	4.71	341	N/A	N/A	N/A	N/A	N/A
Oil and Grease	N.D	0.281	N.D	N.D	N.D	N/A	N/A	N/A	N/A	N/A
TEH as Diesel	N/A	N/A	N/A	N/A	N/A	6.58E-04	0.0522	0.0324	0.0540	8.88
BOD	0.190	N/A	N/A	N/A	N/A	0.0295	1.34	1.12	5.33	0.22
COD	2.14	0.536	9.91	2.48	242	0.440	7.13	7.48	32.7	1.50
n-Hexane	N.D	N.D	N.D	N.D	N.D	N.D	5.17E-05	N.D	0.00152	N.D
Methyl t-Butyl Ether	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D
Benzene	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D
Toluene	N.D	N.D	N.D	8.56E-06	4.09E-04	N.D	N.D	N.D	N.D	N.D
Ethylbenzene	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D	N.D
Napthalene	N.D	N.D	N.D	N.D	2.56E-04	N.D	N.D	N.D	N.D	N.D

N/A: No data available.

N.D: Non-detected.

Appendix E Seed mixtures for Nebraska highways

Different regions in Nebraska are better suited for different grass mixtures. The Nebraska Department of Roads (NDOR) has separated the state into 6 landscape regions, as presented in Figure 38. There is a suggested seed mixture for each region in the following tables. Table 31 shows suggested mix for urban areas, which gives a manicured appearance and can tolerate frequent mowing (NDOR 2010).

Each region has grass mix suggestions for the shoulder region and the foreslope, ditch, backslope areas. The shoulder areas is the area within 16 ft (4.9 m) of the paved surface, and the foreslope, ditch, backslope areas is the area from the shoulder area to the end of the limits of the project (NDOR 2010).

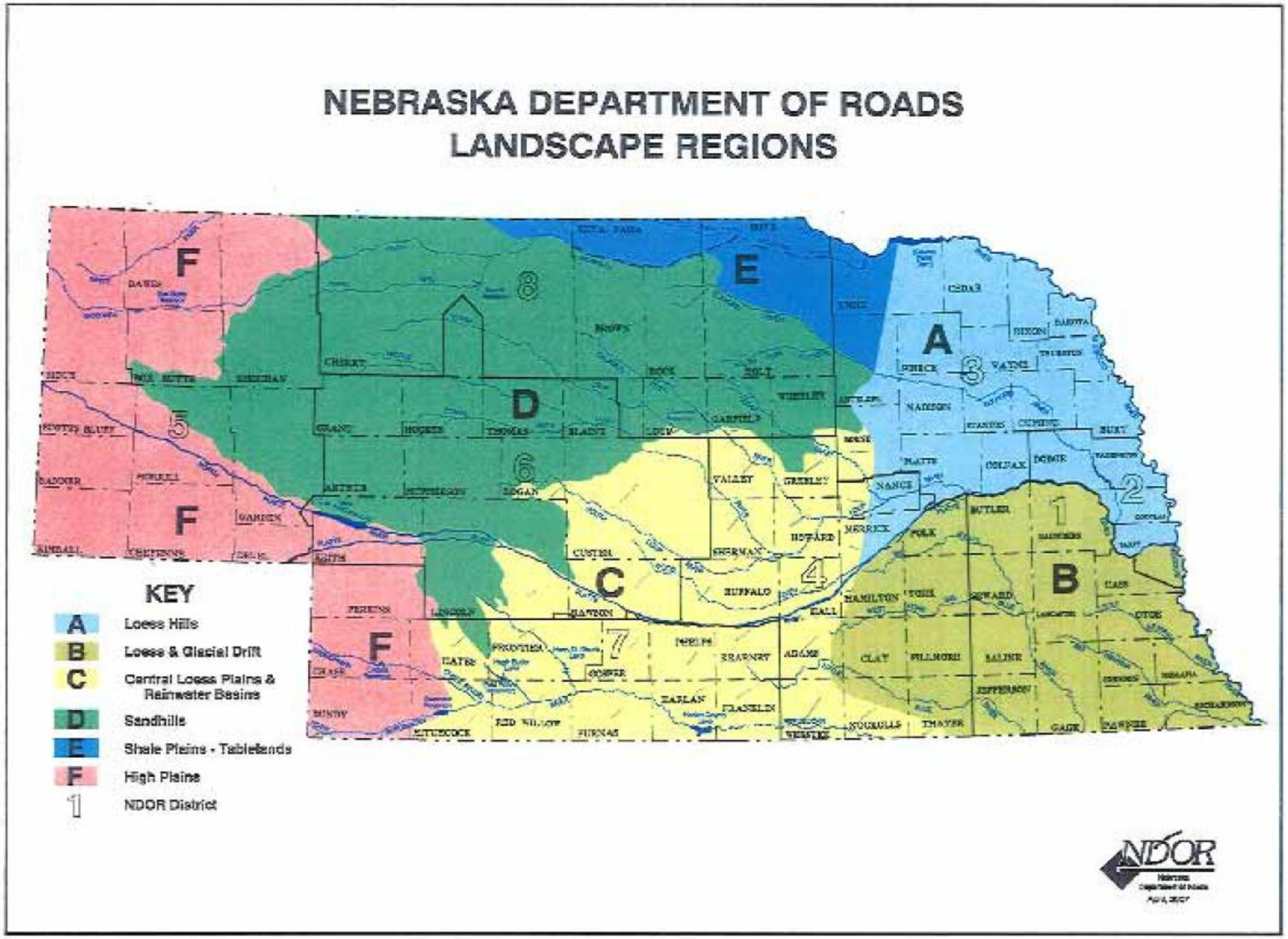
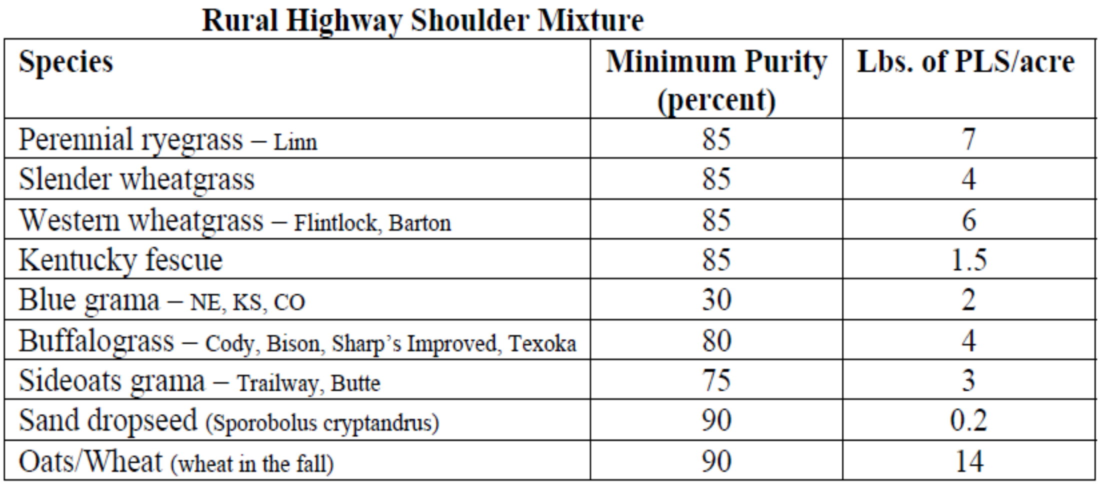


Figure 38 Nebraska Department of Roads landscape regions (NDOR 2010)

E.1 Seed Mixture for Region A: Loess Hills

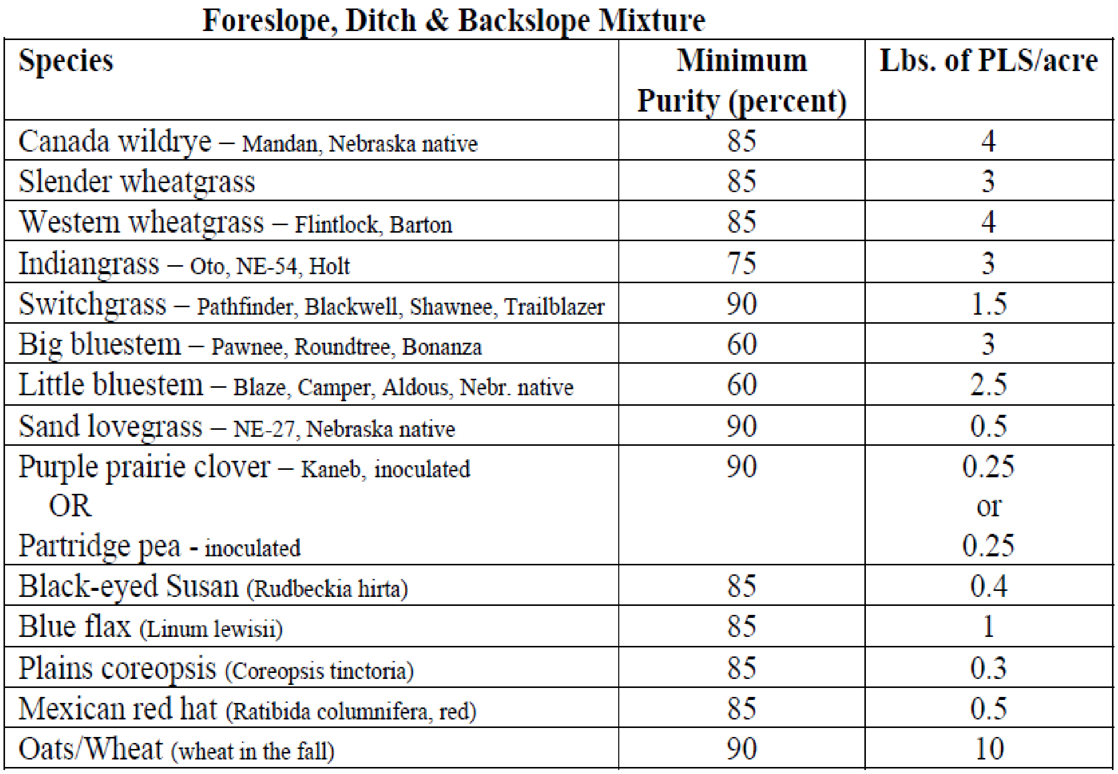
Table 19 Rural highway shoulder mix Region A (NDOR 2010)



Rural Highway Shoulder Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/acre
Perennial ryegrass – Linn	85	7
Slender wheatgrass	85	4
Western wheatgrass – Flintlock, Barton	85	6
Kentucky fescue	85	1.5
Blue grama – NE, KS, CO	30	2
Buffalograss – Cody, Bison, Sharp's Improved, Texoka	80	4
Sideoats grama – Trailway, Butte	75	3
Sand dropseed (Sporobolus cryptandrus)	90	0.2
Oats/Wheat (wheat in the fall)	90	14

Table 20 Grass mixture for foreslopes, ditches, and backslopes for Region A (NDOR 2010)



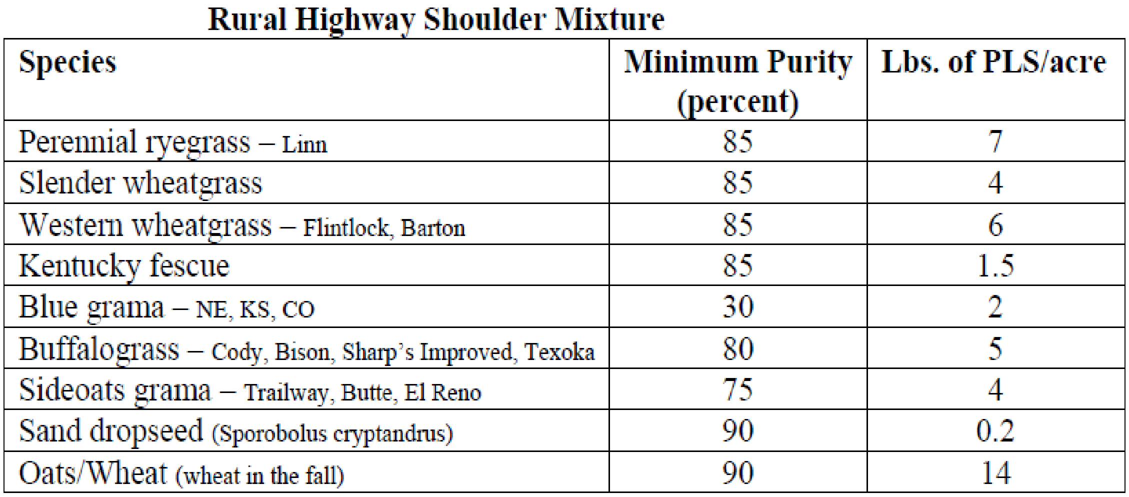
Foreslope, Ditch & Backslope Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/acre
Canada wildrye – Mandan, Nebraska native	85	4
Slender wheatgrass	85	3
Western wheatgrass – Flintlock, Barton	85	4
Indiangrass – Oto, NE-54, Holt	75	3
Switchgrass – Pathfinder, Blackwell, Shawnee, Trailblazer	90	1.5
Big bluestem – Pawnee, Roundtree, Bonanza	60	3
Little bluestem – Blaze, Camper, Aldous, Nebr. native	60	2.5
Sand lovegrass – NE-27, Nebraska native	90	0.5
Purple prairie clover – Kaneb, inoculated OR Partridge pea - inoculated	90	0.25 or 0.25
Black-eyed Susan (Rudbeckia hirta)	85	0.4
Blue flax (Linum lewisii)	85	1
Plains coreopsis (Coreopsis tinctoria)	85	0.3
Mexican red hat (Ratibida columnifera, red)	85	0.5
Oats/Wheat (wheat in the fall)	90	10

PLS (pure live seed) describes the amount of seed that will germinate.

E.2 Seed Mixture for Region B: Loess and Glacial Drift

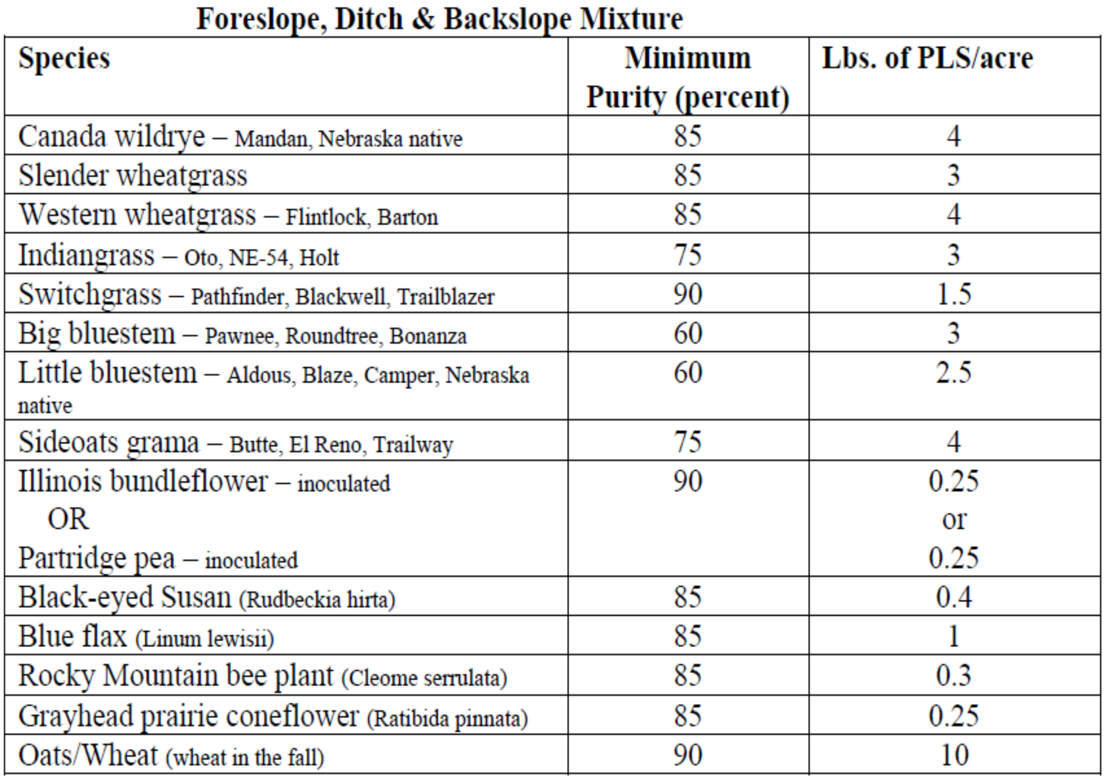
Table 21 Rural highway shoulder mix Region B (NDOR 2010)



Rural Highway Shoulder Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/acre
Perennial ryegrass – Linn	85	7
Slender wheatgrass	85	4
Western wheatgrass – Flintlock, Barton	85	6
Kentucky fescue	85	1.5
Blue grama – NE, KS, CO	30	2
Buffalograss – Cody, Bison, Sharp's Improved, Texoka	80	5
Sideoats grama – Trailway, Butte, El Reno	75	4
Sand dropseed (Sporobolus cryptandrus)	90	0.2
Oats/Wheat (wheat in the fall)	90	14

Table 22 Grass mixture for foreslopes, ditches, and backslopes for Region B (NDOR 2010)



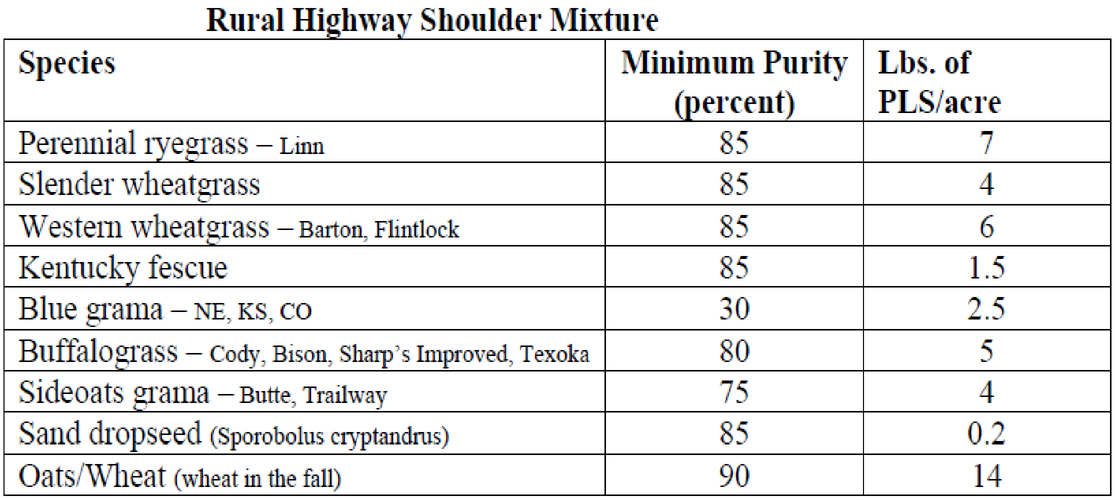
Foreslope, Ditch & Backslope Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/acre
Canada wildrye – Mandan, Nebraska native	85	4
Slender wheatgrass	85	3
Western wheatgrass – Flintlock, Barton	85	4
Indiangrass – Oto, NE-54, Holt	75	3
Switchgrass – Pathfinder, Blackwell, Trailblazer	90	1.5
Big bluestem – Pawnee, Roundtree, Bonanza	60	3
Little bluestem – Aldous, Blaze, Camper, Nebraska native	60	2.5
Sideoats grama – Butte, El Reno, Trailway	75	4
Illinois bundleflower – inoculated OR Partridge pea – inoculated	90	0.25 or 0.25
Black-eyed Susan (Rudbeckia hirta)	85	0.4
Blue flax (Linum lewisii)	85	1
Rocky Mountain bee plant (Cleome serrulata)	85	0.3
Grayhead prairie coneflower (Ratibida pinnata)	85	0.25
Oats/Wheat (wheat in the fall)	90	10

PLS (pure live seed) describes the amount of seed that will germinate.

E.3 Seed Mixture for Region C: Central Loess Plains and Rainwater Basin

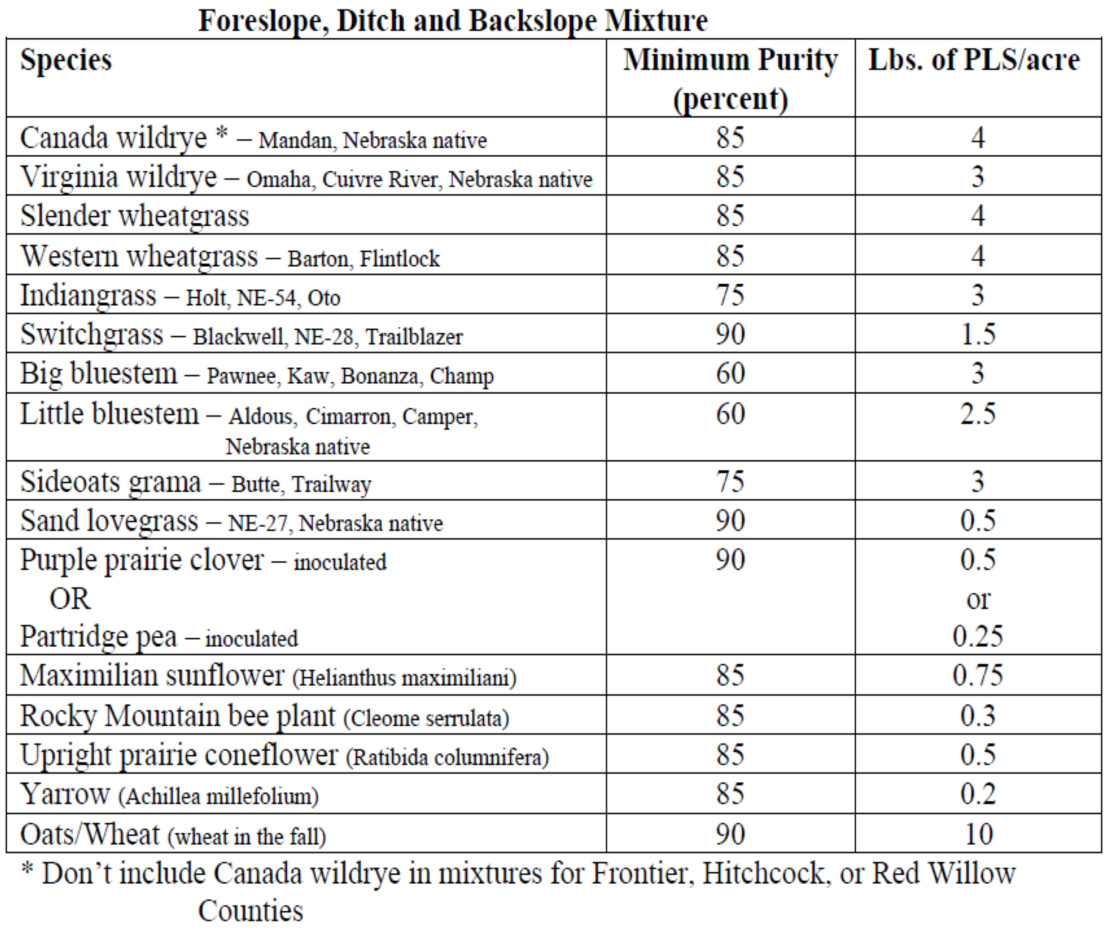
Table 23 Rural highway shoulder mix Region C (NDOR 2010)



Rural Highway Shoulder Mixture

Species	**Minimum Purity (percent)**	**Lbs. of PLS/acre**
Perennial ryegrass – Linn	85	7
Slender wheatgrass	85	4
Western wheatgrass – Barton, Flintlock	85	6
Kentucky fescue	85	1.5
Blue grama – NE, KS, CO	30	2.5
Buffalograss – Cody, Bison, Sharp's Improved, Texoka	80	5
Sideoats grama – Butte, Trailway	75	4
Sand dropseed (Sporobolus cryptandrus)	85	0.2
Oats/Wheat (wheat in the fall)	90	14

Table 24 Grass mixture for foreslopes, ditches, and backslopes for Region C (NDOR 2010)



Foreslope, Ditch and Backslope Mixture

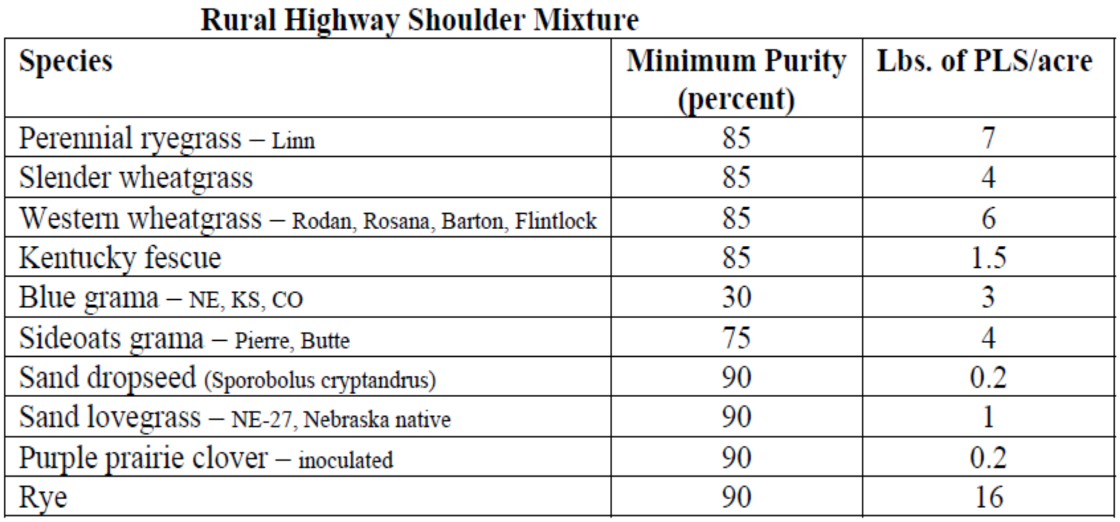
Species	**Minimum Purity (percent)**	**Lbs. of PLS/acre**
Canada wildrye * – Mandan, Nebraska native	85	4
Virginia wildrye – Omaha, Cuivre River, Nebraska native	85	3
Slender wheatgrass	85	4
Western wheatgrass – Barton, Flintlock	85	4
Indiangrass – Holt, NE-54, Oto	75	3
Switchgrass – Blackwell, NE-28, Trailblazer	90	1.5
Big bluestem – Pawnee, Kaw, Bonanza, Champ	60	3
Little bluestem – Aldous, Cimarron, Camper, Nebraska native	60	2.5
Sideoats grama – Butte, Trailway	75	3
Sand lovegrass – NE-27, Nebraska native	90	0.5
Purple prairie clover – inoculated OR Partridge pea – inoculated	90	0.5 or 0.25
Maximilian sunflower (Helianthus maximiliani)	85	0.75
Rocky Mountain bee plant (Cleome serrulata)	85	0.3
Upright prairie coneflower (Ratibida columnifera)	85	0.5
Yarrow (Achillea millefolium)	85	0.2
Oats/Wheat (wheat in the fall)	90	10

* Don't include Canada wildrye in mixtures for Frontier, Hitchcock, or Red Willow Counties

PLS (pure live seed) describes the amount of seed that will germinate.

E.4 Seed Mixture for Region D: Sandhills

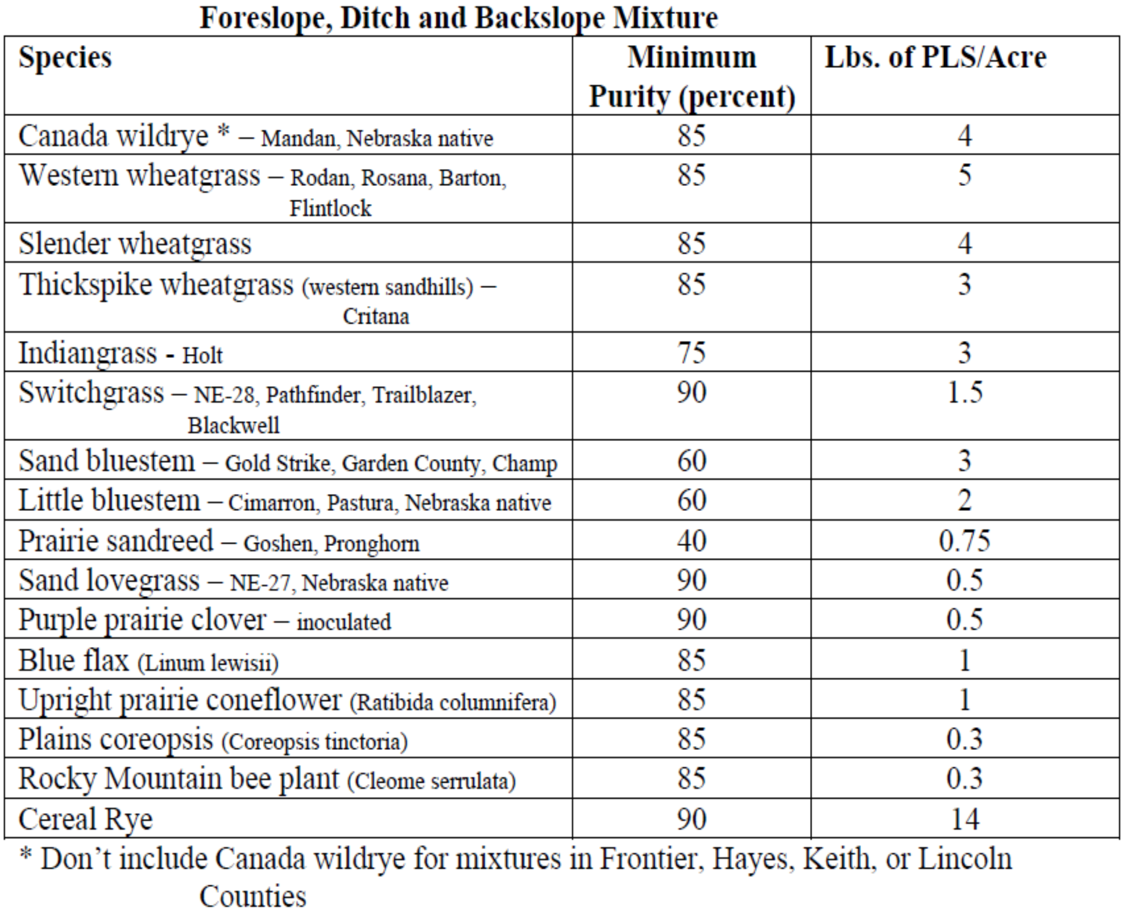
Table 25 Rural highway shoulder mix Region D (NDOR 2010)



Rural Highway Shoulder Mixture

Species	**Minimum Purity (percent)**	**Lbs. of PLS/acre**
Perennial ryegrass – Linn	85	7
Slender wheatgrass	85	4
Western wheatgrass – Rodan, Rosana, Barton, Flintlock	85	6
Kentucky fescue	85	1.5
Blue grama – NE, KS, CO	30	3
Sideoats grama – Pierre, Butte	75	4
Sand dropseed (Sporobolus cryptandrus)	90	0.2
Sand lovegrass – NE-27, Nebraska native	90	1
Purple prairie clover – inoculated	90	0.2
Rye	90	16

Table 26 Grass mixture for foreslopes, ditches, and backslopes for Region D



Foreslope, Ditch and Backslope Mixture

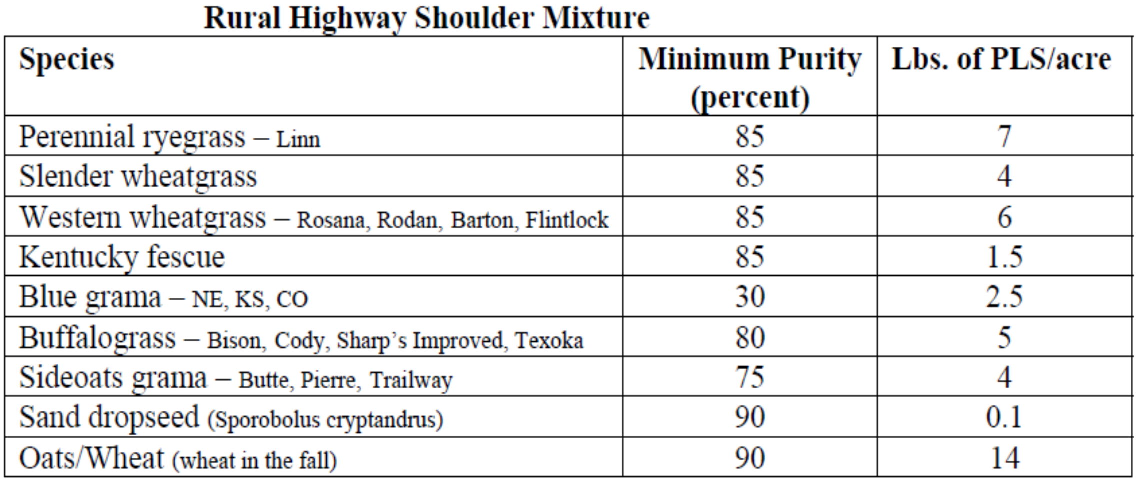
Species	**Minimum Purity (percent)**	**Lbs. of PLS/Acre**
Canada wildrye * – Mandan, Nebraska native	85	4
Western wheatgrass – Rodan, Rosana, Barton, Flintlock	85	5
Slender wheatgrass	85	4
Thickspike wheatgrass (western sandhills) – Critana	85	3
Indiangrass - Holt	75	3
Switchgrass – NE-28, Pathfinder, Trailblazer, Blackwell	90	1.5
Sand bluestem – Gold Strike, Garden County, Champ	60	3
Little bluestem – Cimarron, Pastura, Nebraska native	60	2
Prairie sandreed – Goshen, Pronghorn	40	0.75
Sand lovegrass – NE-27, Nebraska native	90	0.5
Purple prairie clover – inoculated	90	0.5
Blue flax (Linum lewisii)	85	1
Upright prairie coneflower (Ratibida columnifera)	85	1
Plains coreopsis (Coreopsis tinctoria)	85	0.3
Rocky Mountain bee plant (Cleome serrulata)	85	0.3
Cereal Rye	90	14

* Don’t include Canada wildrye for mixtures in Frontier, Hayes, Keith, or Lincoln Counties

PLS (pure live seed) describes the amount of seed that will germinate.

E.5 Seed Mixture for Region E: Shale Plains-Tablelands

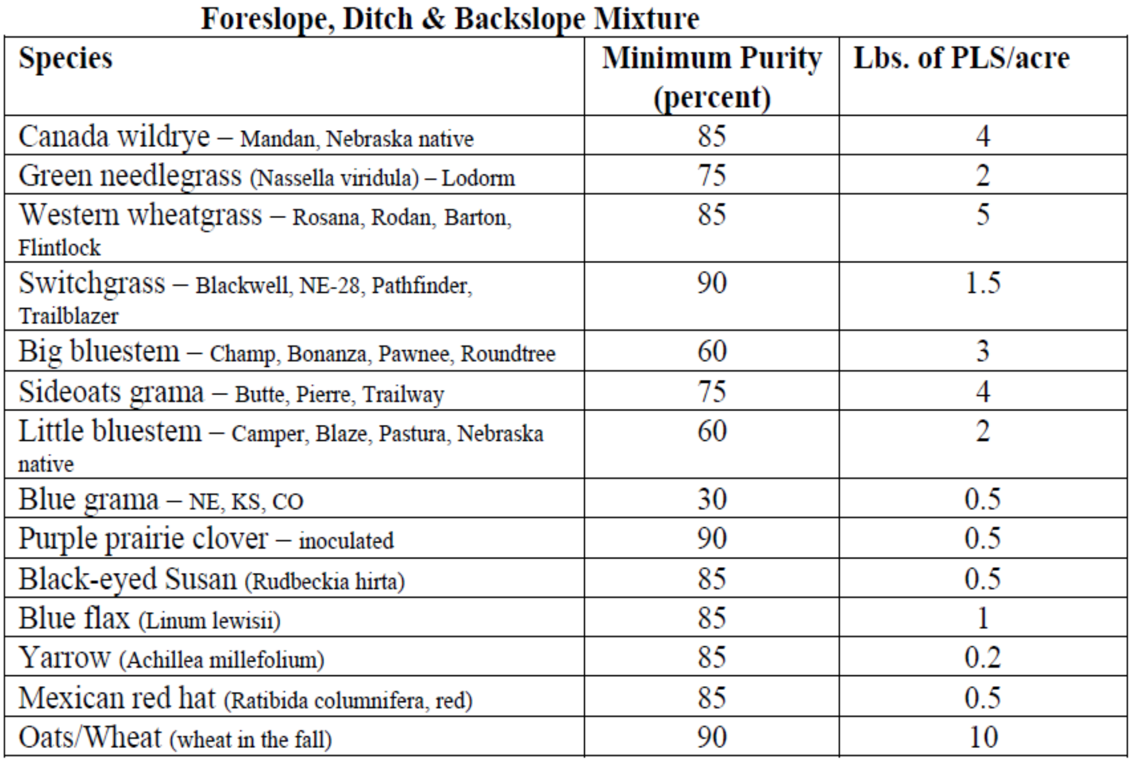
Table 27 Rural highway shoulder mix Region E (NDOR 2010)



Rural Highway Shoulder Mixture

Species	**Minimum Purity (percent)**	**Lbs. of PLS/acre**
Perennial ryegrass – Linn	85	7
Slender wheatgrass	85	4
Western wheatgrass – Rosana, Rodan, Barton, Flintlock	85	6
Kentucky fescue	85	1.5
Blue grama – NE, KS, CO	30	2.5
Buffalograss – Bison, Cody, Sharp's Improved, Texoka	80	5
Sideoats grama – Butte, Pierre, Trailway	75	4
Sand dropseed (Sporobolus cryptandrus)	90	0.1
Oats/Wheat (wheat in the fall)	90	14

Table 28 Grass mixture for foreslopes, ditches, and backslopes for Region E (NDOR 2010)



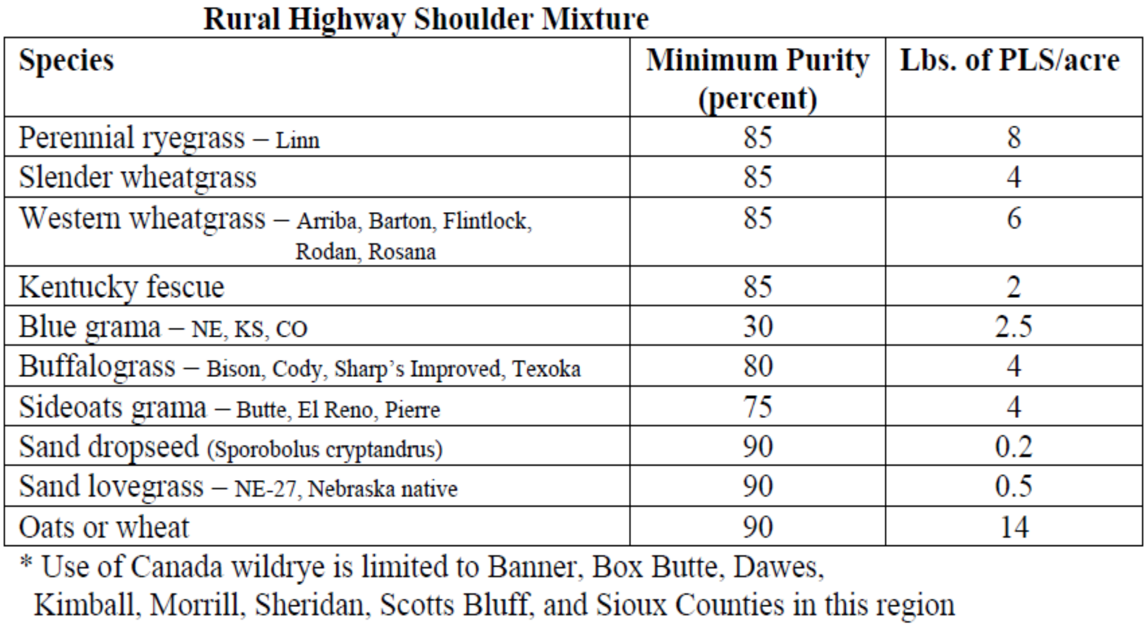
Foreslope, Ditch & Backslope Mixture

Species	**Minimum Purity (percent)**	**Lbs. of PLS/acre**
Canada wildrye – Mandan, Nebraska native	85	4
Green needlegrass (Nassella viridula) – Lodorm	75	2
Western wheatgrass – Rosana, Rodan, Barton, Flintlock	85	5
Switchgrass – Blackwell, NE-28, Pathfinder, Trailblazer	90	1.5
Big bluestem – Champ, Bonanza, Pawnee, Roundtree	60	3
Sideoats grama – Butte, Pierre, Trailway	75	4
Little bluestem – Camper, Blaze, Pastura, Nebraska native	60	2
Blue grama – NE, KS, CO	30	0.5
Purple prairie clover – inoculated	90	0.5
Black-eyed Susan (Rudbeckia hirta)	85	0.5
Blue flax (Linum lewisii)	85	1
Yarrow (Achillea millefolium)	85	0.2
Mexican red hat (Ratibida columnifera, red)	85	0.5
Oats/Wheat (wheat in the fall)	90	10

PLS (pure live seed) describes the amount of seed that will germinate.

E.6 Seed Mixture for Region F: High Plains

Table 29 Rural highway shoulder mix Region F (NDOR 2010)

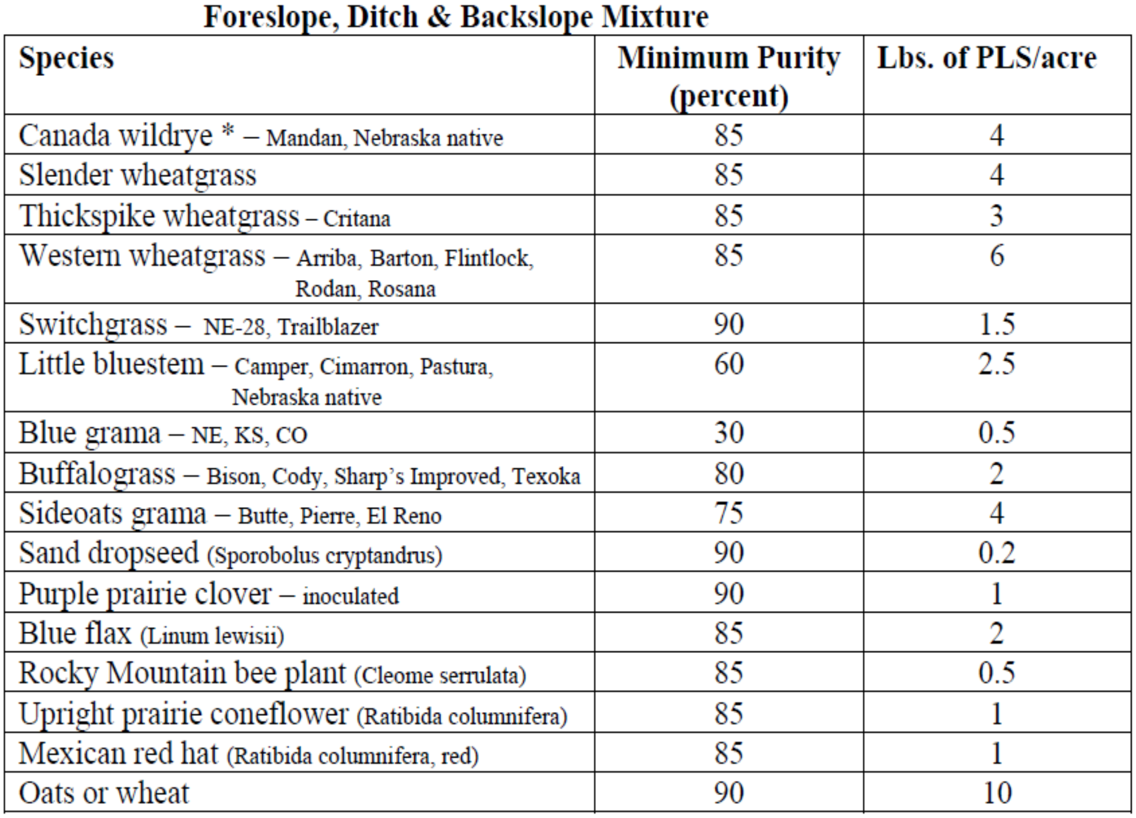


Rural Highway Shoulder Mixture

Species	**Minimum Purity (percent)**	**Lbs. of PLS/acre**
Perennial ryegrass – Linn	85	8
Slender wheatgrass	85	4
Western wheatgrass – Arriba, Barton, Flintlock, Rodan, Rosana	85	6
Kentucky fescue	85	2
Blue grama – NE, KS, CO	30	2.5
Buffalograss – Bison, Cody, Sharp's Improved, Texoka	80	4
Sideoats grama – Butte, El Reno, Pierre	75	4
Sand dropseed (Sporobolus cryptandrus)	90	0.2
Sand lovegrass – NE-27, Nebraska native	90	0.5
Oats or wheat	90	14

* Use of Canada wildrye is limited to Banner, Box Butte, Dawes, Kimball, Morrill, Sheridan, Scotts Bluff, and Sioux Counties in this region

Table 30 Grass mixture for foreslopes, ditches, and backslopes for Region F (NDOR 2010)

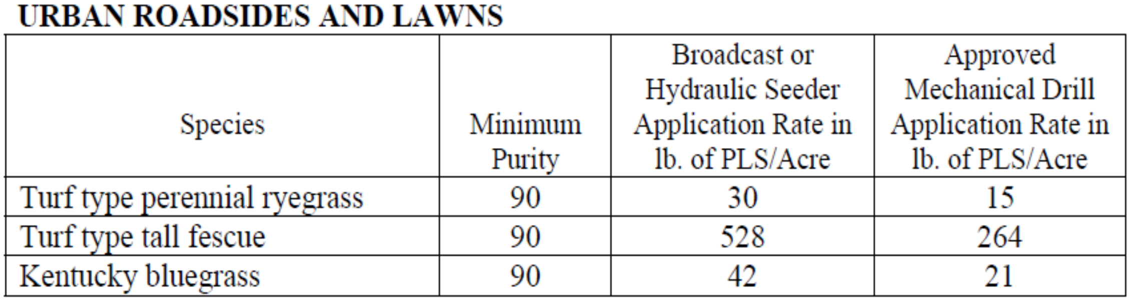


Foreslope, Ditch & Backslope Mixture

Species	**Minimum Purity (percent)**	**Lbs. of PLS/acre**
Canada wildrye * – Mandan, Nebraska native	85	4
Slender wheatgrass	85	4
Thickspike wheatgrass – Critana	85	3
Western wheatgrass – Arriba, Barton, Flintlock, Rodan, Rosana	85	6
Switchgrass – NE-28, Trailblazer	90	1.5
Little bluestem – Camper, Cimarron, Pastura, Nebraska native	60	2.5
Blue grama – NE, KS, CO	30	0.5
Buffalograss – Bison, Cody, Sharp's Improved, Texoka	80	2
Sideoats grama – Butte, Pierre, El Reno	75	4
Sand dropseed (Sporobolus cryptandrus)	90	0.2
Purple prairie clover – inoculated	90	1
Blue flax (Linum lewisii)	85	2
Rocky Mountain bee plant (Cleome serrulata)	85	0.5
Upright prairie coneflower (Ratibida columnifera)	85	1
Mexican red hat (Ratibida columnifera, red)	85	1
Oats or wheat	90	10

PLS (pure live seed) describes the amount of seed that will germinate.

Table 31 Grass mixture for urban roadsides and lawns (NDOR 2010)



URBAN ROADSIDES AND LAWNS

Species	Minimum Purity	Broadcast or Hydraulic Seeder Application Rate in lb. of PLS/Acre	Approved Mechanical Drill Application Rate in lb. of PLS/Acre
Turf type perennial ryegrass	90	30	15
Turf type tall fescue	90	528	264
Kentucky bluegrass	90	42	21

PLS (pure live seed) describes the amount of seed that will germinate.

Appendix F Gradation for common BMP media

Table 32 Gradation for AASHTO M-6 and ASTM C33 sands

	Cumulative Passing by Weight	
U.S. Standard Sieve Size	AASHTO M-6 (Belgard 2012)	ASTM C33 (Division 30 2008)
3/8”	100	100
#4	95 to 100	95 to 100
#8	80 to 100	85 to 100
#16	50 to 85	50 to 85
#30	25 to 60	25 to 60
#50	10 to 30	0 to 30
#100	2 to 10	2 to 10

Table 33 Gradation for AASHTO #3 gravel

	Cumulative Passing by Weight
U.S. Standard Sieve Size	AASHTO # 3 (PROP 2003)
2.5"	
2"	100
1.5"	90 to 100
1"	35 to 70
3/4"	0 to 15
#4	0 to 5
#200	<5