ABSTRACT

BEAN, EBAN ZACHARY. A Field Study to Evaluate Permeable Pavement Surface Infiltration Rates, Runoff Quantity, Runoff Quality, and Exfiltrate Quality. (Under direction of William F. Hunt)

The surface infiltration rates of 48 permeable pavement sites were tested in North Carolina, Maryland, Virginia, and Delaware. Two surface infiltration tests (pre- and post-maintenance) were performed on 15 concrete grid paver (CGP) lots filled with sand. Maintenance consisted of removing the top layer of residual material (13 - 19 mm (0.5 – 0.75 in)). Maintenance significantly (p = 0.007) improved the surface infiltration rate. The median site surface infiltration rate increased from 4.9 cm/h (1.9 in/h) for existing conditions to 8.6 cm/h after simulated maintenance. Fourteen permeable interlocking concrete paver (PICP) and eleven porous concrete (PC) sites were also tested. PICP and PC sites built in close proximity to disturbed soil areas had surface infiltration rates that were significantly (p = 0.0014 and p = 0.0074, respectively) lower than stable landscape sites. Median PICP surface infiltration rates of for each condition were 80 cm/h (31 in/h) and 2000 cm/h (800 in/h), respectively. Median PC surface infiltration rates with and without fines were 13 cm/h (5.1 in/h) and 4000 cm/h (1600 in/h), respectively. This study showed that (1) the location of permeable pavements and (2) maintenance of permeable pavements were critical to maintaining high surface infiltration rates.

Three permeable interlocking concrete pavements (PICP) sites were monitored for runoff quality in North Carolina in Cary, Goldsboro, and Swansboro. The Cary site was located in clay loam soil; 15 samples of exfiltrate and rainfall were analyzed for

pollutant concentrations from February 2004 to November 2004. NH_4-N, PO_4, and Bound Phosphorus (BP) concentrations were significantly ($p \leq 0.05$) lower in exfiltrate than collected rainfall. The Goldsboro site was constructed in 2002 to compare the water quality of asphalt runoff to exfiltrate of adjoining permeable pavement. Up to 14 samples of Zn, NH_4-N, TKN, NO_{2+3}-N, TP, and Cu were collected from the asphalt runoff and the permeable pavement exfiltrate from July 2003 to November 2004. The Swansboro site was constructed in 2003 and instrumented to monitor runoff flow, record rainfall rates, and collect water samples from PICP exfiltrate and runoff from March 2004, to December 2004; however, during the entire monitoring period, precipitation on the 740 m^2 parking lot produced no runoff.

Permeable pavement surface infiltration rates presented herein were compared to grassed lawn infiltration rates from other studies. Ninety percent of permeable pavement surface infiltration rates in sandy soils were greater than 5.4 cm/h (2.1 in/h), compared to 6.4 cm/h (2.5 in/h) for grassed sandy loam lawns in urban soils. Rational coefficients and Curve Numbers were determined for monitored permeable pavement sites in eastern North Carolina and compared to those of grassed lawns. Comparable ratios of impermeable surface to grassed lawns were also determined. These comparisons were used to assign a suggested percent perviousness to be given to permeable pavements. A credit could then be developed for permeable pavements related to grassed lawns in sandy soils. The credit could be an equivalent ratio of grassed lawn area to impervious area for permeable pavements based on runoff volumes.

A FIELD STUDY TO EVALUATE PERMEABLE PAVEMENT SURFACE INFILTRATION RATES, RUNOFF QUANTITY, RUNOFF QUALITY, AND EXFILTRATE QUALITY

by

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A thesis submitted to the Graduate Faculty of
North Carolina State University
In partial fulfillment of the
Requirements for the Degree of
Master of Science

BIOLOGICAL AND AGRICULTURAL ENGINEERING

Raleigh, North Carolina

2005

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Quotes as Impressions

Don’t let school get in the way of your education.
I was tryin’ to figure out what you were doing,……but I couldn’t..
You wanna fight a bull!?!

Thanks

Thanks for helping me keep the sanity
Thanks for embracing my lunacy
Thanks for being difficult when I was easy
Thanks for being easy when I was difficult
Thanks for pushing me harder everyday, even when that meant resting
Thanks for not making life boring
Thanks for taking chances on me
Thanks for letting me dream, but reminding me of reality
Thanks for reminding me of the big picture
Thanks for all the love that I never deserved
Thanks for teaching me each time we spoke

Without struggle there is no value in overcoming

I don't have it all figured out, but I'd like to think I'm getting better at it.

BIOGRAPHY

Eban Zachary Bean was born in Goldsboro, North Carolina, on June 12, 1981. He is the son of David Alan and Pamela Safrit Bean of Mount Olive, North Carolina, and the younger of two children. His father works at Southern Bank and Trust while his mother is retired from Wayne County Public Schools. Eban lived his entire life in Mount Olive before attending North Carolina State University. He majored in Biological Engineering and received his bachelor's degree in May 2003. Eban is a third generation graduate of NCSU, preceded by his father and grandfather. As an undergraduate Eban interned with the NCSU Water Quality Group, working with stream restoration surveys and designs. Upon graduation, Eban continued his studies in the Biological and Agricultural Engineering Department at North Carolina State University towards his Master's Degree. He will begin work toward a Ph.D. in the fall of 2005 at the University of Florida in the Agricultural and Biological Engineering Department studying low-impact development.

ACKNOWLEDGEMENTS

The author would like to acknowledge many of the people who contributed to the success of this research project. The author would like to thank his advisor, Dr. Bill Hunt, for without his guidance, this research project would not have become a reality.

The author would also like to thank all those who worked directly on the field research sites. Many thanks go to Brandon Eckrote, Lucas Sharkey, Zach Woodward, and Jon Hathaway for enduring extreme heat while testing surface infiltration rates. The assistance of the Town of Swansboro Public Works employees who collected samples was vital to the success of this project. In addition, thanks goes to Dr. Ralph Smith and his AP Chemistry class at Eastern Wayne High School for collecting samples from Goldsboro. This research would have been impossible without the help of Mr. Jonathan Smith, who always stopped to assist any issue. Mr. Dave Bidelspach was critical to initiating this project.

The author would like to thank all his friends and family who repeatedly offered up support and assistance. Special thanks again to Dr. Bill Hunt for providing the research and serving not only as the advisor of the author, but also as a mentor and a friend. Special thanks also go to Ms. Danielle Ericksen who endured the author and supported him unwavering through this project. Most of all, the author thanks his God, Savior, and Provider, for blessing him with the talents and abilities to complete this project. All things can be done through Him.

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CHAPTER 1

A FIELD SURVEY OF PERMEABLE PAVEMENT SURFACE INFILTRATION RATES

ABSTRACT

The surface infiltration rates of 48 permeable pavement sites were tested in North Carolina, Maryland, Virginia, and Delaware. Two sets of surface infiltration tests (pre- and post-maintenance) were performed on 15 concrete grid pavers (CGP) lots filled with sand. Maintenance was simulated by removing the top layer of residual material (13 - 19 mm (0.5 – 0.75 in.)). Maintenance significantly ($p = 0.007$) improved the surface infiltration rate. The median site surface infiltration rate increased from 4.9 cm/h (1.9 in/h) for existing conditions to 8.6 cm/h (3.4 in/h) after simulated maintenance. Fourteen permeable interlocking concrete paver (PICP) and eleven porous concrete (PC) sites were also tested. PICP and PC sites built in close proximity to disturbed soil areas had surface infiltration rates significantly ($p = 0.0014$ and $p = 0.0074$, respectively) less than sites with stable sediment contributing areas. Median PICP surface infiltration rates of each condition were 80 cm/h (31 in/h) and 2000 cm/h (800 in/h), respectively. Median PC surface infiltration rates with and without fines were 13 cm/h (5.3 in/h) and 4000 cm/h (1600 in/h), respectively. This study showed that (1) the location and (2) maintenance of permeable pavements were critical to maintaining high surface infiltration rates.

INTRODUCTION

Permeable pavements are an alternative to traditional impermeable asphalt and concrete surfaces. Permeable pavements allow stormwater to either infiltrate into an underground storage basin or exfiltrate to the soil and ultimately recharge the groundwater, while also potentially removing pollutants (US EPA, 1999). Urbanization has a detrimental effect on surface waters. Increased runoff rates from impervious surfaces have increased peak flow through stream channels, causing erosion and stream bank instability (Leopold, et al, 1964). Runoff from impervious surface areas carries pollutants, such as sediments, nutrients, and heavy metals, into surface waters. To reduce the effects of urbanization, state and local governments in North Carolina and throughout the United States have established regulations for stormwater management for new development and redevelopment (U.S. EPA, 2000). One stormwater management option is to minimize the amount of a project's impervious surface by utilizing permeable pavements (Bradley Bennett, personal communication, November 3, 2003). As a result, the use of permeable pavements is poised to grow.

Like many states, North Carolina has implemented a stormwater credit system for developed sites to manage onsite runoff (NC DENR, 1997). Several Best Management Practices (BMPs) were assigned credits for nutrient reduction, sediment reduction, and peak flow mitigation. At this time, permeable pavements haves not been assigned BMP credit because they are prone to clogging. However, regulators in North Carolina allow the use of permeable pavement as an "innovative BMP," (Bradley Bennett, personal communication, November 3, 2003) which requires monitoring on an individual site basis to assess performance (NC DENR, 1995). Few landowners have been willing to conduct the required monitoring, thus limiting the number of state approved permeable pavement installations.

Some recent studies have found that permeable pavements reduce runoff and improve water quality. The use of permeable pavement, in place of traditional asphalt or concrete, has been shown to decrease surface runoff volumes and substantially lower peak discharge (Pratt, et al, 1995; Booth, et al, 1996; Rushton, 2001; Hunt, et al, 2002). Permeable pavements have also been shown to filter pollutants such as metals and automotive oil (Brattebo and Booth, 2003; Pratt, et al, 1995; Rushton, 2001).

Figures 1a, 1b, and 1c show examples of Concrete Grid Pavers (CGP), Permeable Interlocking Concrete Pavers (PICP), and Porous Concrete (PC). Photoanalysis, a procedure that uses close-up photographs of surfaces was applied to determine the percent of a permeable pavement surface area that was impermeable due to the pavement block. The remaining surface area was considered to be the open or void area. CGP paving systems are comprised of concrete blocks with internal voids and gaps between the blocks. Photoanalysis determined that CGP surface was approximately 30% open, or void. In this study, the sites examined had voids filled with either sand or No. 78 stone, ASTM D448 (ASTM, 2003b). PICP are concrete block pavers that, when installed, have voids located at the corners and midpoints of the individual pavers. Photoanalysis determined that a PICP surface was at least 9% open, or void. Most previous and recent research conducted on permeable pavements focused on PICP systems (Balades et al., 1995; Pratt et al., 1995; Gerritts and James, 2002). PC is different from standard concrete, in that fine aggregate has been removed from the mix, allowing interconnected void spaces to form during curing.



Figure 1a. PICP **Figure 1b. CGP** **Figure 1c. PC**

Pratt, et al, (1995) found that clogging can result from fine particle accumulation in void spaces of permeable pavements. Smaller particles trap larger particles; therefore, the rate of clogging increases as larger fines are trapped (Balades et al., 1995). However, clogging can be reduced by regular maintenance, either by vacuum sweeper or pressure washing (Balades et al., 1995). Removing the top 15 – 20 mm (0.6–0.8 in.) of void space material for low to medium traffic areas substantially regenerated infiltration capacity. Permeable pavements in higher traffic areas improved when 20–25 mm (0.8–1.0 in.) of material was removed (Gerrits and James, 2002).

The goals of this study were to: (1) determine surface infiltration rates of each pavement type; (2) compare and evaluate infiltration rates by pavement type; (3) analyze whether maintenance restored surface infiltration rates on CGP; (4) determine if pavement location impacted surface infiltration rates for PICP and PC; and (5) offer basic siting guidelines based upon results.

METHODS AND MATERIALS

Fifteen CGP, 14 PICP, and 11 PC sites were tested to determine surface infiltration rates. Either double-ring infiltrometers, single-ring infiltrometers, or combinations were used to measure surface infiltration rates at each site. At most CGP sites, two sets of tests were conducted: the first set measured the surface infiltration rate of existing pavement conditions, while in different locations, the second set measured surface infiltration rates of locations where simulated maintenance had been performed. Each set of tests included three surface infiltration tests conducted at different locations on the pavement to address variability of surface conditions and associated surface infiltration rates of the permeable pavement. By visually evaluating a site, locations for these tests were chosen to be representative of the entire surface (i.e., potentially low, medium, and high surface infiltration areas were selected for testing).

ASTM D 3385 (ASTM, 2003a), the "Standard Test Method for Infiltration Rate in Field Soils Using Double-Ring Infiltrometer", was the procedural basis for measuring surface infiltration rates. This test measures infiltration rates for soils with a hydraulic conductivity between 10^{-6} cm/s and 10^{-2} cm/s. The test used for this study modified some of the methods and materials in ASTM D 3385 (ASTM 2003a) to operate on the unique pavement environments (hard pavement) and with a limited supply of water. The double-ring infiltrometers utilized consisted of two 16 gauge ("thickness") galvanized steel rings. The inner rings had diameters between 280 mm (11 in.) and 305 mm (12 in.). The outer rings had diameters between 760 mm (30 in.) and 910 mm (36 in.), or approximately three times the diameter of the inner rings. The single-ring infiltrometer method utilized only the inner rings.

Once locations were selected for testing at each site, the inner ring was sealed to the test surface. A thin ribbon of putty, about 40 mm (1.5 in.) wide, was molded along the bottom edge of the inner ring. The ring was then placed putty side down and pressed to the surface. The putty was depressed to form a tight seal between the surface and the ring. The inner ring was then filled with water to a depth of approximately 50 mm (2 in.) above the testing surface to determine if any leakage to the outer ring existed, and whether hydraulic head would be maintainable during a Double-Ring Infiltrometer Test (DRIT). A hydraulic head was determined to be maintained if the water level rose while dispersing water into the rings using a submersible pump with a maximum flow of 1.6 l/s (25 gal/min). If hydraulic head was maintained during the trial, then a DRIT was conducted on the surface. The outer infiltrometer ring was sealed to the surface using putty in the same manner as the inner ring. The outer ring was then filled to a depth of approximately 50 mm (2 in.) above the testing surface to determine if any leakage from the outer ring formed that could not be compensated. Figure 2 shows three DRITs conducted simultaneously.

Figure 2. Typical arrangement and setup of double-ring infiltrometers at a site.

Once all leaks, if any, were plugged or manageable, both the inner and outer rings were filled to a depth between 125 mm (5 in.) and 175 mm (7 in.). The initial level of water in the inner ring, outer ring, and current time (effectively time 0) were recorded. All three parameters were measured and then recorded approximately every five minutes. Each water level measurement (inner and outer) was taken from the top of the inner ring to the water level from the same location along the rim for each measurement. A test was complete when enough time, typically between 30 and 45 minutes, had elapsed to determine the surface infiltration rate. Tests at all sites were preceded by a dry period of at least 24-hour.

One goal of this study was to compare existing condition surface infiltration rates to simulated maintained condition surface infiltration rates for concrete grid pavers (CGP). At each CGP site three tests were conducted under existing conditions. After simulated maintenance three additional tests were conducted in different locations. An “existing” test was defined to be a surface infiltration test where the paver surface remained unaltered prior to the surface infiltration test. A “simulated maintenance” test was a surface infiltration test conducted where between 13 mm (0.5 in.) and 19 mm (0.8 in.) of void material was removed to simulate maintenance by a street sweeper (Stevens, 2001). Figure 3 displays a maintained CGP location. If the measured existing surface infiltration rates of a site were lower than 25 cm/h (10 in/h), a simulated maintenance test was conducted.

Figure 3. CGP surface after simulated maintenance was performed.

Many sites had surface infiltration rates greater than the filling rate for the DRIT (>150 cm/h (60 in/h)). A modified version of the DRIT, the single-ring infiltrometer test (SRIT), was performed at these sites. When conducting the SRIT, an inner ring of the double ring infiltrometer was sealed to the test surface and a scale was vertically taped inside the ring (Figure 4). Using a 19 L (5 gal) container, water was quickly poured into the sealed inner ring; time was recorded from the moment water started entered the inner ring. The time was then recorded when the container was completely emptied (along with the peak level of water inside the ring), and again every 30 to 60 seconds until the water completely infiltrated the pavement. If complete infiltration occurred in less than 30 seconds, the time of complete infiltration was recorded. The test was then repeated at the same location and the two rates would then be averaged. The mean for that location was then grouped with the surface infiltration rates of the other two locations tested at the paver site and averaged to determine an overall surface infiltration rate for that site. The SRIT was neither as accurate nor as precise as the double-ring-infiltrometer test, because the SRIT did not prevent horizontal migration of the water once it entered the pavement surface.

However, it provided a method for relatively quantifying the surface infiltration rate on highly pervious surfaces.

After data were collected, the water levels were plotted as functions of time for each surface infiltration test (Figure 4). The infiltration rate is equivalent to the maximum-steady state or average incremental infiltration velocity (ASTM, 2003a). Therefore, the slope of the least squares line for each test was determined to be equal to the surface infiltration rate of the tested surface. Furthermore, if it was determined that removing the initial two or three water level and time recordings from a test's dataset caused the least squares line to be more representative of the surface infiltration rate, then those initial data points were omitted from the calculation of the least squares line. The stated surface infiltration rate for each site was determined by averaging the results from the three test locations.

Infiltration rate is defined as the rate of "water entry into the soil (or pavement) surface", while hydraulic conductivity is "a function of the effective diameter of the soil pores and dynamic viscosity of the fluid" (Schwab, et al, 1993). Infiltration rate is simply a characteristic of a soil or pavement surface, while hydraulic conductivity is a term that can be used to indicated how quickly water can pass through a particular set of underlying soils. Therefore, while infiltration rate is dependent on the hydraulic conductivity of the media being infiltrated, hydraulic conductivity is not dependent on the infiltration rate, but rather soil properties. Determining the hydraulic conductivity of the layers comprising each pavement site would have been a valuable addition to this study. ASTM D3385 states that hydraulic conductivity and infiltration rate "cannot be directly related unless the hydraulic boundary conditions are known or can be reliably estimated" (ASTM, 2003a). This would have required monitoring of soil water conditions during testing, which may not have been possible due to preservation of the pavement surface integrity, and the types of equipment needed. As a result since boundary conditions were unknown, hydraulic conductivities could not be determined from the data collected in this study.

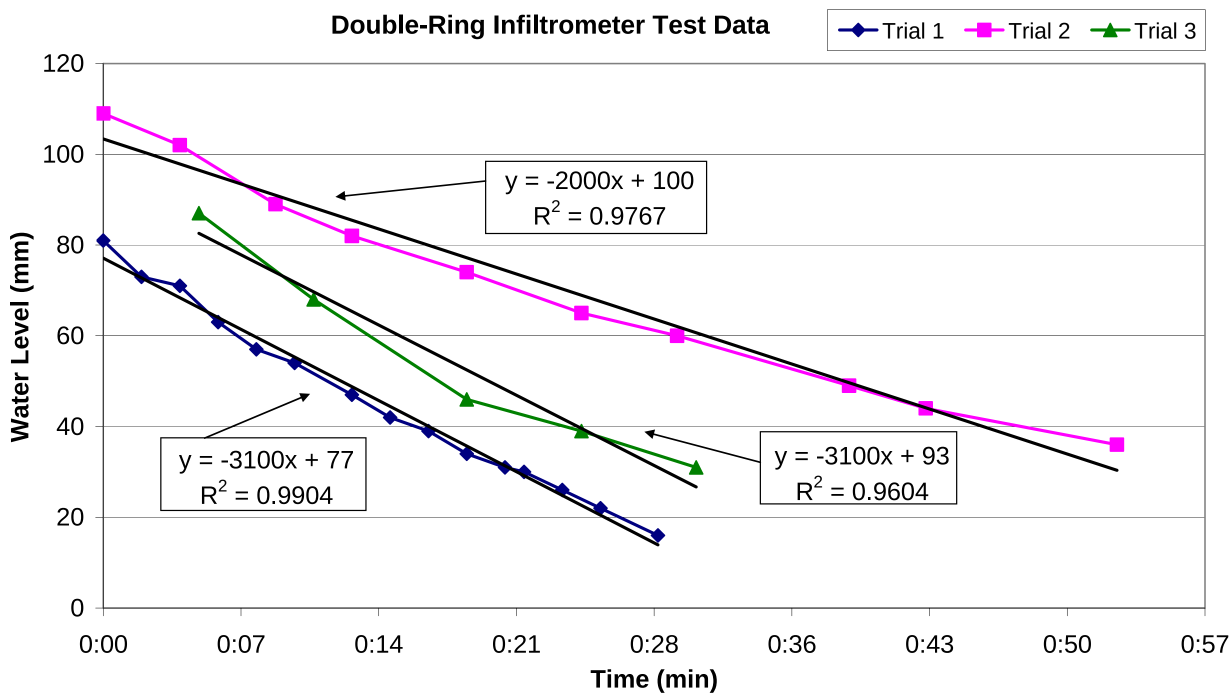

Figure 4. Typical graph of double ring infiltrometer inner ring depths with corresponding times from three surface infiltration tests at one site. Linear regression lines were applied to the data to determine the surface infiltration rate. data and regression lines.

Two major limitations differentiated the procedure followed in this study from ASTM D 3385 (2003a). In principal, the ASTM method should be applied to "field measurement of the rate of infiltration of liquid into soils using double-ring infiltrometer" (ASTM 2003a). The media tested in this study was not soil, but a combination of various permeable pavement types and corresponding coarse grained fill material. Although the testing media differed, the hydraulic conductivities of all tests were within ASTM testing limitations, assuming that surface infiltration rates were surrogates for the hydraulic conductivity (ASTM, 2003a). Since permeable pavements were tested rather than soil media, infiltrometer rings were not driven into the pavements, so surface infiltration occurred below the bottom of the rings instead of above. Pavement surfaces would have been damaged and a substantial amount of additional time, resources and effort would be needed to drive the infiltrometer rings into the surfaces; this was not practical.

Rather than employ a constant head, this procedure used a falling head to determine surface infiltration rates. This resulted from a water resource constraint. A limit of 1100 L (300 gal) could be transported at a given time, which would have required additional time for refilling and transporting for a constant head study. By using a falling head procedure, the volume of water needed for all studies was substantially less than volumes necessary to maintain a constant head. As a result of using a falling head test, surface infiltration rates were variable during a test. However, R^2 values for water depth versus time relationships were nearly all greater than 0.9 and the majority met or exceeded 0.99. The variation of the surface infiltration rate compared to the average surface infiltration rate was minimal; therefore, the data here were comparable to a constant head surface infiltration test.

RESULTS

The findings for all three pavement types were reviewed. Specific site information for each site and surface infiltration test data can be found in Appendix A. The CGP tests examined whether simulated maintenance had a significant impact on surface infiltration rates. Tests conducted on PICP and PC sites were used to determine whether siting permeable pavements adjacent to disturbed soil, a potential source of fines, had a significant effect on surface infiltration rates.

Concrete Grid Pavers

Surface infiltration rates were measured from 15 CGP sites (Table 1). Each of the 15 sites had both existing and post-maintenance tests conducted on them (Appendix A). Of the 15 sites that had post- maintenance tests, 14 had greater surface infiltration rates than those of the existing, non-maintained, pavers. The only site that the maintained surface infiltration rate was not greater was Blackman Beach Access. This anomaly was likely due to a high surface infiltration rate of 22 cm/h (8.8 in/h) for an existing test (Appendix A), while the other tests at Blackman Beach Access, existing and maintained, had surface infiltration rates less than 10 cm/h (3.9 in/h). Simulated maintained surface infiltration rates were significantly ($p = 0.007$) higher than rates for existing surface conditions (SAS™, 2003). The median existing surface infiltration rate was 4.9 cm/h (1.9 in/h), while the median maintained surface infiltration rate was 8.6 cm/h (3.4 in/h). The median surface infiltration rate after simulated maintenance was 76% greater than the existing median surface infiltration rate.

The lowest surface infiltration rate, measured at the Town of Cary Public Works (1.0 cm/h (0.38 in/h)), could have resulted from several factors, including: no maintenance, frequent heavy traffic, and/or a clay soils in its sediment contributing area.

Table 1. CGP average surface infiltration rates (SIR) and average R^2 values for pre- and post- maintenance.f

	Existing		Maintained	
Site Name	**Avg. SIR (cm/h)**	**Avg. R^2**	**Avg. SIR (cm/h)**	**Avg. R^2**
Atlantic Station (High)	19	0.99	32	0.99
Indian Beach Access	16	0.996	27	0.99
Blackman	13	0.99	6.7	0.99

Conch	9.2	0.99	10	0.97
Municipal Building	7.9	0.99	27	0.99
Gull	5.1	0.97	7.3	0.99
Glidden	5.0	0.99	7.5	0.99
Carrabba's	4.9	0.98	7.5	0.99
Govenor	4.6	0.97	8.6	0.97
Atlantic Station (Low)	4.4	0.995	31	0.98
Epstein	4.2	0.92	9.7	0.99
Bainbridge	4.2	0.99	4.6	0.99
Loggerhead	3.6	0.99	9.3	0.98
Hargrove	1.7	0.97	6.5	0.95
Cary Public Works	1.0	0.88	1.6	0.93
Median	**4.9§**		**8.6§**	

§ The difference in surface infiltration rate pre- and post- maintenance was significant. ($p=0.0070$, $df=1$)

Permeable Interlocking Concrete Pavers

Fourteen PICP sites were tested including: seven in Maryland, four in North Carolina, two in Virginia and one in Delaware (Appendix A). Eight sites were tested using only the SRIT, due to high surface infiltration rates (>150 cm/h (60 in/h)). A hydraulic head was maintained at Havre de Grace when filling the double-ring infiltrometers for testing, but the mean surface infiltration rate was 100 cm/h (39 in/h). One of the three tests run at the Penny Road site was an SRIT; however, the other two were DRITs. The Penny Road site was a determined to be affected by fines as clay accumulation (a result of on-going construction) was observed in the void spaces. Surface infiltration rates at the four remaining PICP sites were slow enough to maintain a hydraulic head so that DRITs could be performed. These four sites were located in close proximity to areas containing exposed and transportable soil particles, e.g., a gravel drive, a river bed, or a beach. Table 2 shows measured surface infiltration rates for permeable pavement systems using PICP. The last five surface infiltration rates in Table 2 are the PICP sites which had surfaces partially filled by fine soil particles. Surface infiltration rates of sites located adjacent to disturbed soils, or that had fines deposited on them, were significantly ($p = 0.0014$) lower than those rates from sites free of fines. The median surface infiltration rate for sites affected by fines was 8.0 cm/h (3.1 in/h); the median surface infiltration rate for sites away from fines was 2000 cm/h (900 in/h). There were three orders of magnitude difference and an overall decrease of more than 99% when comparing the median surface infiltration rates of stable sites to the median of sites affected by fine soil particulates. The surface infiltration rates of sites impacted by fines (sand) were very comparable to those of CGP filled with sand reviewed earlier.

Table 2. PICP average surface infiltration rates (SIR) and average R^2 values.

Site Name	Average SIR (cm/h)	Average R^2
Without Fines		
Mickey's Pastries	4000	N/A
CVS Pharmacy	4000	N/A
Wal-Mart	3000	N/A
Dough Rollers	2000	N/A
Swansboro	2000	0.96
Captiva Bay Condos	2000	0.97
PNMC Walkway	1000	0.98
Baywoods	1000	0.97
Harve de' Grace	100	0.98
Median	**2000§**	
With Fines		
Penny Road PICP	200	0.99
PNMC Parking Lot	50	0.98
River Bend	8.0	0.97
Boat Ramp	2.9	0.90
Somerset Dr.	1.6	0.99
Median	**8.0§**	

§ The difference in surface infiltration rate when PICP was sited in stable versus disturbed watersheds was significant. ($p = 0.0014$, $df = 1$)

Porous Concrete

The surface infiltration rates were tested for eleven PC sites located in the Piedmont and Coastal Plain of North Carolina (Table 3). Surface infiltration rates were high enough at five sites so that only SRITs could be performed (>150 cm/h (60 in/h)). A combination of SRITs and DRITs was used to determine a site’s surface infiltration rate at the Ready Mix Lab, and Bailey’s Landing I sites. Surface infiltration rates were low enough to maintain a hydraulic head at the four other PC sites so that DRITs were used. The four sites where DRITs were used, like the PICP sites, were located in areas that accumulated fine soil particles, e.g., receiving wind blown particles near beaches or deposition of soil particles from vehicular traffic. The first seven sites in Table 3 were relatively free of fines, while the last four had visual evidence of sediment deposition on the surface. The infiltration rates of the last four sites (with fines) were significantly lower ($p = 0.0074$) than rates of the first seven. The median surface infiltration rates for sites with fines was 13 cm/h (5.3 in/h); the median surface infiltration rates for sites free of fines was 4000 cm/h (2000 in/h). The median surface infiltration rates for stable sediment contributing areas were two orders of magnitude greater.

Table 3. PC average site surface infiltration rates (SIR) and average R^2 values.

Site Name	Average SIR (cm/h)	Average R^2
Without Fines		
Catawba College	7000	N/A
Loflin Concrete	6000	0.94
Bailey's Landing II	6000	0.97
Penny Rd. PC	4000	0.96
FCPR PC	2000	0.97
Ready Mix Lab	1000	0.94
Bailey's Landing I	600	0.90
Median	**4000§**	
With Fines		
McCrary Park	27	0.98
Atlantic Beach PC	14	0.97
Bryarton I	13	0.60
WB Church	11	0.97
Median	**13§**	

§ The difference in surface infiltration rate when PC was sited in stable versus disturbed watersheds was significant. ($p = 0.0074$, $df = 1$)

Porous Asphalt

The surface infiltration rates were tested for five PA sites (Appendix A) located in the Piedmont and Coastal Plain of North Carolina. Due to the low number of sites and substantial differences between sites, no statistical analyses were performed. Surface infiltration rates ranged from 6000 cm/h (2500 in/h) for a site free of fines (Friday Center Park and Ride, PA) to 3.9 cm/h (1.5 in/h) for a site (Atlantic Beach, PA, II) with substantial sedimentation (Appendix A). Another site located in the same town, Atlantic Beach, NC, had a surface infiltration rate of 13.3 cm/h (5.2 in/h). Two other sites, located in Fayetteville, NC, with minimal sedimentation, were constructed in 1986 and 1996, respectively. The sites, both being at least seven years old at the time of testing, still had surface infiltration rates of 5.7 cm/h (2.3 in/h) and 5.4 cm/h (2.1 in/h).

Additional Sites

Three additional sites were also tested, but they were each unique and therefore were not included in other analyses. Two sites were located adjacent to one another, a CGP site and a Plastic Turf Reinforcing Grid (PTRG) site. The CGP site, Kinston CGP, had a surface infiltration rate of 58 cm/h (23 in/h) (Appendix A). Since the existing surface infiltration rate was so high, a post-maintenance test was not conducted. As a result, Kinston CGP data were not included in the analysis of maintenance on improved surface infiltration rates. Two different tests were run on the PTRG site, Kinston GP; one in grassed areas and one in areas mostly barren of grass. Areas with grass had a surface infiltration rate of 31 cm/h (12 in/h), while barren areas had a rate of 9.1 cm/h (3.6 in/h). The decreased surface infiltration rate for barren areas was likely due to compaction of the top soil. The last site tested, Wynn Plaza, was a CGP site with No. 78 stone (ASTM, 2003b) filled into the voids. It had a surface infiltration rate of 11 cm/h (4.5 in/h).

ANALYSIS AND CONCLUSIONS

Several observations were drawn from this field study: (1) maintenance is key to sustaining high surface infiltration rates for CGP, (2) the siting of permeable pavement applications, including PICP and PC, away from disturbed sediment contributing areas is a significant factor in preserving high surface infiltration rates, and (3) permeable pavements that were installed in sandy soil environments maintained relatively high surface infiltration rates, without regard to pavement age or type

Fourteen of 15 sites had increased infiltration rates after removal of void space material (13 mm (0.5 in.) typical depth). Without maintenance, the median surface infiltration rate was 4.9 cm/h (1.9 in/h); while with maintenance the median infiltration rate was 8.6 cm/h (3.4 in/h). A Mixed Procedure (SAS^{TM}, 2003) analysis showed that there was a statistically significant ($p = 0.007$) difference between existing and maintained infiltration rates. Therefore, simulated maintenance significantly improved infiltration rates for the CGP sites filled with sand.

Infiltration rates of PICP filled with pea gravel were not limited by their surface infiltration capacity provided sediment contribution areas were stable. The median PICP infiltration rate was 2000 cm/hr (900 in/h), while the PICP sites near disturbed soils with fines was 8.0 cm/h (3.1 in/h), a decrease of 99.6%. A Mixed Procedure (SAS^{TM}, 2003) analysis was used to determine there was a significant ($p = 0.0014$) difference between the surface infiltration rates of PICP near fines and free of fines.

Eleven PC sites were measured for surface infiltration rates. Like PICP sites, infiltration rates of PC were not limited by their surface infiltration capacity as long as they were sited in areas unlikely to accumulate fines. The median surface infiltration rate for PC sites relatively free of small particle deposition was 4000 cm/h (2000 in/h); compared to 13 cm/h (5.3 in/h) for sites with deposition of fines. The difference was a 99.7% reduction of the median surface infiltration rate and a Mixed Procedure (SAS™, 2003) statistical analysis showed a significant ($p = 0.0074$) difference in surface infiltration rates.

Even though test sites for PICP and PC were located in two different geographical and soil regions, there were not enough data to draw conclusions on permeable pavement use in clay soil regions. All piedmont (clay soil) PC sites were without fines, while all but one PC site in the coastal plain (sandy soil) were adjacent to a disturbed soil.

Lastly, 45 of 48 sites tested had surface infiltration rates greater than 2.5 cm/h (1.0 in/h). These rates were comparable to rates expected for some hydrologic group A soils (loamy sands, sandy loams) covered with grass (USDA, 1986). Clogging at the permeable pavement surface in predominantly coarse grain (sandy) soil environments, therefore, does not cause permeable pavements to have surface infiltration rates reduced below some naturally grassed areas. This study, however, did not address clogging that sometimes occurs at lower depths within permeable pavements, nor did it address the impacts that poor siting and/or construction techniques have on flow rate below the pavement surface.

The following siting and maintenance guidelines were developed as a result of this study:

1) For CGP sites filled with sand:

   to sustain higher surface infiltration rates, maintenance, using a vacuum sweeper, should be performed on regular intervals. Removal of the top 13 mm (0.5 in.) – 18 mm (0.75 in.) of material accumulated within void spaces has been shown to significantly improve infiltration rates. Sand should then be backfilled into the void spaces to prevent sealing at a lower depth.

2) For PICP / PC sites:

   PICP and PC sites installed for infiltration purposes should not be located adjacent to areas with disturbed soils as accumulations of fine particles have been shown to significantly decrease surface infiltration rates. Maintenance should include regular use of a vacuum sweeper as needed (annually, if not more frequent) for sediment accumulation on the surface. Problems with fines should be addressed before the fines are either compacted into void spaces or migrate to lower, harder to maintain depths within the pavement void profile. Construction sequencing is critical for maintaining high surface infiltration rates. Permeable pavements installed in stable watersheds will function substantially better than those constructed in unstable watersheds.

This and other studies (Balades, et al, 1995; Pratt, et al, 1995; Hunt, et al, 2002) suggest that permeable pavements do considerably reduce runoff, provided the following conditions are met: (1) the pavement is sited in a sandy or loamy sand soil, (2) it is located in soils without seasonally high water tables, (3) the pavement is well-maintained, (4) proper construction materials and techniques are used, (5) the pavement is essentially flat and away from disturbed fine soils, and (6) does not have excessive structural loads beyond designed capacity.

ACKNOWLEDGEMENTS

Financial support was provided by the Interlocking Concrete Pavement Institute (ICPI) and the North Carolina Department of Environmental Health and Natural Resources, administrator of the EPA 319(h) grants. The contributions of Brandon Eckrote, Zach Woodward, Lucas Sharkey, and L. T. Woodlief were critical in the data collection. The authors appreciate Jonathan Smith's assistance, guidance, and support. The property owners, town officials, and extension agents who assisted in site selection and background information for each site were also vital to this study.

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CHAPTER 2

WATER QUALITY AND QUANTITY PERFORMANCE MONITORING OF PERMEABLE INTERLOCKING CONCRETE PAVEMENT SITES IN NORTH CAROLINA

ABSTRACT

Impermeable surfaces have greatly increased the amount of pollutant-carrying runoff entering surface waters. To counteract this, permeable pavements can be installed to allow water to infiltrate, thus reducing runoff and potentially acting as a filter. Three permeable interlocking concrete pavers (PICP) sites were monitored in North Carolina; located in Cary, Goldsboro, and Swansboro. The Cary site was located on a clay loam soil; 15 samples of exfiltrate and rainfall were analyzed for pollutant concentrations between February 2004 and November 2004. The Goldsboro site was constructed in 2002 to compare pollutant concentrations of asphalt runoff to exfiltrate of adjoining permeable pavement. Up to 14 samples of Zn, NH_4-N, TKN, NO_{2+3}-N, TP, and Cu were collected from the asphalt runoff and the permeable pavement exfiltrate from July 2003, to November 2004. Concentrations of Zn, NH_4-N, TKN, and TP were significantly ($p \leq 0.05$) lower in PICP exfiltrate than in asphalt runoff. The Swansboro site was constructed in 2003 and instrumented to monitor runoff flow rates, record rainfall rates, and collect samples from PICP exfiltrate and runoff between March 2004 and December 2004; however, during the entire monitoring period, the 740 m^2 parking lot produced no runoff.

INTRODUCTION

Urbanization has had a detrimental effect on surface water quantity and quality. Runoff from paved surfaces have increased peak flow, time to peak, and runoff volumes through stream channels, causing overland erosion and stream bank instability (NRCS, 1986). Urban runoff also carries pollutants, such as sediments, nutrients, and heavy metals, into surface waters (Barrett, et al, 1998; Davis, et al, 2000; Lee and Bang, 2000; He, et al, 2001)

In 1972, Congress created the Federal Water Pollution Control Act, commonly known as the Clean Water Act (CWA), to protect surface waters of the United States (FWPCA, 2002). Section 303 of the CWA gave the responsibility of enforcing water quality to the individual states and established Total Maximum Daily Loads (TMDLs) as the pollutant measurement standard. TMDLs are the maximum pollutant loads that a water body can bear and still meet water quality standards for its intended use. These standards initially focused on point sources of pollution such as discharges of industrial process wastewater and municipal sewage treatment plants. However, non-point sources, such as stormwater runoff and discharge, still accounted for a substantial amount of pollution for impaired waters (US EPA, 1996). Approximately 46% of identified estuarine water quality impairment cases surveyed across the United States were attributable to storm sewer runoff. As a result, Congress amended the CWA in 1987 establishing requirements for storm water quality (US EPA, 1996).

The National Pollutant Discharge Elimination System (NPDES) storm water program was developed to regulate stormwater discharges in large (Phase I) and medium (Phase II) communities. Phase I (1990) communities, or groups of municipalities, had populations exceeding 100,000 (EPA, 1996), while Phase II (1999) addressed municipalities of less than 100,000 people each. In an effort to reduce these effects of urbanization and to comply with NPDES rules, several municipalities in North Carolina established regulations that limit the amount of impervious surfaces (Bennett, 2003). In 2000, urban stormwater runoff remained among the top three sources of pollution for lakes, ponds, reservoirs and estuaries in the United States (US EPA, 2000).

In North Carolina, regulated pollutants in stormwater include nitrogen, phosphorus, pathogenic bacteria, and total suspended sediments. Fish kills in recent years in the Pamlico Sound and its tributaries have been attributed to increased nitrogen and phosphorous levels (Burkholder and Glasgow, 1997). As a result, municipalities in the Neuse and Tar-Pamlico watersheds are required to limit their nutrient loadings (NCDENR, 2001; NCDENR, 1998). Across North Carolina, any developments in water supply watersheds are required to treat runoff from the 2.5 cm (1 in.) event and reduce runoff TSS by 85% (NCDENR, 2004).

The EPA lists numerous Best Management Practices (BMPs), both passive and structural, as potential tools for improving water quality for NPDES permitting. Ice (2000) examined the role of BMPs in addressing degraded waters. BMPs represent a balance between the need to control non-point source pollution and the need for those practices to be feasible and practical (Ice, 2004). North Carolina has implemented a stormwater credit system using BMPs for developed sites to manage onsite runoff to comply with NPDES rules. North Carolina regulators gave several BMPs credit for pollutant reduction, sediment reduction, and peak flow detention;

however, permeable pavements were not included. Permeable pavements are only a credited BMP under the “innovative BMP” classification. Innovative BMPs, however, must be monitored on an individual basis to assess their performance; few landowners have been willing to assume the cost of the required monitoring (Bennett, 2003).

The State of North Carolina did not grant credit to permeable pavements due to their history of perceived clogging. Research by Balades (1995) found that pavements clog by initially trapping large particles and gradually trapping progressively smaller particles over time. Results from several studies have shown that maintenance, or simulated maintenance, can potentially restore infiltration capacities (Gerrits and James, 2002; Hunt et al., 2002; and Bean et al., 2004). Permeable pavements, when correctly maintained, allow stormwater to infiltrate into either a storage basin below the pavement or exfiltrate to the soil and ultimately recharge the water table, while also potentially removing pollutants (US EPA, 1999). As a result, the use of permeable pavements is poised to grow, if they were given credit by the State of North Carolina officials.

“Permeable pavements” is a general term referring to three distinctly different types of surfaces. Porous, or pervious, pavement commonly refers to porous asphalt or porous concrete, along with any other surface that must be poured that cures to create void spaces. Traditional permeable pavements are surfaces assembled with individual paver blocks or stones, e.g., permeable interlocking concrete pavers (PICP) and concrete grid pavers(CGP), that have open spaces filled with a permeable material such as sand or No. 78 stone (ASTM, 2003), commonly known as pea gravel. Plastic reinforcing grid pavers (PRGP) are used as reinforcement mats, designed to be filled with either soil and grass or loose gravel. Each of these pavement types provides a varying degree of permeability, depending on factors including appropriate design, proper construction, traffic load, and regular maintenance.

Recent studies have found various types of permeable pavements reduce runoff. Hunt, et al, (2001) monitored a permeable pavement parking lot that was divided between sand filled concrete grid pavers (CGP) and grassed pavers (Appendix A 46 and 47, respectively). Rainfall and runoff rates were recorded from June 1999, to March 2001. Only 25% of storms produced runoff from the site. Most runoff occurred during short time periods of extremely intense rainfall, typically greater than 4.1 cm/h (1.6 in/h), that overwhelmed the pavement's infiltration capacity. Calculated rational coefficients for events greater than 1.3 cm (0.5 in.), ranged from 0.08 to 0.36, while coefficients for events greater than 2.5 cm (1 in.) ranged from 0.2 to 0.48. Hunt, et al, estimated an applicable coefficient was between 0.15 and 0.30 (Hunt, et al, 2001).

In a study by Pratt, et al, (1995) from 1987 to 1989, four permeable (non-interlocking) concrete paver cells with various sub-base materials were monitored for runoff, exfiltrate and water quality improvement. Of 62 rainfall events, ranging from 0.28 cm (0.11 in.) to 2.3 cm (0.9 in.), runoff occurred from at least 42 of the events for each cell. Between 34% and 47% of all rainfall exfiltrated the reservoir structures for various sub-base materials. The pavers absorbed a portion of rainfall, directly related to the duration of events. Pratt (1995) also stated that initial exfiltrate flows typically occurred after 0.28 cm (0.11 in) to 0.32 cm (0.13 in) of rainfall, which typically occurred 25-50% into the event duration.

A field study of parking stalls, constructed in 1996 in King County, Washington, compared runoff production from an asphalt surface (1 stall) and four types of permeable pavements (2 stalls each): PICP, CGP, PRGP with grass, and PRGP with gravel (Brattebo and Booth, 2003). Runoff and infiltration rates for the site were recorded throughout November 2001, and from January to early March 2002. Fifteen individual events totaled 57 cm (20 in.) or rainfall during the monitoring period with 12 cm (4.7 in.) resulting from the largest event. Permeable pavements infiltrated virtually all rainfall, with only 0.4 cm (0.16 in) of runoff produced from the PRGP with grass stalls from the largest event (Brattebo and Booth, 2003). However, the maximum rainfall intensity during monitoring was 0.78 cm/h (0.30 in/h), substantially lower than the runoff producing intensity reported in the Hunt, et al, study (2001).

Permeable pavements also reduce runoff concentrations and loadings. Brattebo and Booth (2003) analyzed asphalt runoff and exfiltrate from each of the permeable pavements for pollutants from nine of the fifteen rainfall events. After normalization of results by log-transformation, asphalt runoff had significantly higher concentrations of Zn and Cu ($p < 0.01$).

In a two year study in Florida, water samples were analyzed from asphalt runoff and outflow from grassed swales that captured runoff from three different pavements: asphalt, concrete, and porous concrete (Rushton, 2001). Due to the high infiltration of the native soil (9.9% of soil >0.2 cm (0.1 in.)), only rainfall events of greater than 0.84 cm (0.33 in.) produced flow in the swales. Total rainfall during the two year study was 155 cm (61 in.), which was considered to be drought conditions for the location. Comparing loads from porous concrete swale and asphalt swale samples, Zn loads from the porous concrete were approximately half of the loads from the asphalt, while Cu loads were less than half. Compared to the asphalt runoff, Zn and Cu loads from the porous concrete with a swale were on average 58% and 84% lower, respectively. In addition, Rushton (2001) reported that the porous concrete with swale system, when compared to asphalt runoff, reduced the overall pollutant loadings in runoff by over 99% due to limited runoff production.

In a French study by Pagotto, et al, (2000), a section of highway asphalt was replaced with porous asphalt. The stretch of bridge highway originally opened in 1993 and was instrumented to monitor runoff rates and pollutant loads from March 1995 to February 1996. During the summer of 1996, 3.0 cm (1.2 in.) of porous asphalt over an impervious material replaced the traditional asphalt and was monitored from June 1997 to May 1998. During the first monitoring phase, 125 events produced 69.8 cm (27.5 in.) of rainfall, while during the second period, 162 events produced 79.6 cm (31.3 in.) of rainfall. Runoff loadings of TSS, Cu, and Zn from porous asphalt were significantly less than loadings from impervious asphalt. Pagatto, et al, (2000) stated that from sediment analysis of pollutants the primary pollutant removal process of the porous asphalt was filtering.

For this study, three PICP sites, one each in Cary, Goldsboro, and Swansboro, were equipped with monitoring equipment to collect runoff and/or exfiltrate samples. During this study, any water that passed through the PICP pavers until exiting the storage basin is referred to as infiltrate, while water that left the storage basin through a drainpipe or was collected from within the storage layer was referred to as exfiltrate. Equipment at the sites in Cary and Swansboro also recorded exfiltrate or runoff rates. The goals of this study were as follows: (1) develop an SCS

Curve Number (CN) and Rational Coefficient (C) for two field sites, (2) monitor pollutant levels of rainfall, asphalt runoff, PICP exfiltrate and PICP runoff to determine whether the PICP systems reduced pollutant concentrations, and (3) offer siting guidelines based upon these results.

STUDY SITES

Three PICP sites in central and eastern North Carolina were instrumented to monitor water quality, two of which were also equipped to record water quantities. Figure 1 shows the relative location of each site.

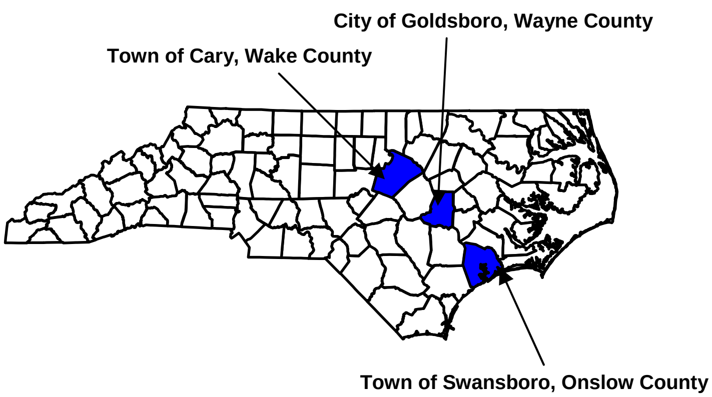

Figure 1. North Carolina counties map highlighting counties and municipalities where research sites were located.

Cary

The western-most site, in Cary (Figure 2a), was constructed on clay loam soil in the fall of 2003 with a surface area of 480 m^2 (4200 ft^2) (Appendix B). The SF Rima™ [1] pavers were 8 cm (3 in.) thick and were laid over a compacted layer of at least 25 cm (10 in.) of washed No. 57 stone

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[1] The use of trade names is for project information only and does not constitute an endorsement by North Carolina State University.

(ASTM, 2003), with a 5 cm (2 in.) layer of No. 78 stone between the two layers (Appendix A 25). Photo-analysis was used to determine that open, or void, space of PICP in Figure 2b was approximately four percent (4%) of the surface area. The storage basin under the pavers was divided into two separately drained basins of 185 m^2 (1996 ft^2) and 243 m^2 (2619 ft^2). Clay loam soil infiltration rates are typically too low for drainage basins to recurrently empty before subsequent rainfall events. Therefore, one 10 cm (4 in) corrugated plastic pipe drained each of the drainage basins. An ISCO 674® tipping bucket rain gauge connected to a data logger recorded precipitation rates. The storage basin, or pavement support layer, was lined with an impermeable geo-textile to prevent deep seepage and shrink/swell associated with some clay soils.

Figures 2a and 2b. Cary monitoring site and close up of the PICP surface.

Goldsboro

About 70 miles east of the Cary site, the Goldsboro site was a parking lot for a bakery (Figure 3a) with loamy sand in-situ soils. In the summer of 2001, the parking lot was constructed for asphalt runoff and exfiltrate collection, analysis and comparison. Photo-analysis was used to determine that open space of PICP in Figure 3b was approximately nine percent (9%) of the surface area. The 8 cm (3 in.) thick UNI® Eco-stone® pavers overlaid 8 cm (3 in.) of No. 72 stone, which were, in turn, laid over 20 cm (8 in.) of washed No. 57 stone (ASTM, 2003) (Appendix A 16).

Figures 3a and 3b. Goldsboro monitoring site and close up of the PICP surface.

An 8 cm (3 in) PVC pipe was installed under a section of the PICP during construction for exfiltrate sample collection. Holes were cut into the pipe to allow infiltrate to fill the pipe (Figure 4). The pipe drained approximately 120 m^2 (1300 $ft.^2$) of PICP (Appendix C) and was capped by a hand valve. To capture asphalt runoff, the drive path was graded so that runoff would flow towards a metal channel, where water samples were collected by a Sigma 900™ automated sampler on a temporal basis.

Figure 4. PVC drainpipe with infiltrate holes installed under PICP in stone layer at the Goldsboro site.

Swansboro

The eastern-most site, Swansboro, was a public parking lot (Figure 5a) constructed in the fall of 2003 with an area of 740 m^2 (8000 ft^2) (Appendix D). Eight (8) cm (3 in.) thick UNI® Eco-stone® pavers overlaid 8 cm (3 in.) of No. 72 stone, which overlaid 20 cm (8 in.) of washed No. 57 stone (ASTM, 2003) over an in-situ sandy soil (Appendix A 20). By photo-analysis, open space of PICP in Figure 5b was approximately nine percent (9%) of the surface area. The site was slightly sloped (0.4%) so that runoff would flow to a concrete swale, which emptied into a weir box to measure runoff rates. An 8 cm (3 in) PVC drainpipe was installed in the drainage basin during construction to collect exfiltrate for water quality analysis. A hand valve, which opened for sample collection and drainage, capped the drainpipe. The site was constructed to collect runoff and exfiltrate samples for water quality from PICP and monitor rainfall intensities and runoff rates.

Figures 5a and 5b. Swansboro monitoring site and close up of PICP at the site.

MATERIALS AND METHODS

Hydrologic Study

To address the first goal of this study, Swansboro, was equipped with flow monitoring equipment. Infiltrate volumes were determined from rainfall and runoff flow monitoring data. At the Swansboro site, runoff flowed down a cement spillway into a weir box with a baffle and a 90^{o} V-notch weir. The weir box was approximately 0.6 m (2 ft) wide by 0.9 m (3 ft) deep by 0.46 m (1.5 ft) tall. Baffles were installed 0.3 m (1 ft) in front of the inlet to still the flow and weirs were installed 0.15 m (0.5 ft) in front of the baffles. An ISCO® 6712 Automatic Sampler® with 730 Flow Bubbler Module® was used to record water depth within the weir box. The sampler and bubbler were stored in a large metal housing, fabricated by the Biological and Agricultural Engineering (BAE) Department at North Carolina State University (NCSU). The bubbler increased a pressure within a tube that outlet into the weir box, as long as the tube was submerged. If the tube was not submerged, then no resistance formed against the air flow to build up pressure. When enough pressure built up that the tube released a bubble, the sampler recorded the pressure. The bubbler tube was installed midway between the baffle and the weir. The sampler automatically converted the pressure to water level. Then Equation 1 was used to calculate flow rates, Q (l/s), from recorded water level, or Head, H (m), above the weir inverts, for 90^{o} V-notch weirs. The sampler was programmed to collected runoff samples from the weir box when the water level was higher than the weir invert. However, since no runoff occurred at the Swansboro site during the monitoring period, the sampler neither collected nor recorded any runoff.

$$Q = 1380 * H^{2.5} \quad \text{(Equation 1)}$$

To determine the volume of water entering the site, an ISCO® 674 tipping bucket rain gauge was installed at the Swansboro site to record rainfall. Rainfall (RF) was then multiplied by the pavement area (PA), assuming no off-site runoff infiltrated the pavement area. By quantifying the volume of runoff (V_{RO}) produced by the site, the volume of water entering the storage basin through the pavers, infiltrate (V_{IN}), was calculated using Equation 2. The Swansboro rain gauge connected directly to the sampler, which recorded the precipitation data. The rain gauge collected 0.025 mm (0.01 in.) of rainfall per tip and as a back up, a generic manual rain gauge also collected rainfall. Figure 6 displays the monitoring setup at the Swansboro site.

$$V_{IN} = RF * PA - V_{RO} \quad \text{(Equation 2)}$$

Figure 6. Swansboro monitoring layout from left to right: housing for sampler and bubbler, rain gage, weir box, and runoff spillway.

The Cary site was instrumented to determine runoff attenuation performance as well. However, due to repeated instrument failures and the quality of data collected, few hydrologic conclusions were made from the site. Therefore, data and results were listed in Appendix E.

Water Quality

To address the second goal of this study, water samples were collected from each of the three sites for analysis to determine whether PICP exfiltrate had lower pollutant concentrations when compared to runoff and rainfall. The Cary site was instrumented to collect exfiltrate and rainfall samples, while the Goldsboro site was instrumented to collect asphalt runoff and PICP exfiltrate samples. Although no runoff occurred during the monitoring period, the Swansboro site was instrumented to collect PICP runoff and exfiltrate samples. The Goldsboro and Cary sites were analyzed as paired watersheds, while the Swansboro site was intended as an inflow versus outflow comparison.

At the Cary site exfiltrate flowed into a weir box where samples were collected. Similar to the Swansboro site, an ISCO® 6712 Automatic Sampler® with 730 Flow Bubbler Module® recorded the water level behind the weir within the weir box. The Cary and Swansboro samplers were programmed to collect 200 ml (8 oz.) of exfiltrate or runoff, respectively, every 5 minutes when the water level was higher than the weir invert. At the Cary site, rainfall was captured using a plastic catch basin for water quality analysis.

The Goldsboro site was instrumented to compare runoff from an asphalt area to exfiltrate from a PICP cell. Asphalt runoff was collected where the curb opens into a grassy swale. A metal channel captured a portion of the runoff at the threshold where the curb met the grassed swale. A pipefitting was attached through the side of the metal channel to secure pump tubing for sampling. The tubing connected to a Sigma® 900 Max® automatic sampler. The sampler was programmed to automatically suction a 75 ml (2.6 oz) sample of runoff every 20 minutes, regardless of whether runoff was present. The sampler deposited runoff samples in a 11 liter (3 gallon) glass container until samples were collected. The sampler was locked in a large metal housing unit, fabricated by the BAE Department at NCSU, adjacent to the metal channel.

Exfiltrate from the PICP cell was collected and stored in a drainpipe running under the PICP parking stalls. The collection pipe was an 8 cm (3 in.) PVC pipe with 0.64 cm (0.25 in.) diameter holes drilled in at a frequency of approximately 15 per linear foot. The holes were only drilled through the top half of the PVC pipe. A hand valve, located at the PVC drainpipe's outlet, was opened to collect a sample. Once open, exfiltrate flowed into a wooden housing surrounding the end of the pipe. Once water rose above the walls of the housing it overflowed into a bioretention basin. When sampling, the hand valve would initially be opened to flush any residual exfiltrate that may have accumulated. Once water filled the housing, the valve was closed and a small pump was engaged to pump water into the bioretention and consequently lowered the water below the invert of the valve, so that a sample could be taken. The valve was then opened again and a 250 or 500 ml (8.5 or 17 oz.) bottle was filled with stored exfiltrate from the PVC drainpipe. After the sample was collected, the valve remained open to allow the PVC drainpipe to empty. With the pipe empty and the water level below the valve invert, the valve was closed to capture exfiltrate from the next event.

Exfiltrate samples from the Swansboro site were collected similarly to those from Goldsboro. The drainpipe at Swansboro opened onto boulders at the top of a grassed slope. To collect samples, a hand valve was opened to allow any residual sample within the pipe to flush out for approximately five to ten seconds. The hand valve was then closed and reopened for sample collection. The valve was reopened and a 250 or 500 ml (8.5 or 17 oz.) bottle was filled with stored exfiltrate from the PVC drainpipe. The valve remained open after the sample was collected to allow the PVC drainpipe to empty. The valve was then closed to capture exfiltrate from the next event.

All collected samples were either frozen or acidified with H_2SO_4 within 24 hrs. One drop of sulfuric acid was added for every 50 ml (1.7 oz.) of sample. Samples from all three sites were analyzed for concentrations of Total Nitrogen (TN), Nitrate+Nitrite in Water (NO_{2+3}-N), Total Kjeldahl Nitrogen in Water (TKN), Ammonia in Water (NH_4-N), Organic Nitrogen (ON), Total Phosphorus (TP), Orthophosphate (PO_4), and Bound Phosphorus (BP). The initial eight (8) sets of samples from Goldsboro were also analyzed for Copper (Cu) and Zinc (Zn) concentrations. Only the last six (6) sets of samples from Goldsboro, collected from August 3 thru December 12, 2004, were analyzed for NH_4-N and PO_4 which allowed analysis for ON and BP. Tritest of Raleigh analyzed all samples from the Goldsboro site from June 11, 2003, until February 13, 2004. The Analytical Services Laboratory (ASL) at NCSU analyzed samples from Goldsboro after February 13, 2004, and all samples from both Cary and Swansboro. Only runoff and exfiltrate from Goldsboro and exfiltrate from Cary were analyzed for Total Suspended Solids (TSS). TSS samples were analyzed at the NCSU Water Quality Group lab in Raleigh, NC. Table 1 lists analyses performed on samples, method of testing, and minimum detectable level (MDL). For statistical analysis, if pollutant concentrations were less than the MDL, the concentrations were set to one-half of the MDL.

Table 1. Nutrient analyses performed, abbreviations, sources of analysis method, and minimum detectable levels for laboratory analysis and results during this study.

Test Performed	Abbreviation	Analysis Method	MDL (mg/l	
		(EPA, 1983; EPA, 1993)	Tritest	ASL
Total Nitrogen Calculation	TN	TN = TKN + NO_{2+3}-N	##	##
Nitrate—Nitrate in Water	NO_{2+3}-N	EPA 353.2	0.02	0.1
Total Kjeldahl Nitrogen in Water	TKN	EPA 351.2	0.25	0.1
Ammonia in Water	NH_4-N	EPA 350.1	##	0.1
Organic Nitrogen Calculation	ON	ON = TKN – NH_4	##	##
Total Phosphorus	TP	EPA 365.4	0.05	0.01
Orthophosphate	PO_4	EPA 365.1	##	0.01
Bound Phosphorus Calculation	BP	IP = TP – PO_4	##	##
Total Suspended Solids	TSS	EPA 160.2	1	1
Zinc in Water	Zn	EPA 200.8	0.01	##
Copper in Water	Cu	EPA 200.8	0.01	##

indicates laboratory did not test for specific pollutant.

Water quality data from Goldsboro and Cary were analyzed for the control and treatment samples to determine whether a significant ($p \leq 0.05$) difference existed between pollutant concentration sets. The Descriptive Statistics function in Microsoft® Excel® (2003) was applied to each data set to determine whether values were normally distributed. Data sets were normally distributed if the absolute value of skewness was less than or equal to one (≤ 1). If data for both data sets for a pollutant were normally distributed, then a student t-test (Microsoft®, 2003) was applied to the data to determine whether the means were significantly different. If the absolute value of skewness for either data set was greater than one, then both data sets were log-transformed and skewness was determined again. If the absolute value of skewness was less than or equal to one (≤ 1), for both log-transformed data sets, then a student t-test was applied to the log-transformed data. After log-transformation, if the absolute value of skewness for either data set was greater than one, then a sign test was performed on the non-transformed data to determine whether a significant ($p \leq 0.05$) difference existed between the populations.

RESULTS AND ANALYSIS

Hydrology Study

Runoff and rainfall data from the Swansboro site were collected for ten consecutive months, March through December 2004. During the entire monitoring period, from March 1 until December 31, 2004, 107 cm (42 in.) of rainfall fell on the site, but no runoff was produced. The largest event recorded was 8.8 cm (3.5 in). Four events had at least 5 cm (2 in.) of rainfall. An average SCS Curve Number (CN) of 44 was determined by calculation using the SCS runoff curve number method (NRCS, 1986), using Equations 3 and 4 below. Rainfall events ranged in size from 5.1 cm (2.0 in.) to 8.9 cm (3.5 in) for calculations, all less than the 2-yr 24-h event (11.4 cm (4.47 in.) (NOAA, 2005)). The variables S and P are the initial abstraction and rainfall depth, respectively, while Q is the resultant runoff depth. The CN was determined by manipulating S until Q was equal to 0 for recorded precipitations, since no runoff occurred, using Equation 3. The S value was then used to determine a CN using Equation 4. A rational coefficient of zero was calculated using the Rational Method (APWA, 1981) (Equation 5).

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \text{(Equation 3)}$$

$$CN = \frac{1000}{S + 10} \qquad \text{(Equation 4)}$$

$$Q_P = CIA \qquad \text{(Equation 5)}$$

Q_P was the peak runoff, while I and A were the peak rainfall intensities and watershed area. C is the rational coefficient based on a ratio between peak runoff rate and infiltration rate for the watershed.

During the summer of 2004, a single ring infiltration test was conducted at the Swansboro lot and extremely high surface infiltration rates, with a measured average of 2000 cm/h (800 in/hr) (Chapter 1, Appendix A 20). From a visual perspective, there was no clogging of the surface throughout the study. The lack of runoff from this site during the entire monitoring period is explained by (1) being located on very permeable soil, (2) having a large gravel storage volume, 20 cm (8 in) thick, and (3) having a surface free of fines. The Swansboro parking lot may be considered an ideal location for a PICP installation.

Water Quality Study

Summaries of water quality and statistical results from all three sites (Goldsboro, Cary and Swansboro) are reviewed in this section. Tables and graphs of all water quality analyses are in Appendix F, while statistical analyses outputs are in Appendix G.

Goldsboro

Water quality data were collected for 14 storms at the Goldsboro site from June 2003 until December 2004 (Table 2). Rainfall depths were determined from rainfall data collected at the Goldsboro-Wayne County Municipal Airport, approximately 10 km (6 mi.) north of the site. Seymour Johnson Air Force Base was closer to the site, approximately 4 km (2.5 mi.); however, only daily rainfall data was available from there.

Table 2. Rainfall event dates and depths for samples collected from Goldsboro site from June 2003 through December 2004.

Event	Date	Rainfall (cm)	Rainfall (in.)
1	6/11/2003	0.08	(0.03)
2	7/24/2003	2.13	(0.84)
3	8/15/2003	1.63	(0.64)
4	9/19/2003	1.04	(0.41)
5	9/23/2003	0.89	(0.35)
6	10/9/2003	1.45	(0.57)
7	2/4/2004	0.69	(0.27)
8	2/13/2004	0.79	(0.31)
9	8/3/2004	0.71	(0.28)
10	8/4/2004	0.15	(0.06)
11	8/6/2004	0.99	(0.39)
12	8/16/2004	3.15	(1.24)
13	10/13/2004	0.13	(0.05)
14	12/12/2004	0.08	(0.03)

Table 3 summarizes the statistical mean pollutant concentrations and factors of significance for population differences. It was hypothesized that exfiltrate outflow pollutant concentrations would be significantly (p-value $\leq$ 0.05) lower than asphalt runoff concentrations. As noted previously, earlier studies have shown that permeable pavement systems, including PICP, can remove pollutants from runoff. Table 3 shows that exfiltrate concentrations of Zn, NH_4-N, TP, and TKN were significantly lower than asphalt runoff concentrations of the same pollutants. Exfiltrate outflow concentrations for TN, ON, PO_4, BP, TSS, and Cu were not significantly lower than asphalt runoff concentrations, but were arithmetically lower. Nitrate-Nitrite, NO_{2+3}-N, was the only pollutant to have arithmetically higher concentrations in the exfiltrate than the runoff.

Table 3. Statistical mean pollutant concentrations and p-values from the Goldsboro site.

Pollutant Analysis	Asphalt Runoff (mg/l)	PICP Exfiltrate (mg/l)	p-value [Test]	Events
Total Nitrogen Calculation mg/l (TN)	1.33	0.77	0.0511 [LN]	1-14
Nitrate-Nitrite/Water mg/l as N (NO_{2+3}-N)	0.30	0.44	0.1668 [N]	1-14
Total Kjeldahl Nitrogen/Water mg/l (TKN)	1.03	0.41	**0.0074** [LN]	1-14
Ammonia mg N/l (NH_4)	0.31	0.05	**0.0003** [LN]	9-14
Organic Nitrogen mg/l (ON)	0.88	0.54	0.6875 [SN]	9-14
Total Phosphorus/Water mg/l (TP)	0.134	0.049	**0.0017** [LN]	1-14
Orthophosphate mg P/l (PO_4)	0.038	0.022	0.2730 [LN]	9-14
Bound Phosphorus (BP)	0.077	0.057	0.2752 [N]	9-14
Total Suspended Solids mg/l (TSS)	43.8	12.4	0.5811 [SN]	1-12,14
Copper by ICP/MS-Water mg/l (Cu)	0.016	0.006	0.2188 [SN]	1-8
Zinc by ICP/MS-Water mg/l (Zn)	0.067	0.008	**0.0001** [N]	1-8

*Paired student t-test using MS Excel™ and sign test using SAS™ statistical analysis program were used to determine p-values. Bold values are significant (p $\leq$ 0.05) differences. N: normal student t-test; LN: log transformed normal student t-test; SN: sign test.

Permeable pavements have been shown to significantly remove Zn and Cu, as documented in several earlier studies (Brattebo and Booth, 2003; Rushton, 2001; Pagotto, 2000). PICP exfiltrate from the Goldsboro site had significantly less Zn (p = 0.001) (Figure 7) and arithmetically less

Cu (Figure 8) than runoff concentrations. Sansalone and Buchberger (1997) determined that Zn and Cu in runoff were typically in a dissolved form, rather than particulate. Sansalone and Buchberger (1997) also examined pH of runoff in their study and found that the low pH of rainfall and holding time of infiltrate contributed to metals being in solution. They suggested that the use of concrete could be effective at increasing the pH of runoff enough to precipitate metals out of solution (Sansalone and Buchberger, 1997). Pratt (1995) recorded pH values for concrete pavers over gravel to be between 6.9 and 9.3; both metals precipitate above a pH of 7. Therefore, it is possible that the lower concentrations of Cu and Zn were due to infiltrate having flowed over and through concrete pavers, increasing the pH to precipitate these metals. Precipitated metals would have collected on the base soil surface and not entered the collection drain.

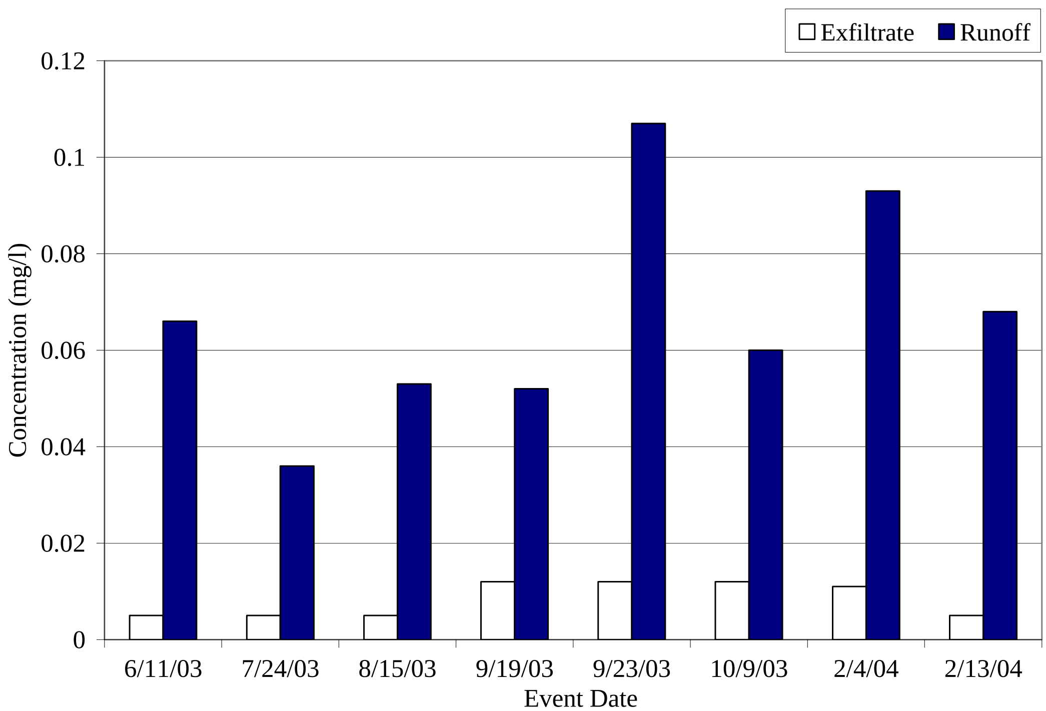

Figure 7. Zn concentrations for asphalt runoff and PICP exfiltrate from the Goldsboro site from June 2003 through February 2004.

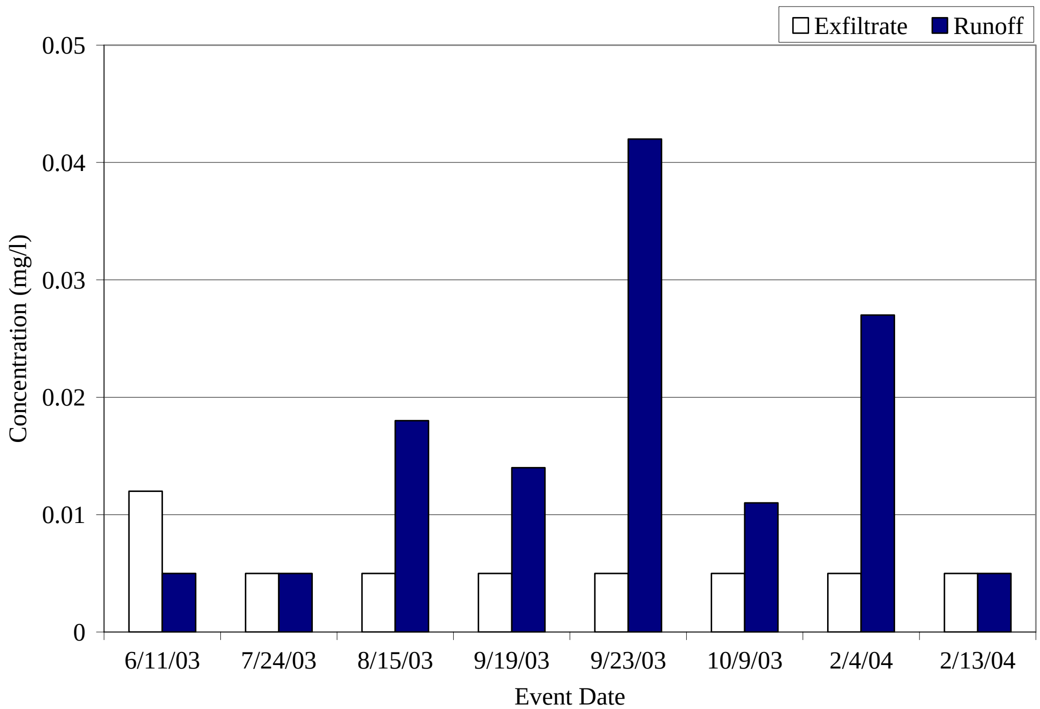

Figure 8. Cu concentrations for asphalt runoff and PICP exfiltrate from Goldsboro site for events between June 2003 and February 2004.

The PICP cell at the Goldsboro site was designed to drain quickly to facilitate aerobic conditions. In aerobic conditions, NH_4-N, and possibly ON, could be converted to NO_{2+3}-N through nitrification, while, denitrification, the conversion of NO_{2+3}-N to N_2 gas, only occurs in anaerobic conditions.

For each of the six exfiltrate samples analyzed for NH_4-N (Events 9-14), concentrations were less than the MDL (0.1 mg/l), and concentrations were significantly different ($p \leq 0.0001$) from asphalt runoff concentrations. NO_{2+3}-N concentrations were not significantly different, but were arithmetically higher in exfiltrate than runoff. If waters infiltrating the PICP were comparable to the asphalt runoff, it could be concluded that the cell produced aerobic conditions that nitrified

NH_4-N. However, since this was a paired watershed study, conclusions cannot be made concerning pollutant transformations based on assumptions about the PICP inflow.

Concentrations of TN were not significantly different, however the p-value ($p = 0.0511$) indicates that the difference is close to significant ($p \leq 0.05$). TN is the summation of NO_{2+3}-N and TKN. On average, runoff TN concentrations were largely comprised of TKN (80%), which were comprised mostly of ON (71%), while exfiltrate TN concentrations were composed of relatively equal concentrations of TKN and NO_{2+3}-N, on average, 47% and 53%, respectively. TKN exfiltrate concentrations were significantly ($p = 0.0074$) lower than runoff concentrations, NO_{2+3}-N concentrations were not significantly different. However, Table 4 shows that, on average, NO_{2+3}-N concentrations were slightly higher in exfiltrate than runoff.

TKN is composed of ON and NH_4-N. NH_4-N concentrations were significantly ($p = 0.0003$) lower in exfiltrate than runoff, while ON concentrations were not significantly different. Exfiltrate TKN concentrations were mostly comprised of ON (89%), while runoff concentrations were 79% ON. For the six storms analyzed for NH_4-N, exfiltrate concentrations were each less than the MDL. On average, ON runoff concentrations were slightly higher than exfiltrate concentrations (Table 4).

Table 4. Goldsboro normal mean pollutant concentration and differences for samples collected from June 2003 through December 2004.

Pollutant Analysis	Runoff (mg/l)	Infiltrate (mg/l)	Difference (mg/l)	% Difference
Total Nitrogen Calculation mg/l (TN)	1.52	0.98	-0.54	-35
Nitrate-Nitrite in Water mg/l as N (NO3)	0.30	0.44	0.14	48
Total Kjeldahl Nitrorgen/Water mg/l (TKN)	1.22	0.55	-0.67	-55
Ammonia mg N/l (NH4)	0.35	0.05	-0.30	-86
Organic Nitrogen mg/l (ON)	0.88	0.54	-0.34	-39
Totoal Phoshphorus/Water mg/l (TP)	0.20	0.07	-0.13	-65
Phosphate mg P/l (PO4)	0.06	0.03	-0.04	-58
Bound Phosphorus (BP)	0.08	0.06	-0.02	-25
Total Suspended Solids mg/l (TSS)	43.8	12.4	-31.37	-72
Copper by ICP/MS-Water mg/l (Cu)	0.016	0.006	-0.01	-63
Zinc by ICP/MS-Water mg/l (Zn)	0.067	0.008	-0.06	-88

If pollutant concentrations of asphalt runoff and water infiltrating the PICP cell were assumed identical, then it could be suggested that NH_4-N and possibly some ON were nitrified into NO_{2+3}-N and the cell was functioning aerobically. However, since this was a paired watershed study, conclusions about nutrient transformations stemming from assumptions about inflow are not valid.

For 11 of 15 events NO_{2+3}-N exfiltrate concentrations were higher than runoff concentrations (Figure 9). However, for four events, runoff concentrations were higher than exfiltrate. The first two occurrences may have resulted from the cell producing anaerobic conditions. Rainfall on these dates (July 24, 2003 and August 15, 2003) had the second and third most rainfall totals during the monitoring period. Therefore, if the drainage basin remained saturated anaerobic conditions may have developed, which would have prevented NH_4-N and ON nitrification. However, if anaerobic conditions were present in the basin NO_{2+3}-N concentrations would be expected to be lower due to denitrification. Without knowing inflow concentrations, it cannot be determined whether NO_{2+3}-N concentrations were reduced by passing through the drainage basin.

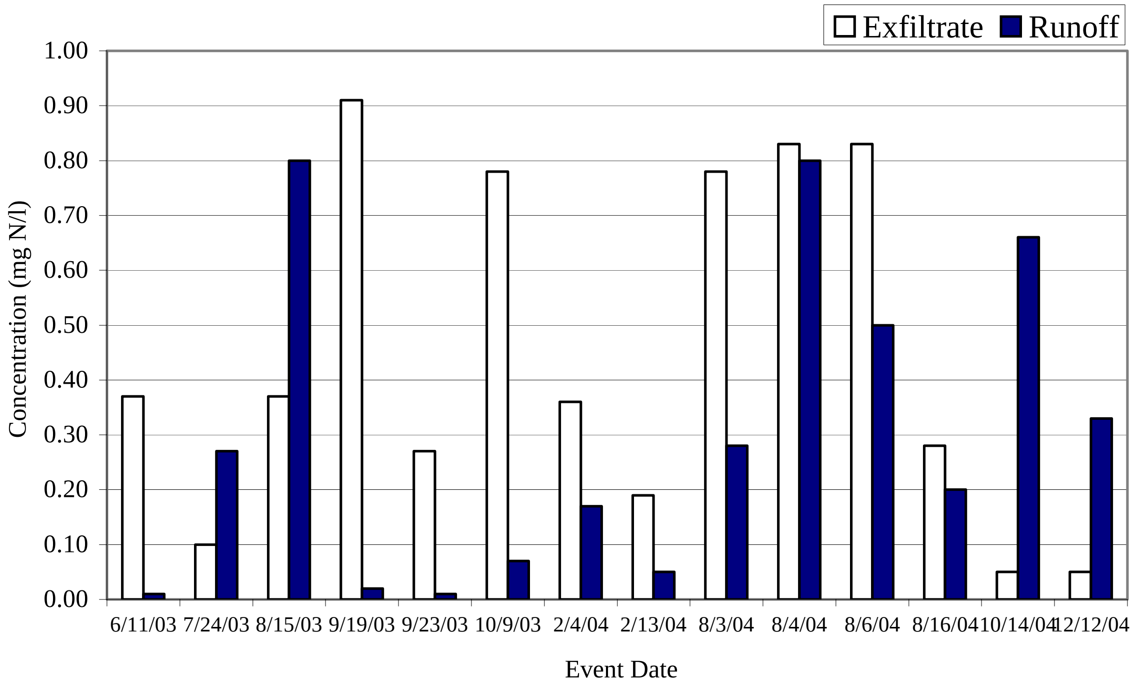

Figure 9. NO_{2+3}-N concentrations for PICP exfiltrate and asphalt runoff from Goldsboro site from June 2003 through December 2004.

Total phosphorus exfiltrate concentrations were significantly ($p = 0.0017$) less than runoff concentrations. Total Phosphorus is comprised of PO_4 and BP. TP exfiltrate and runoff concentrations each consisted of slightly more BP than PO_4; 68% and 63% BP for exfiltrate and runoff, respectively. Neither BP nor PO_4 concentrations were significantly different between exfiltrate and runoff. Therefore, reduced TP concentrations resulted from lower concentrations of each in exfiltrate. TP concentrations may have been lower due to filtering of BP and binding of PO_4 with available cations within the drainage cell.

Abnormally high concentrations of TP in both exfiltrate and runoff occurred on September 23, 2003 (Figure 10).It is possible that TP was applied in or near both areas. It is unknown whether the TP was mostly PO_4 or BP since samples from this date were not analyzed for PO_4. No other nutrient or pollutant had higher than normal concentrations from that event, therefore TP the source was not a combination of pollutants. A study sited approximately 6 km (4 mi) north of this research site from May to December, 2003, recorded TP concentrations in rainfall to be less than 0.2 mg/L. Additionally, atmospheric bulk deposition only accounts for approximately 30% of TP in runoff in North Carolina. Therefore, by eliminating atmospheric deposition as the source, the increased concentration may have resulted from vehicle deposition or fertilizer application.

The last seven runoff samples analyzed may express a trend of seasonal variation as determined by May et al (2001). Phosphorus is bound in the late spring and early fall by plant uptake and then released during the winter months (May, 2001) during die off.

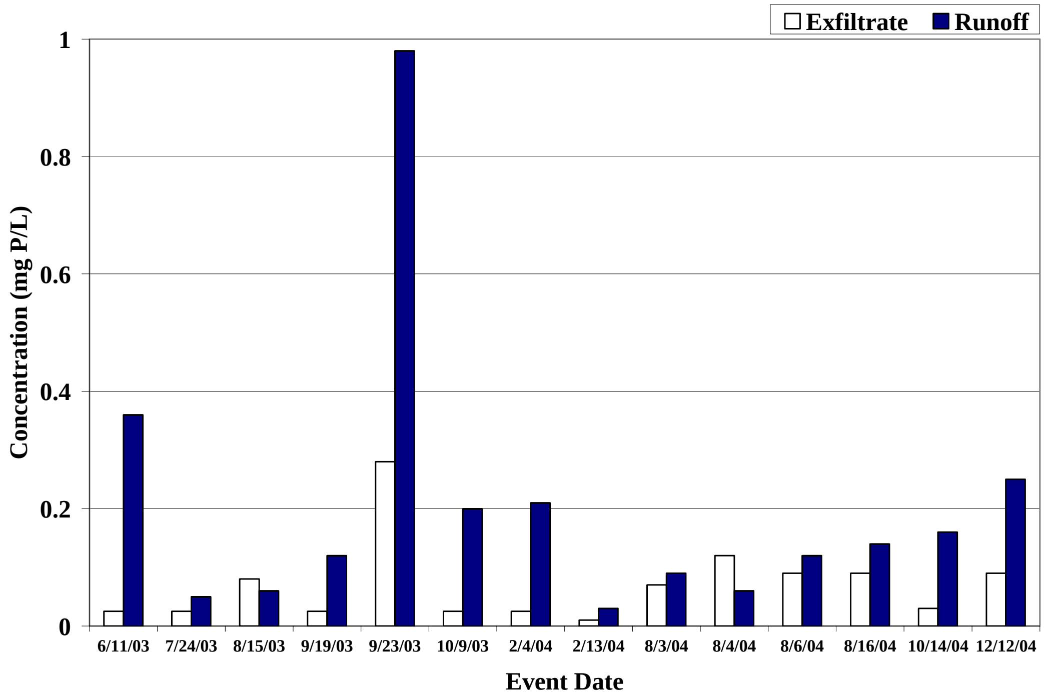

Figure 10. TP concentrations for asphalt runoff and PICP exfiltrate from Goldsboro from June 2003 through December 2004.

TSS concentrations were not significantly different between exfiltrate and runoff, however, runoff had substantially higher concentrations than infiltrate (Figure 11). Exfiltrate concentrations ranged from 0.05 – 63 mg/l, with an average concentration of 12 mg/l; comparable to concentrations reported by Pagatto (2000) (13 mg/l, mean), however, somewhat lower than levels reported by Pratt (1995) (12—160 mg/l).

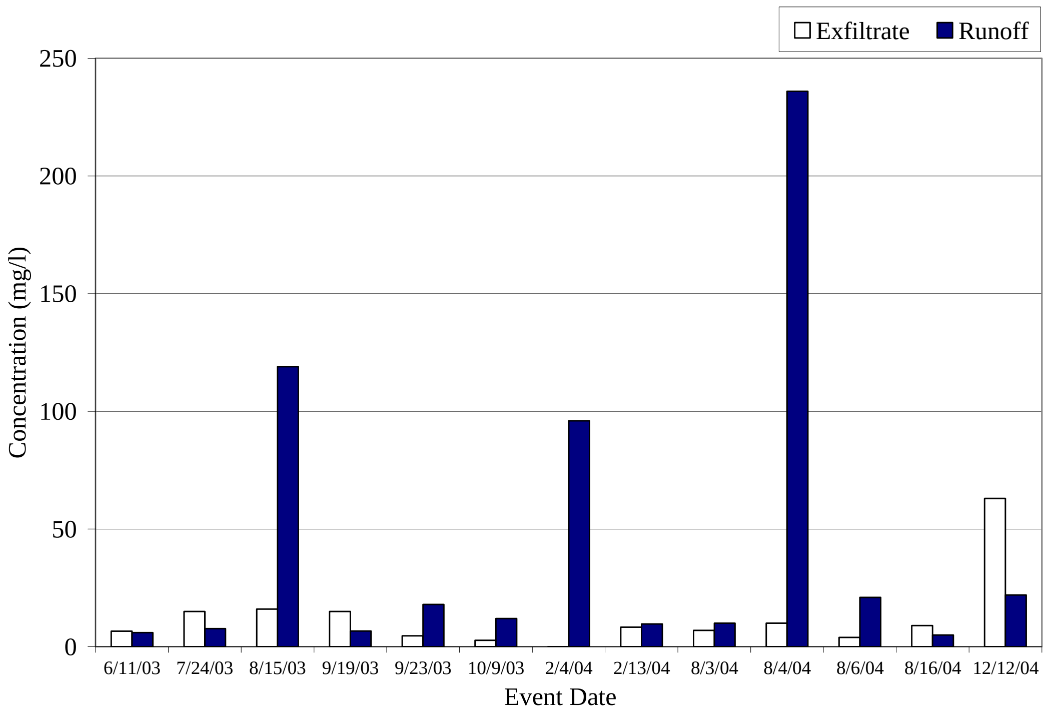

Figure 11. TSS concentrations for exfiltrate and runoff from Goldsboro site from June 2003 through December 2004.

Distributions of exfiltrate and runoff were neither normally nor log-normally distributed. Runoff concentrations had greater skewness than exfiltrate concentrations, likely due to three samples with above normal concentrations. These three events had TSS runoff concentrations of greater than 95 mg/l, while all other samples were less than 23 mg/l. Increased runoff concentrations were likely due to excessive vehicle deposition of fines. Corresponding exfiltrate TSS concentrations were not elevated, suggesting that the sediment load entering the PICP may not have been as elevated as the sediment load in the runoff. Nine of the fourteen storms, runoff concentrations were higher than exfiltrate concentrations. Although not significant, the reduced TSS concentrations, were likely due to filtering by the PICP system. Only one exfiltrate sample was above 50 mg/l, while all others were less than 20 mg/l. Also, the PICP area was parking stalls while the asphalt area was drive path. With less traffic, sediment loads entering the PICP were likely should have been less than the runoff.

Cary

The Cary site was constructed so that inflows would be entirely composed of rainfall and bulk deposition; no contributing runoff would enter onto the site. Per studies by Wu et al. (1998) in Charlotte, NC, this is a reasonable, slightly conservative assumption for TKN and NH_4. However, this assumption is extremely conservative for TP and NO_{2+3}-N and could under predict these removal rates; bulk deposition only accounted for 10 – 30% of these in runoff. Like the Goldsboro site, the Cary site was analyzed as a paired watershed, since inputs to the PICP infiltrate were a combination of bulk deposition and rainfall pollutant concentrations. Without knowing infiltrate pollutant concentrations it is difficult to make conclusions based on nutrient or sediment concentrations, thereby transformations as well, within the PICP.

From February until December 2004, exfiltrate outflow and rainfall samples were collected from the Cary site for 15 storms. Table 5 lists event dates and total rainfall depths. Rainfall depths were normally distributed for this data set. Table 6 displays water quality results from these storms. Statistical mean concentrations of inflow and outflow and distribution analysis are listed along with p-values (paired t-test (Microsoft, 2003); sign test (SAS, 2003)) to determine significance.

Table 5. Rainfall event dates and depths for samples collected from the Cary site between February 2004 and December 2004.

Event:	Date:	Rainfall cm	(in)
1	2/9/04	1.52	(0.60)
2	2/13/04	1.60	(0.63)
3	4/13/04	2.44	(0.96)
4	4/28/04	1.55	(0.61)
5	5/4/04	3.43	(1.35)
6	5/27/04	0.10	(0.04)
7	5/31/04	3.53	(1.39)
8	6/4/04	1.50	(0.59)
9	8/3/04	0.20	(0.08)
10	8/13/04	2.51	(0.99)
11	8/16/04	2.49	(0.98)
12	10/25/04	0.13	(0.05)
13	11/24/04	1.68	(0.66)
14	12/1/04	0.08	(0.03)
15	12/24/04	0.81	(0.32)

Table 6. Statistical mean pollutant concentrations and factors of significance for Cary site for samples collected from February 2004 through December 2004.

Pollutant Analysis	Rainfall (mg/l)	Exfiltrate (mg/l)	p-value
Total Nitrogen Calculation mg/l (TN)	1.62	2.13	0.4036 [LT]
Nitrate-Nitrite in Water mg/l as N (NO_{2+3}-N)	0.39	1.66	0.3018 [SN]
Total Kjeldahl Nitrogen/Water mg/l (TKN)	1.26	1.04	0.5107 [LT]
Ammonia mg N/l (NH_4-N)	0.64	0.06	**0.0005** [SN]
Organic Nitrogen mg/l (ON)	0.85	0.98	0.6673 [LT]
Total Phosphorus/Water mg/l (TP)	0.255	0.404	0.1185 [SN]
Orthophosphate mg P/l (PO_4)	0.083	0.341	**0.0352** [SN]
Bound Phosphorus (BP)	0.098	0.041	**0.0142** [LT]

Bold p-value indicates a significant difference was determined.
[LT] log-transformed student t-test (Microsoft, 2003); [SN] sign test (SAS, 2003).

Ammonia and bound phosphorus were the only pollutants significantly ($p = 0.0005$ and $p = 0.0142$, respectively) lower in exfiltrate concentrations than rainfall concentrations. PO_4 was significantly ($p = 0.352$) higher in exfiltrate concentration than rainfall concentration. On average, TP and TN concentrations were higher in exfiltrate than rainfall, substantiating that bulk deposition may not account for all TP and NO_{2+3}-N loadings.

For 11 of the 15 storms sampled, TN exfiltrate concentrations were higher than rainfall concentrations, suggesting an additional nitrogen contribution (Figure 12). Above normal concentrations of TN in rainfall (May 27 and October 25) (Figure 12) correspond with the two highest concentrations of TP in rainfall. Two of the three lowest rainfall totals occurred on these dates as well (0.10 cm (0.4 in) and 0.13 cm (0.5 in), respectively). TN exfiltrate concentrations for these two dates were due to increased NH_4 and ON (TKN) concentrations. Therefore, the source of TP and TKN was likely organic material. Higher concentrations could have also resulted from normal loadings combined with lower rainfall totals as well.

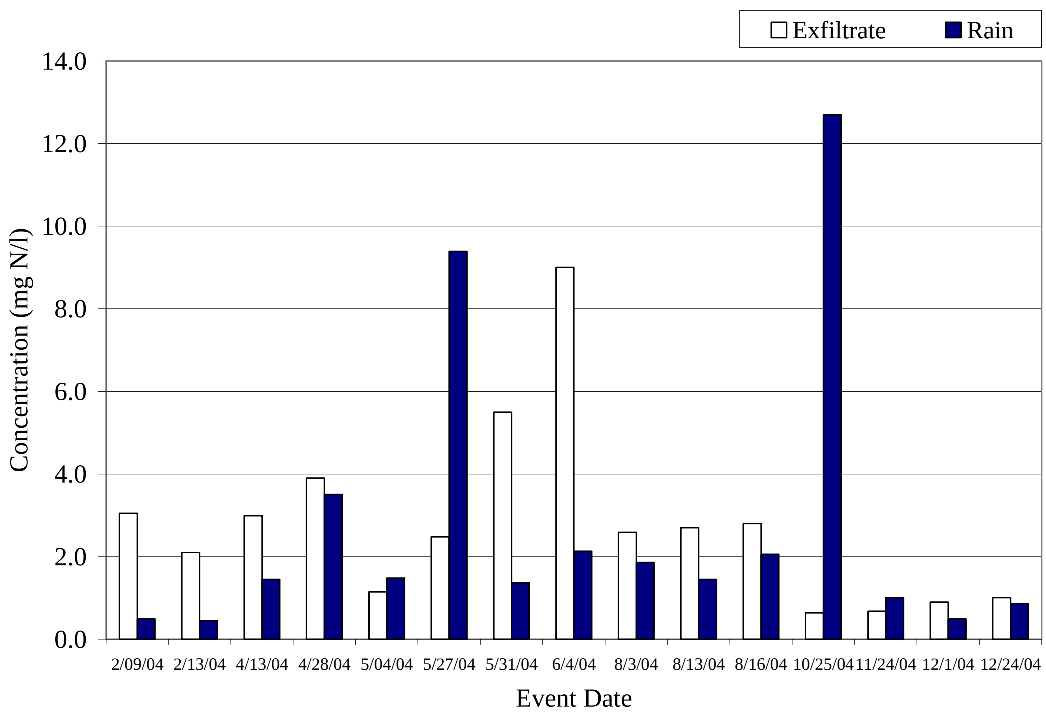

Figure 12. TN concentrations for exfiltrate and rainfall from the Cary site from February 2004 through December 2004.

Exfiltrate concentrations of NH_4-N were significantly ($p = 0.0005$) lower than rainfall concentrations. All NH_4-N exfiltrate concentrations were less than the minimum detectable level, except for the final event. If NH_4-N was consistently nitrified into NO_{2+3}-N, then cold temperatures may have inhibited bacteria from nitrifying the NH_4-N. However, without knowing infiltrate concentrations, conclusions about transformations cannot be made. In aerobic conditions NH_4-N is nitrified into NO_{2+3}-N, which could explain higher NO_{2+3}-N concentrations in exfiltrate than rainfall (Table 7). However, examination of normal means shows that the decrease of NH_4-N was less than the increase in NO_{2+3}-N (Table 7). Therefore, in addition to rainfall NO_{2+3}-N loads, an additional source of NO_{2+3}-N may have been present, which was supported by research by Wu et al. (1998), where only 10 — 30 % of NO_{2+3}-N was from bulk deposition.

Table 7. Normal mean nitrogen concentrations and percent composition for PICP exfiltrate and rainfall samples collected from the Cary site between February 2004 and December 2004.

	Rainfall		Exfiltrate	
	mg N/l	% of TN	mg N/l	% of TN
TN	2.71		2.77	
NO_{2+3}-N	0.39	14	1.66	60
TKN	2.33		1.11	
NH_4	0.64	24	0.06	2
ON	1.68	62	1.06	38

Determining whether the pavement was predominantly aerobic, anaerobic, or a combination of the two, would have largely depended on the water that remained in the storage layer after the gravel layer had drained. Hydrologic data for this site (Appendix E) suggests that the lag time between peak rainfall and peak exfiltrate may have been just over one hour for 1 cm/h (0.4 in/h) rainfall intensities. However, exfiltrate outflows may have lasted for many hours after rainfall ceased. If infiltrate remained in depressional storage in the cell, anaerobic conditions may have existed. However, NO_{2+3}-N exfiltrate concentrations were higher than rainfall concentrations, suggesting aerobic conditions. Due to: 1) a lack of data to support that anaerobic conditions were produced and 2) the design of the system was to be sloped drainage, the cell likely functioned under aerobic conditions.

For 11 of the 15 events, TP exfiltrate concentrations were higher than rainfall concentrations. Exfiltrate TP concentrations were neither normally nor log-normally distributed. Therefore a sign test (SAS, 2003) was used to determine a p-value for significance ($p = 0.1185$). Bulk deposition only accounts for 10 — 30% of TP in runoff (Wu et al., 1998). Therefore, it is possible that TP concentrations for water infiltrating the PICP had much higher TP concentrations than rainfall samples. PICP cell may have removed TP from water passing through the PICP and concentrations may have been higher than rainfall. An abnormally high TP exfiltrate concentration on June 4 resulted from extremely high PO_4 concentrations and slightly higher BP concentrations (Figures 13 and 14). The increased TP concentration corresponds with the highest NO_{2+3}-N exfiltrate concentration. This suggests the source was a concentrated source with a combination of these two nutrient forms, which may have been fertilizer.

Wu et al. (1996) showed that only 10 – 30% of TP from runoff in Charlotte, NC, could be attributed to bulk deposition. The PICP driveway was entirely constructed before the house was finished and before adjoining areas stabilized. Construction in adjacent land areas continued through late 2004. During ongoing construction in the surrounding areas, construction vehicles deposited native clay soils on the PICP surface. A surface infiltration test found a high surface infiltration rate of 4000 cm/h (1600 in/h) (Chapter 1), however, it was noted that fines were present in the voids of the PICP surface. Additionally, since a portion of the surrounding area was a manicured lawn, fertilizers intended for the lawn could have accumulated on the surface. These factors may have also contributed to higher TP concentrations.

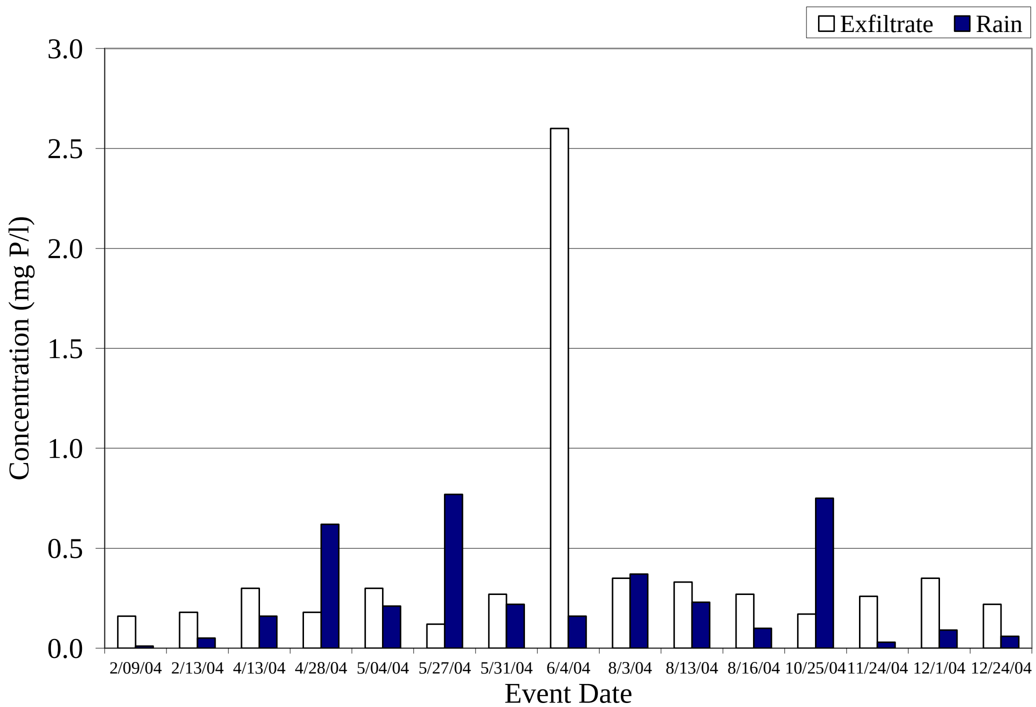

Figure 13. TP concentrations for PICP exfiltrate and rainfall samples collected from the Cary site from February 2004 through December 2004.

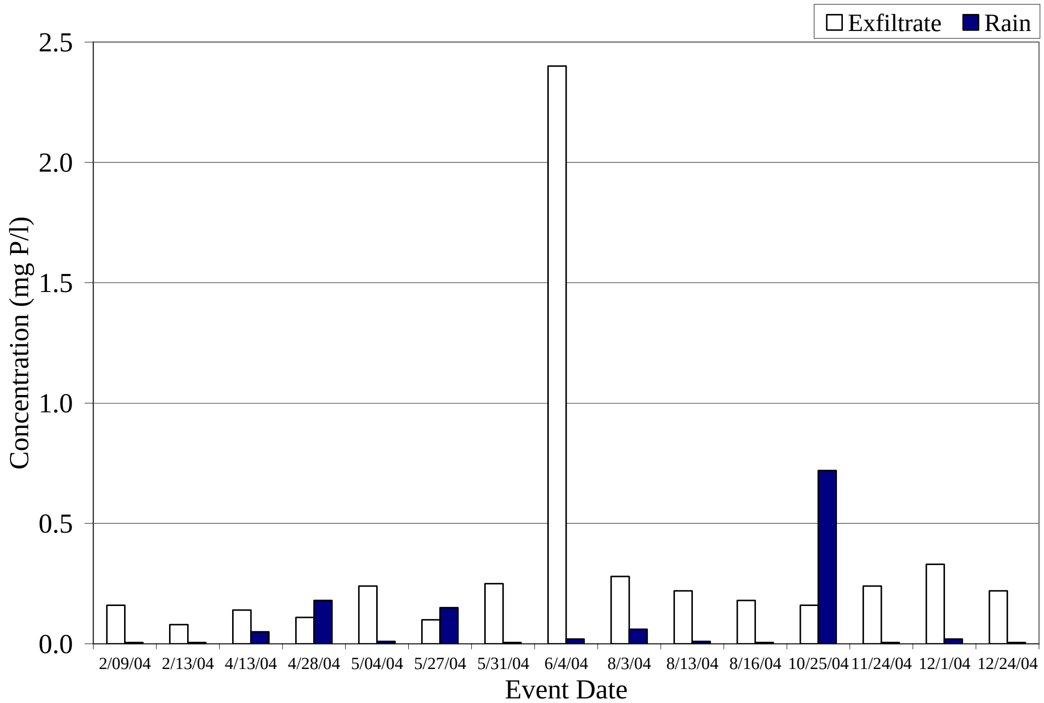

Figure 14. PO_4 concentrations for PICP exfiltrate and rainfall samples collected from the Cary site from February 2004 through December 2004.

Mean exfiltrate TSS concentrations for Goldsboro and Cary were essentially equal, 12 mg/l. This may give a predictable exfiltrate TSS concentration. TSS concentrations ranged from 1 – 31 mg/l, with no apparent pattern for fluctuations in concentrations. The most substantial difference between the Cary site and the Goldsboro site was that the Cary site was lined with an impervious geo-fabric, while the Goldsboro site was allowed to seep into the sandy subsoil. The lining may or may not have facilitated formation of anaerobic pockets, while the Goldsboro site likely had aerobic conditions throughout the pavement system.

Swansboro

The Swansboro and Goldsboro sites were similarly constructed, except that the Swansboro parking lot was constructed on a sandy soil, rather than loamy sand. Therefore, exfiltrate from Swansboro was expected to be comparable to the Goldsboro site. If runoff had been produced at the Swansboro site, it could have been analyzed as an inflow versus outflow system. However, since no runoff occurred at the Swansboro site, no runoff samples were compared to exfiltrate.

Table 8 summarizes exfiltrate concentrations and type of distribution for the data. Means for log-normal (LN) distributions were equal to 10^X, where X was the mean of the log-values. Similar to the Cary and Goldsboro sites, NH_4-N exfiltrate concentrations for each storm were less than the minimum detectable level. Other nitrogen exfiltrate concentrations were higher from Goldsboro and Cary. The mean TP concentration (0.057 mg P/l) and range (0.005 – 0.140 mg P/l) for Swansboro were comparable to the Goldsboro exfiltrate mean (0.048 mg P/l [LT]) and range (0.025 – 0.28 mg/l). These two sites were relatively free of fines (Appendix A 16 and A 25), and concentrations of TP in exfiltrate (0.005 – 0.28 mg/l) could be expected for PICP sites free of fines in sandy soil regions. Except for NH_4-N concentrations, other nitrogen concentrations were much higher for Goldsboro than Swansboro.

Table 8. Maximum, statistical mean, minimum, median and distribution of pollutant concentrations for Swansboro PICP exfiltrate samples collected between March 2004 and November 2004.

	Concentrations (mg/l)				
Pollutant	**Max**	**Mean**	**Min**	**Median**	**Distribution**
TN	0.93	0.36	0.1	0.36	N
NO_{2+3}-N	0.36	0.17	0.05	0.18	N
TKN	0.65	0.13	0.05	0.17	LN
ON	0.6	0.05	0	0.12	LN
NH_4	0.05	0.05	0.05	0.05	LN
TP	0.14	0.057	0.005	0.06	N
PO_4	0.08	0.025	0.005	0.005	N
BP	0.135	0.011	0	0.03	LN

N: Normal; LN: Log-Normal

Each exfiltrate sample had a NH_4-N concentration less that minimum detectable level. However, NO_{2+3}-N and TKN concentrations varied. Figure 15 shows exfiltrate concentrations of NO_{2+3}-N and TKN for Swansboro exfiltrate. Between August 3 and August 13, concentrations of both species drastically decreased. Decreased TN concentrations corresponded with an equally drastic increase in PO_4 concentrations of exfiltrate. The water table may have risen within the drainage layer, creating anaerobic conditions that allowed conversion of NO_{2+3}-N to N_2 gas. However, if anaerobic conditions existed, then TKN levels would not have decreased as well. The decrease in concentrations could have resulted from a seasonal effect, where vegetation suddenly bound nitrogen. Additional monitoring may determine whether this occurs annually. Another possibility is that the samples may have been contaminated before or during pollutant concentration analysis.

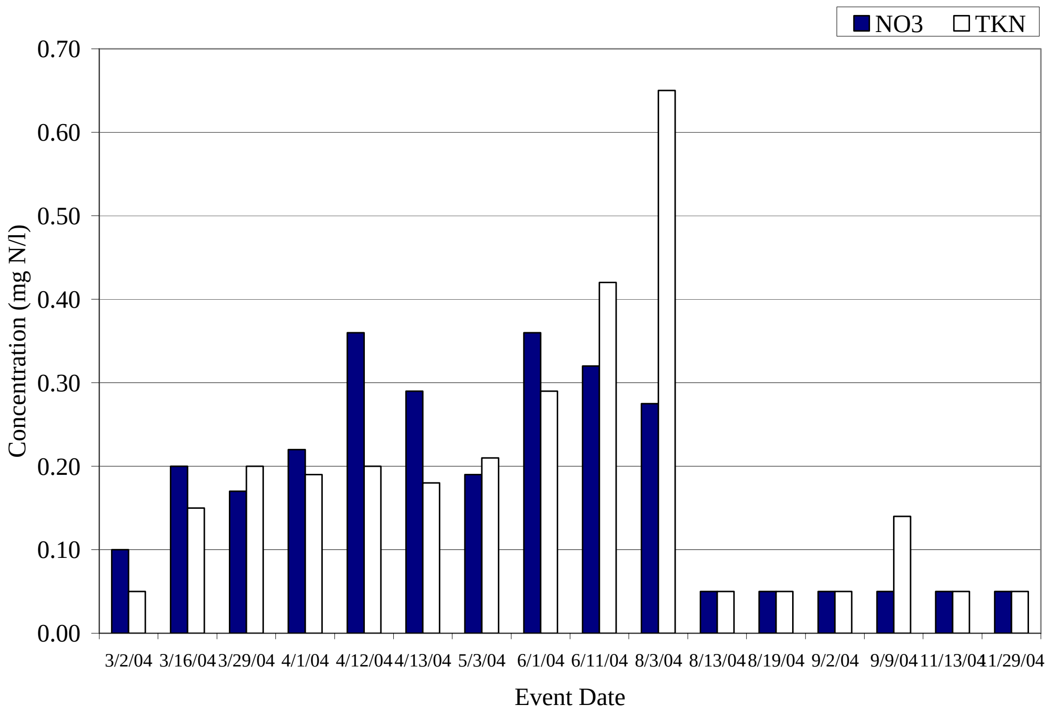

Figure 15. TKN and NO_{2+3}-N concentrations for PICP exfiltrate from the Swansboro site from March 2004 through November 2004.

PO_4 and BP concentrations drastically increased between June 1 and August 13 (Figure 16). However, increased concentrations may be due to seasonal variances as reported by May (2000). Soluble phosphorus, PO_4, would have been bound from the beginning of spring, when it was being taken up by vegetation and organisms, until early fall when phosphorus would have been released due to vegetation die off. BP concentrations may have resulted from binding of PO_4 to available cations following release. BP concentrations began increasing on June 1. The delay of increase between PO_4 and BP may have resulted from an available cation threshold; however without having analyzed exfiltrate for cation concentrations, whether cations affected PO_4 concentrations cannot be determined.

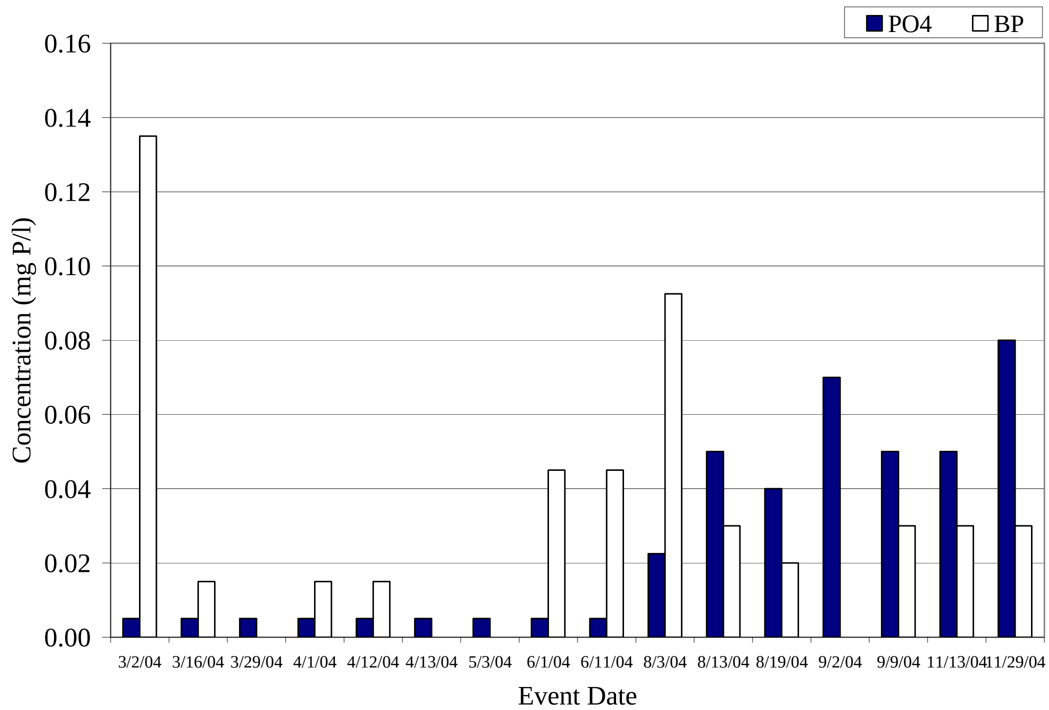

Figure 16. PO_4 and BP concentrations of PICP exfiltrate from the Swansboro site from March 2004 through November 2004.

Table 9 presents pollutant loads passing through the Swansboro PICP site. A weighted average of each pollutant ($Conc_{WA}$) was determined by rainfall totals (RF_i) for analyzed storms, concentration ($Conc_i$) and the pavement area (Area) (Equation 6).

$$Conc_{WA} = \frac{\sum_{1}^{16} RF_i * Area * Conc_i}{\sum_{1}^{16} RF_i * Area} \qquad \text{(Equation 6)}$$

Mass loads ($Mass_{MP}$) were estimated for the entire monitoring period by multiplying the weighted average of concentrations ($Conc_{WA}$) by total rainfall depth during the monitoring period (RF_{MP}) and the pavement area (Area) (Equation 7).

$$Mass_{MP} = Conc_{WA} * RF_{MP} * Area \qquad \text{(Equation 7)}$$

Over 10 months, for 0.01 ha (0.24 ac), 0.4 kg (0.9 lb) of TN and 0.06 kg (0.12 lb) of TP entered exfiltrated the site. For one complete year, the Swansboro parking lot eliminated approximately 0.31 kg (0.68 lb) or 3.8 kg/ha/yr (3.4 lbs/ac/yr) of TN and 0.04 kg (0.10 lbs) or 0.53 kg/ha/yr (0.48 lbs/ac/yr) of TP from runoff. TN loads are slightly less than predicted yields for total nitrogen (5.09 kg/ha/yr) for minimally disturbed watersheds in the United States (Lewis, 2002).

After entering the pavement, soluble pollutants likely migrated down to the water table. After they entering the water table, pollutants likely migrated towards the White Oak River. Fringe wetlands along the banks of the river could remove nitrogen through denitrification. Nitrogen and phosphorus could have also been absorbed by vegetation.

Table 9. Estimated pollutant mass reductions from runoff at the Swansboro PICP site from 16 storms from March 2004 through November 2004.

Pollutant	kg	kg/ha	kg/ha/yr	lbs	lbs/ac	lb/ac/yr
TN	0.31	3.15	3.78	0.68	2.81	3.37
TP	0.04	0.44	0.53	0.10	0.40	0.48

It would have been beneficial to monitor the water table, especially due to (1) the close proximity of the site to the White Oak River, (2) the large rainfall events over the summer and (3) the potential rise of the water table into the drainage layer. Also By not producing any runoff, the Swansboro site essentially was better than natural areas at preventing runoff and recharging the groundwater. All pollutants infiltrated and therefore did not contribute to any pollutant loadings.

CONCLUSIONS

The Swansboro flow-monitoring site had total infiltration; no runoff occurred at the Swansboro site from March 1 until December 31. This was likely a result of the site being free of fines, having a thick storage layer of washed No. 57 stone, and having a very porous sandy sub grade soil.

At both the Goldsboro and Cary sites NH_4-N exfiltrate concentrations were significantly lower than either asphalt runoff or rainfall, and for all three sites, NH_4-N exfiltrate concentrations were greater than the MDL only once out of 37 samples. For the Cary and Goldsboro sites NO_{2+3}-N exfiltrate concentrations were higher than asphalt runoff and rainfall concentrations. Therefore, both cells may have produced aerobic conditions that nitrified NH_4-N into NO_{2+3}-N. However, since neither rainfall from Cary nor runoff from Goldsboro could be assumed equal to PICP infiltrate at their respective sites, conclusions cannot be made about possible transformations within the storage layers. Rather, both sites were analyzed as paired watersheds rather than inflow versus outflow systems.

At the Goldsboro site TP, TKN, and Zn were all present in significantly ($p \leq 0.05$) lower concentrations in exfiltrate samples when compared to those of runoff. In additions, exfiltrate TN concentrations were almost significantly lower ($p = 0.0511$) and exfiltrate Cu concentrations were substantially lower than runoff concentrations. Additional sampling and analysis may result in determining significant differences. Lower Zn and Cu levels could have resulted from increased pH after infiltrate passed over the concrete pavers. Although exfiltrate TSS concentrations were not significantly less than runoff, they were substantially less. Mean exfiltrate TSS concentrations from Goldsboro and Cary sites were 12 mg/l.

At the Swansboro site in mid August, concentrations of TN were suddenly less than the minimal detectable lever, while TP concentration dramatically increased suddenly. While seasonal variance may explain changes in TP, the source of decrease in TN is unknown; possibly due to laboratory error or a seasonal effect. PO_4 may have been bound by available cations, which may have resulted in conversion to BP. TP concentrations were comparable to Goldsboro exfiltrate concentrations, while exfiltrate concentrations of ON and NO_{2+3}-N from Swansboro were lower than both Goldsboro and Cary concentrations. Swansboro site may eliminate 3.8 kg/ha/yr (3.4 lb/ac/yr) of TN and 0.53 kg/ha/yr (0.48 lb/ac/yr) of TP from runoff.

As a result of this study, siting guidelines and assessments are listed as follows:

1) To increase runoff attenuation and limit TSS exfiltrate concentrations, sites should be kept clear of fine sediment accumulation.
2) PICP sites in coastal regions can reduce runoff substantially provided they
    a) are kept free of sediment accumulation,
    b) has a several centimeter thick washed No. 57 stone drainage basin and
    c) are unlined over a highly pervious base soil.
3) a low traffic, high infiltrating coastal PICP sites could expect to eliminate 4 kg/ha/yr and 0.5 kg/ha/yr of TN and TP, respectively, from runoff.
4) lined PICP sites in clay soils may have no benefit for Total Nitrogen reduction.

5) exfiltrate from PICP may produce significantly lower concentrations of TKN, Zn, and TP compared to runoff from an asphalt surface.

6) PICP sites that are unlined in sandy soils or lined with drainpipes should not develop anaerobic conditions.

ACKNOWLEDGEMENTS

Thanks go to the North Carolina Department of Environmental Health and Natural Resources, which administers the EPA 319(h) grants, and helped fund this study. Dr. R. Smith's AP Chemistry class at Eastern Wayne High School and Town of Swansboro Public Works employees collected water quality data. Thanks goes to Jonathan Smith and David Bidelspach for his assistance and guidance throughout this study. The author would like to thank Tri-test, Inc. and the Soil Science Analytical Lab at NCSU, especially Guillermo Ramirez for sample analysis. Thanks go to Jerry Ray of Mickey's Pastry Shop and the Birklands and Rufty Homes for allowing monitoring of their site. The author would like to thank Amy Moran and Mike Regans for their help in setting up sample collection and analysis. Thanks goes to all of the property owners, town officials, and extension agents who assisted in site selection for each site. Their assistance was vital to this study.

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CHAPTER 3: SUMMARY AND RECOMMENDATIONS

SUMMARY AND RECOMMENDATIONS

PERVIOUS ANALYSIS OF PERMEABLE PAVEMENTS

Clogging is one of the main reasons permeable pavements are not fully accepted as Best Management Practices (BMPs) in North Carolina. For permeable pavements to function properly, fine sediment deposition and clogging must be limited. It has been observed that to achieve the best performance from permeable pavements, they must be:

a) sited correctly,

b) designed correctly,

c) constructed correctly, and

d) maintained correctly.

Permeable pavements should be maintained at least on a yearly rotation and more frequently if substantial accumulation of fines occurs, by either a vacuum truck or street sweeper. Hunt et al. (2001) reported that infiltration improved after using a vacuum truck on a CGP monitoring site. In chapter one it was determined that the presence of fines significantly reduced surface infiltration rates on permeable pavements tested. Simulated maintenance, by removing the top 1.3 cm (0.5 in) – 1.9 cm (0.75 in) of void space material was shown to significantly ($p \leq 0.05$) improve surface infiltration rates for concrete grid pavers (CGPs). While maintenance is essential for all permeable pavements, preventing the deposition of sediments as much as possible on permeable pavements is also important.

Permeable pavements should be sited in areas with well drained soils, preferably course sandy soil areas (Chapter 2). Use of permeable pavements in clay soil regions, for the purpose of runoff volume reduction, as noted in Chapter 2, is not recommended, due to the low infiltration rates of in-situ piedmont soils. It is also important that adjacent areas be stabilized so that runoff or traffic from these areas will not deposit sediments on the surface. However, it should be noted that permeable pavements can be employed to reduce runoff peak flows in clay soil regions.

Permeable pavements should also be constructed with a washed No. 57 stone (ASTM D448) drainage basin, typically at least 20 cm (8 in) thick, depending on runoff reduction design. They should also be constructed with less than a 5% slope, if possible, to prevent horizontal seepage (US EPA, 1999a).

Currently, permeable pavement is regarded by the State of North Carolina as 100% impervious. However, research included in Chapters 1 and 2 showed that water can infiltrate permeable pavements and they can be used to reduce runoff volumes. One challenge facing regulators regarding permeable pavement approval, is determining how much runoff reduction credit should be given to permeable pavement. However, as For example, if a parking lot is built using permeable interlocking concrete pavements (PICP), what should the reduced impervious percentage be? One way of approaching this question would be: can permeable pavement be related to a composite of grassed lawn and impervious cement or asphalt based on runoff volumes? For example if a permeable pavement site were replaced with grassed lawn and asphalt, while maintaining the same runoff characteristics of the permeable pavement, what percent of the area could be asphalt and what percent could be grassed lawn area? Thus, what ratio of grassed lawn to asphalt area produces an equivalent hydrology to permeable pavements? Answering these questions relates permeable pavements to “known” current land uses and allows permeable pavements to be models.

If runoff performance of permeable pavements can be related to a known land use, a standard of performance could be established. An ideal land use for comparison is commonly adjacent to parking areas: grassed lawns. By “converting” permeable pavements to quantifiable land uses, such as impermeable pavements and grassed lawns, permeable pavements would essentially receive runoff reduction credit.

In Chapter 1, surface infiltration tests at 48 permeable pavement sites were tested using either a single- or double-ring infiltrometer. Table 1 ranks selected sites of the 37 tested in sandy soils by their average existing surface infiltration rates from the Sandhills, Coastal Plain, and Coastal regions. Sites in clay soils were omitted because runoff volume reduction will not easily occur in these locations.

Table 1. Surface infiltration rates and ranked percentage for sites in the Sandhills, Coastal Plain and Coastal regions from Chapter 1 study.

	Surface Infiltration Rate		
Site	**cm/h**	**(in/h)**	**Ranked %**
Goldsboro PICP	4000	(1600)	3
Dough Rollers PICP	2500	(1000)	11
Harve de' Grace PICP	100	(40)	24
River Bend PICP	23	(9.1)	49
Atlantic Beach PC	14	(5.5)	51
Carrabba's CGP	7.5	(2.9)	76
FTCC I PA	5.4	(2.1)	89
Somerset Dr. PICP	1.6	(0.6)	100

Sites fell approximately at 10, 25, 50, 75, and 90% of surface infiltration rates for sites in the Sandhills, Coastal Plain, and Coastal regions.

A research study by Pitt and his colleagues tested the infiltration rate of 153 compacted and non-compacted urban sands in and around Birmingham, Alabama (US EPA, 1999b). Double-ring infiltrometers were used to measure infiltration rates of the residential vegetated compacted sands and determined an average surface infiltration rate of 6.4 cm/h (2.5 in/h). These test sites were often residential lawns. As an extremely conservative comparison, eighty-nine percent (89%) of permeable pavements tested (Chapter 1) had surface infiltration rates of at least 5.4 cm/h (2.1 in/h), 84% of the residential lawn rate. Therefore, 89% of permeable pavements tested had almost as high, or higher, infiltration rates as the grassed sandy lawns tested by Pitt et al. Curve numbers and Rational Coefficients for fully developed open spaces with <50% grass, assumed compacted, for A and B soils are listed in Table 2 below.

Table 2. Curve numbers and rational coefficients for fully developed open spaces with <50% grass for A and B soils.

Soil	**SCS CN**	**Rational C**
A	68	0.36
B	79	0.58

A limited study in New Jersey examined infiltration rates of different land uses. Eight sites were tested to determine infiltration rates using double-ring infiltrometers with USCS soil classifications of either SW, SP, SM, or a combination. Woods and pasture sites had infiltration rates of 38 cm/h (15 in/h) and 25 cm/h (9.9 in/h), respectively; which fall between the 50 and 75% infiltration rate of permeable pavements tested. However, for two subdivision lawns, mean infiltration rates were 0.36 cm/h (0.14 in/h) and 0.08 cm/h (0.03 in/h). The difference between the undisturbed sites and subdivision lawns was two orders of magnitude, however subdivision lawns are assigned curve numbers and Rational Coefficients listed in Table 2 (Malcom, 1989).

Another way to relate permeable pavements to other land uses is to examine long-term data collected at permeable pavement sites. This data contrasts with surface infiltration rates which are only “snapshots” of surface infiltration. Data from the two monitoring sites from Chapter 2, along with data collected by Hunt and Stephens (2002) and unpublished data by Bidelspach and Hunt (2004) from two additional sites were analyzed to determine Rational Coefficients to relate to quantifiable land uses.

The Swansboro monitoring site, detailed in Chapter 2, produced no runoff between February and December of 2004. The largest rainfall event was 8.9 cm (3.5 in) and four events were larger than 5 cm (2 in) during that period. From Chapter 2, the calculated Curve Number and Rational Coefficient for the site were 44 and 0, respectively.

The quality of data from the Cary monitoring site was marginal at best. A conservative mean Rational Coefficient of 0.44 was calculated based on this data. Rainfall data were not sufficient to determine an SCS Curve number.

The site monitored by Hunt and Stephens was located in Kinston and was split between the two tested sites Kinston CGP and Kinston GP. Forty-eight rainfall events occurred during the monitoring period; 11 produced runoff. Rainfall depths ranged from 1.5 cm (0.6 in) – 36.6 cm (14.4 in). SCS Curve Numbers (CN) were calculated, by back-calculating through the SCS Curve Number method (Equation 1 and 2) for each event.

$$S = \frac{1000}{CN} - 10 \qquad \text{(Equaion 1)}$$

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad \text{(Equation 2)}$$

The initial abstraction (S) is a function of the Curve Number, while runoff depth (Q) is a function of both the initial abstraction and precipitation depth (P). The median CN for the Kinston site for all storms producing runoff was 88. This number was then related to a corresponding Rational Coefficient of 0.76 (Malcom, 1989). Each rainfall event was also modeled using the SCS Small Watershed method (NRCS, 1986). An equivalent ratio of grassed area (61 CN) to impervious pavement (98 CN) was modeled to produce the same runoff as the permeable pavement for each event. The median ratio for all events at the Kinston site was was 71% grassed, with 29% impervious area.

Unpublished data was collected from the McCrary Park porous concrete site in Wilmington were analyzed in the same way Kinston data were, except only rainfall events of greater than 2.5 cm

(1.0 in) were analyzed to calculate CNs. The median CN for the site was determined to be 85, with a corresponding Rational Coefficient of 0.70. This permeable pavement performed as if it were a composite of 33% grassed area and 67% impervious area, with regards to runoff volumes. It should be noted that this site was constructed with no gravel base for storage; a 10 cm (4 in) layer of porous concrete was laid directly on to in-situ soil.

Table 3 summarizes results from the four monitoring sites and surface infiltration test comparisons. Rational coefficients are approximately equivalent to the part of rainfall that becomes runoff, therefore, lower values correspond to less runoff.

Table 3. Summary table of curve numbers, rational coefficients, and equivalent percent grassed area from five different permeable pavement performance comparisons along with impervious surfaces.

	Surface Infiltration	Cary	Swansboro	Kinston	Wilmington	Standard Impervious
SCS Curve Number			36	88	85	98
A Soil	68					
B Soil	79					
Rational C		0.44	0.00	0.76	0.70	0.96
Equivalent % Grassed	84		>100	71	33	0

It can be seen that for each long term monitoring site and the surface infiltration study that permeable pavements have substantially lower CNs and Rational coefficients than traditional impervious surfaces (98 and 0.96, respectively). Permeable pavements produce substantially lower runoff volumes than impermeable pavements, as also evidenced by the equivalent % grassed. Therefore, using permeable pavements, which are more pervious than traditional concrete and asphalt surfaces, results in reduced runoff volumes. By reducing impervious areas, stormwater structure sizes can be reduced as well. For example, if part of an impervious parking lot was replaced with permeable pavement, runoff volumes would be reduced. Therefore, the treatment volume would be reduced for stormwater runoff structures; thus the size of structures could be reduced. A detailed description of a study analyzing the effect of imperviousness on stormwater detention pond size is found in Appendix H.

It should be noted that different permeable pavements perform differently. Initially, grouping permeable pavements together may work well; however, as more research becomes available, individual types of permeable pavements, such as PA, PICP, CGP, etc., may be treated individually based on paver-type.

RECOMMENDATIONS

If the State of North Carolina does decide to assign BMP or pervious area credit to permeable pavements in the near future, recommended credit should initially be conservative. The lowest percent of equivalent grassed area of five analyses was 33% (the Wilmington site). This may be an acceptable initial value for pervious area credit for permeable pavements with in-situ sandy soils. For sites with gravel bases in sandy soil areas, a credit of up to 70% credit for grassed area may be given. As more permeable pavement research data becomes available, the standard can be adjusted.

The research studies presented here in all three chapters have provided valuable information to further understand factors affecting performance of permeable pavements for both water quantity and water quality. While results from this study for long-term hydrologic performance of PICP were mixed, the data did lead to several trends and key findings. One of which was that PICP can be used, if sited, designed, constructed, and maintained properly, as a runoff reducing alternative to traditional impervious surfaces. Reducing runoff rates from traditional pavements would reduce land areas needed for water quality treatment, such as stormwater detention ponds (Appendix H).

Recommendations for Future Research

There are numerous possibilities for future permeable pavement research. The following are suggestions of new or continued permeable pavement research topics.

Additional data is needed from the Cary monitoring site to help determine the effectiveness of permeable pavements in clay soils. In addition, another study involving runoff reduction by installation of PICP in clay soils compared to traditional asphalt and concrete surfaces could provide a better picture of permeable pavement effectiveness in clay soils. Recording water level rise within the storage basin could also determine the effect of the underlying gravel storage basin has on peak delay and reduction. Findings could be applied to permeable pavement modeling to predict exfiltrate flows and draw down times.

There is a great need to assign SCS curve numbers and Rational Coefficients to different types of pavements as well. Plot studies could be utilized to determine these values and later scaled up to field studies.

Creating an anaerobic zone in the drainage basin of permeable pavements could potentially lead to higher TN removal rates. However, the anaerobic zone may increase soluble TP concentrations that were initially bound to sediments. Monitoring of pH for runoff, infiltrate, exfiltrate and/or anaerobic zones could lead to determining species of additional pollutants throughout the removal process. In addition to pH, cation concentraitons, which could be released from cement and concrete pavements, could be monitored which may help determine the effect of cations on pollutant concentrations.

As permeable pavement becomes more common an evaluation procedure for the infiltration volumes should be developed to ensure they are properly maintained. Using a double- or single-ring infiltrometer could be the basis for development of such a procedure. Also, there could be a relationship between the wetted area after a single-ring infiltrometer test and infiltration performance of the site.

Additionally, a study could examine the effect of maintenance, by use of a vacuum truck or street sweeper, through a paired study where one side is maintained and the other is not. These two areas would have to be separated to isolate flows, but this could lead to more specific guidelines regarding maintenance frequency, rather than arbitrarily once or twice per year.

Permeable pavements are a unique and valuable type of BMP. It is one of the only BMPs that does not necessarily require additional land area. Impervious parking lots can treat, mitigate, and store stormwater using the same area set aside for their parking. As land costs continue to rise, permeable pavements will become an economic tool in the future.

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APPENDIX A: SURFACE INFILTRATION RATE SITE SUMMARIES

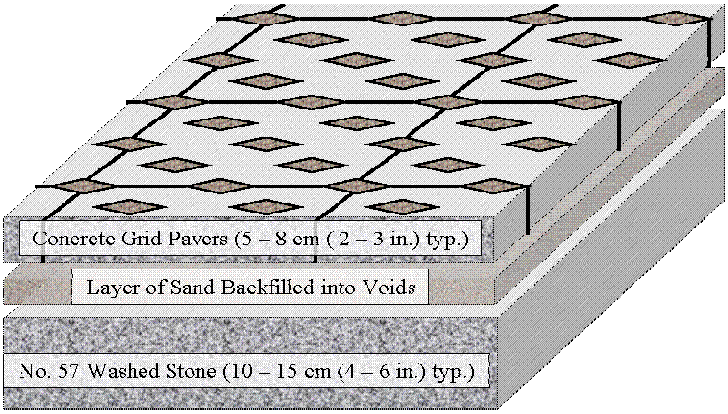
Table 1a. Surface Infiltration Rate Test Sites and Appendices.

Site	Appendix
Atlantic Station High	A1
Indian Beach Access	A2
Blackman	A3
Conch	A4
Municipal Building	A5
Gull	A6
Glidden	A7
Carrabba's	A8
Govenor	A9
Atlantic Station Low	A10
Epstein	A11
Bainbridge	A12
Loggerhead	A13
Hargrove	A14
Cary Public Works	A15
Goldsbor Monitorig Site	A16
CVS Pharmacy	A17
Wal-Mart	A18
Dough Rollers	A19
Swansboro Community Parking Lot	A20
Captiva Bay Condos	A21
Naval Medical Center Walkway	A22
Baywoods	A23
Harve de' Grace	A24
Penny Road PICP	A25
Naval Medical Center Parking Lot	A26
River Bend	A27
Boat Ramp	A28
Somerset Dr.	A29
Catawba College	A30
Loflin Concrete	A31
Bailey's Landing II	A32
Penny Road PC	A33
Friday Center Park and Ride PC	A34
Ready Mix Lab	A35
Bryarton I	A36
McCrary Park	A37
Atlantic Beach PC	A38
Bailey's Landing	A39
Wrightsville Beach Catholic Church	A40
Friday Center Park and Ride PA	A41

Table 1b. Surface Infiltration Rate Test Sites and Appendices.

Atlantic Beach PA I	A42
Fayetteville Technical Community College I	A43
Fayetteville Technical Community College II	A44
Atlantic Beach PA II	A45
Kinston CGP	A46
Kinston GP	A47
Wynn Plaza	A48

Site:	Town of Nags Head Municipal Building
Type of Surface:	Concrete Grid Pavers
Use:	Parking for beach access, library, and municipal building



Pervious Area:	900 sq. m 9700 sq. ft.	30 stalls
Approximate Drainage Area:	1300 sq. m 14000 sq. ft.	
Construction Date:	1986	
Address:	916 Fort Macon Rd., Atlantic Beach, NC	
Test Date:	November 5, 2003	
Visual Assessment:	Grass, oil and sediment accumulation in voids. .	
Maintenance Practice:	None	

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
7.2	19.4	29.4	18.7	cm/hr
2.8	7.6	11.6	7.4	in./hr

Post-Maintenance Condition Test:

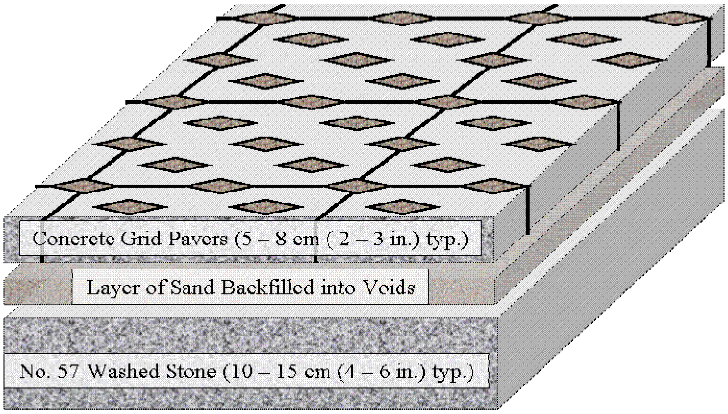
Run #1	Run #2	Run #3	Average	
41.9	17.5	37.1	32.2	cm/hr
16.5	6.9	14.6	12.7	in./hr

Maintenance Performed: Removed top 1.3 cm (0.5 in.) of material from drainage pores.

Site: Indian Beach Beach Access

Type of Surface: Concrete Grid Pavers

Use: Parking for beach access.



Pervious Area: 370 sq. m, 4000 sq. ft. — 20 stalls

Approximate Drainage Area: 740 sq. m, 8000 sq. ft.

Construction Date: 1986

Address: Highway 58, Indian Beach, NC

Test Date: November 5, 2003

Visual Assessment: Grass in voids, fines only on surface of blocks in low lying areas.

Maintenance Practice: None

Existing Conditions Test:

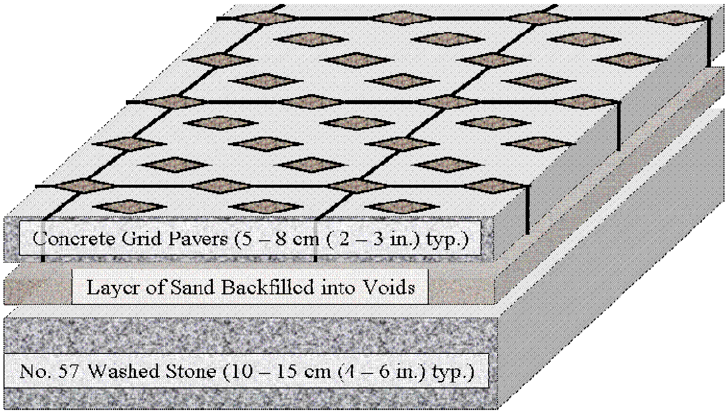
Run #1	Run #2	Run #3	Average	
17.5	16.2	15.5	16.4	cm/hr
6.9	6.4	6.1	6.5	in./hr

Post-Maintenance Condition Test:

Run #1	Run #2	Run #3	Average	
26.7	38.5	16.9	27.4	cm/hr
10.5	15.2	6.7	10.8	in./hr

Maintenance Performed: Removed top 1.3 cm (0.5 in.) of material from drainage pores.

Site:	Blackman Beach Access
Type of Surface:	Concrete Grid Pavers
Use:	Beach Access Parking



Pervious Area:	620 sq. m 6700 sq. ft.	17 stalls
Approximate Drainage Area:	810 sq. m 8700 sq. ft.	
Construction Date:	1985	
Address:	S. Virginia Dare Tr. & Blackman St., Nags Head, NC	
Test Date:	July 17, 2003	
Visual Assessment:	Grass in 30% of voids, very sandy and has pine trees.	
Maintenance Practice:	As needed after large storms (a few times per year).	

Existing Conditions Test:

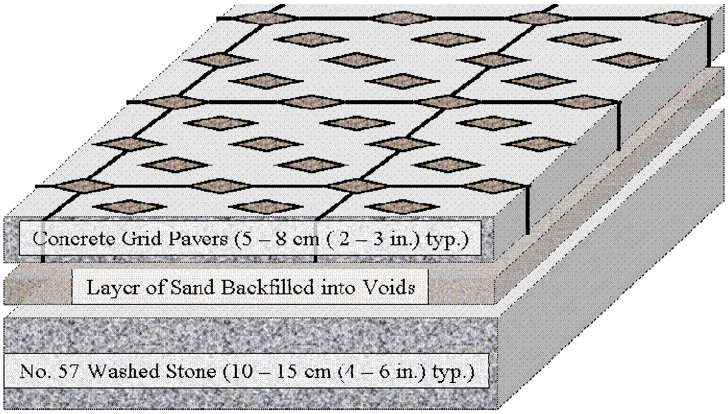
Run #1	Run #2	Run #3	Average	
7.7	8.8	22.2	12.9	cm/hr
3.0	3.5	8.8	5.1	in./hr

Post-Maintenance Condition Test:

Run #1	Run #2	Run #3	Average	
7.1	7.9	5.2	6.7	cm/hr
2.8	3.1	2.0	2.6	in./hr

Maintenance Performed: Removed top 1.9 cm (0.75 in.) of material from drainage pores.

Site:	Conch Beach Access
Type of Surface:	Concrete Grid Pavers
Use:	Beach Access Parking



Pervious Area:	760 sq. m 8200 sq. ft.	20 stalls
Approximate Drainage Area:	870 sq. m 9400 sq. ft.	
Construction Date:	1984	
Address:	S. Virginia Dare Tr. & Conch St., Nags Head, NC	
Test Date:	July 16, 2003	
Visual Assessment:	Very loose sand.	
Maintenance Practice:	As needed after large storms (a few times per year).	

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
7.5	15.8	4.2	9.2	cm/hr
3.0	6.2	1.7	3.6	in./hr

Post-Maintenance Condition Test:

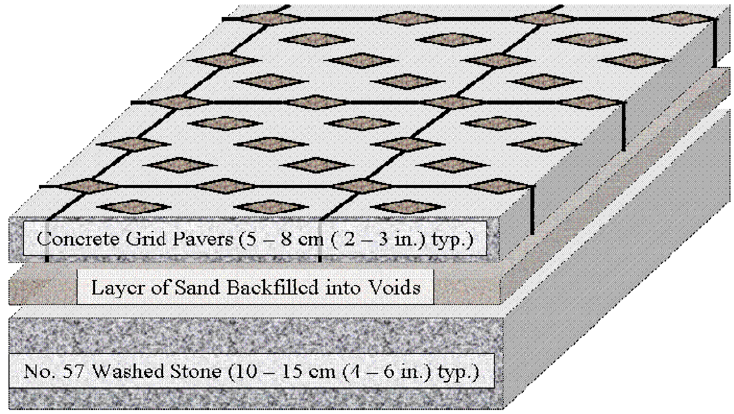
Run #1	Run #2	Run #3	Average	
6.2	8.7	14.9	10.0	cm/hr
2.5	3.4	5.9	3.9	in./hr

Maintenance Performed: Removed top 1.3 cm (0.5 in.) of material from drainage pores.

Site: Town of Nags Head Municipal Building

Type of Surface: Concrete Grid Pavers

Use: Parking for beach access, library, and municipal building



Pervious Area: 380 sq. m, 4100 sq. ft. — 13 stalls

Approximate Drainage Area: 560 sq. m, 6000 sq. ft.

Construction Date: 1982

Address: S. Virginia Dare Trail & Municipal Complex, Nags Head, NC

Test Date: July 14, 2003

Visual Assessment: Grassed voids, fines only on surface of blocks in low areas.

Maintenance Practice: As needed after large storms (a few times per year).

Existing Conditions Test:

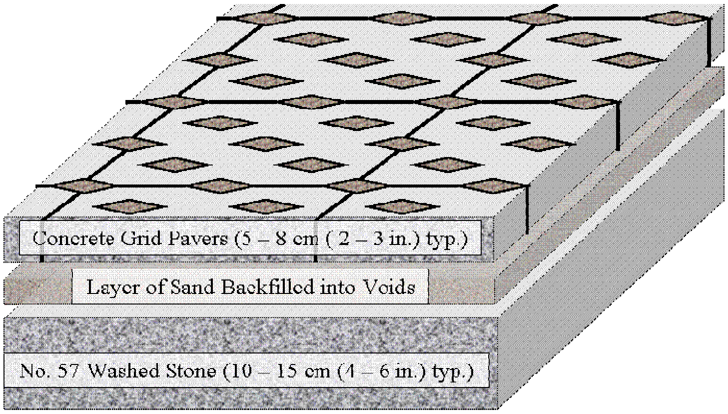
Run #1	Run #2	Run #3	Average	
8.6	8.9	6.3	7.9	cm/hr
3.4	3.5	2.5	3.1	in./hr

Post-Maintenance Condition Test:

Run #1	Run #2	Run #3	Average	
57.5	7.1	15.1	26.6	cm/hr
22.6	2.8	6.0	10.5	in./hr

Maintenance Performed: Removed top 1.3 cm (0.5 in.) of material from drainage pores.

Site:	Gull Beach Access
Type of Surface:	Concrete Grid Pavers
Use:	Beach Access Parking



Pervious Area:	650 sq. m 7000 sq. ft.	12 stalls
Approximate Drainage Area:	750 sq. m 8100 sq. ft.	
Construction Date:	1985	
Address:	S. Virginia Dare Trail and Gull St., Nags Head, NC	
Test Date:	July 15, 2003	
Visual Assessment:	Sand in voids and some grass.	
Maintenance Practice:	As needed after large storms (a few times per year).	

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
2.4	4.0	8.9	5.1	cm/hr
1.0	1.6	3.5	2.0	in./hr

Post-Maintenance Condition Test:

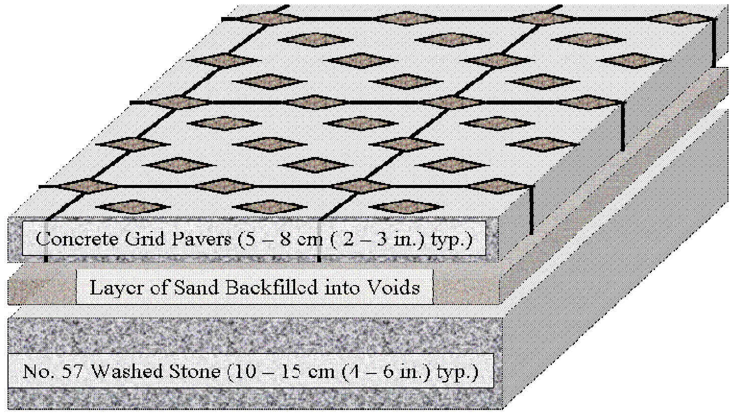
Run #1	Run #2	Run #3	Average	
5.5	7.5	9.1	7.3	cm/hr
2.1	2.9	3.6	2.9	in./hr

Maintenance Performed: Removed top 1.9 cm (0.75 in.) of material from drainage pores.

Site: Glidden Beach Access

Type of Surface: Concrete Grid Pavers

Use: Beach Access Parking



Pervious Area: 470 sq. m
5100 sq. ft.

Approximate Drainage Area: 1000 sq. m
11000 sq. ft.

Construction Date: 1985

Address: S. Virginia Dare Tr. & Glidden St., Nags Head, NC

Test Date: July 16, 2003

Visual Assessment: Surface is caked, silty, trash and grass in 75% of voids.

Maintenance Practice: As needed after large storms (a few times per year).

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
5.5	6.3	3.1	5.0	
2.1	2.5	1.2	2.0	in./hr

Post-Maintenance Condition Test:

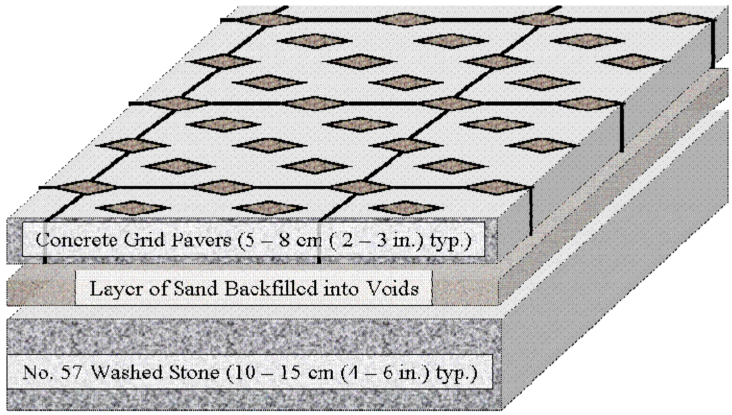
Run #1	Run #2	Run #3	Average	
10.5	4.9	7.1	7.5	
4.1	1.9	2.8	3.0	in./hr

Maintenance Performed: Removed top 1.9 cm (0.75 in.) of material from drainage pores.

Site: Carrabba's Resteraunt

Type of Surface: Concrete Grid Pavers

Use: Overflow parking for multiple resteraunts



Pervious Area: 170 sq. m 14 stalls
1800 sq. ft.

Approximate Drainage Area: 500 sq. m
5400 sq. ft.

Construction Date: 1999

Address: 15 Van Campen Blvd., Wilmington, NC

Test Date: July 23, 2003

Visual Assessment: Grass in 60-80% of void spaces.

Maintenance Practice: None.

Existing Conditions Test:

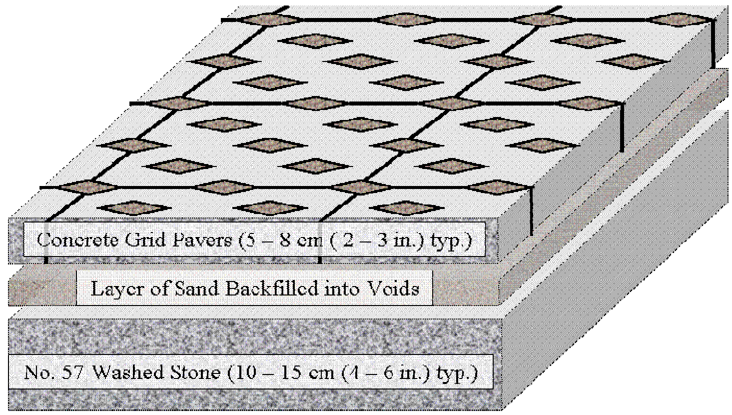
Run #1	Run #2	Run #3	Average	
6.1	3.6	5.1	4.9	cm/hr
2.4	1.4	2.0	1.9	in./hr

Post-Maintenance Condition Test:

Run #1	Run #2	Run #3	Average	
9.2	5.9	7.3	7.5	cm/hr
3.6	2.3	2.9	2.9	in./hr

Maintenance Performed: Removed top 1.3 cm (0.5 in.) of material from drainage pores.

Site:	Govenor Beach Access
Type of Surface:	Concrete Grid Pavers
Use:	Beach Access Parking



Pervious Area:	620 sq. m 6700 sq. ft.	17 stalls
Approximate Drainage Area:	680 sq. m 7300 sq. ft.	
Construction Date:	1984	
Address:	S. Virginia Dare Tr. & Govenor St., Nags Head, NC	
Test Date:	July 17, 2003	
Visual Assessment:	Grass coverage varies from 20-70% and noticeable oil spots.	
Maintenance Practice:	As needed after large storms (a few times per year).	

Existing Conditions Test:

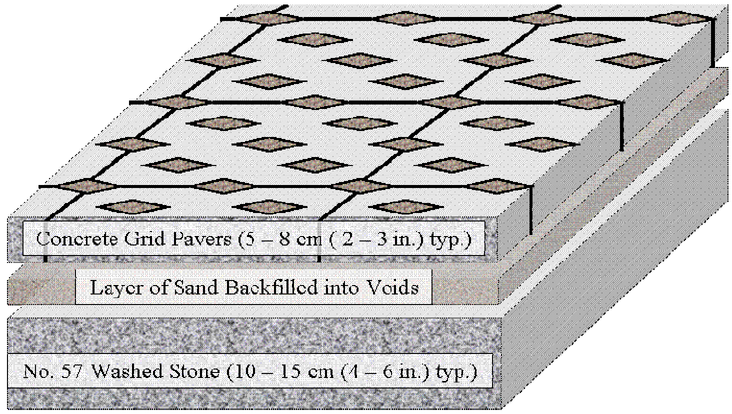
Run #1	Run #2	Run #3	Average	
4.1	4.3	5.6	4.6	cm/hr
1.6	1.7	2.2	1.8	in./hr

Post-Maintenance Condition Test:

Run #1	Run #2	Run #3	Average	
6.7	6.4	12.6	8.6	cm/hr
2.6	2.5	5.0	3.4	in./hr

Maintenance Performed: Removed top 1.9 cm (0.75 in.) of material from drainage pores.

Site:	Town of Nags Head Municipal Building
Type of Surface:	Concrete Grid Pavers
Use:	Parking for beach access, library, and municipal building



Pervious Area:	900 sq. m 9700 sq. ft.	30 stalls
Approximate Drainage Area:	1300 sq. m 14000 sq. ft.	
Construction Date:	1986	
Address:	916 Fort Macon Rd., Atlantic Beach, NC	
Test Date:	June 23,2004	
Visual Assessment:	Severe caking of oil and sediments in void spaces.	
Maintenance Practice:	None	

Existing Conditions Test:

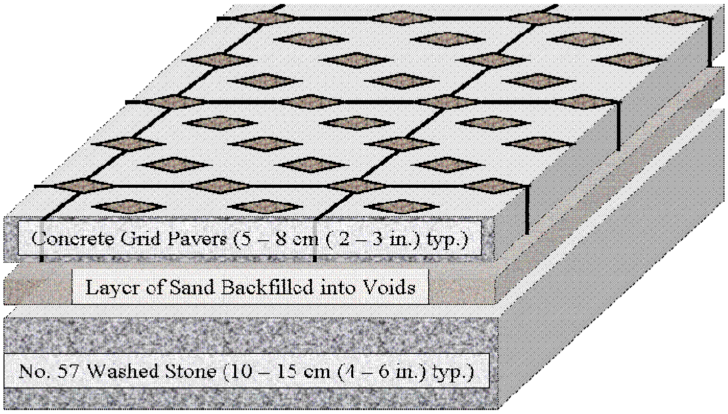
Run #1	Run #2	Run #3	Average	
4.5	5.1	3.6	4.4	cm/hr
1.8	2.0	1.4	1.7	in./hr

Post-Maintenance Condition Test:

Run #1	Run #2	Run #3	Average	
34.5	32.1	25.1	30.6	cm/hr
13.6	12.6	9.9	12.0	in./hr

Maintenance Performed: Removed top 1.3 cm (0.5 in.) of material from drainage pores.

Site:	Epstein Beach Access
Type of Surface:	Concrete Grid Pavers
Use:	Beach Access Parking



Pervious Area:	1100 sq. m 12000 sq. ft.
Approximate Drainage Area:	1100 sq. m 12000 sq. ft.
Construction Date:	1984
Address:	S. Virginia Dare Trail & E. Blue Water Dr., Nags Head, NC
Test Date:	July 15, 2003
Visual Assessment:	Sand and some small grass plants in void spaces.
Maintenance Practice:	As needed after large storms (a few times per year).

Existing Conditions Test:

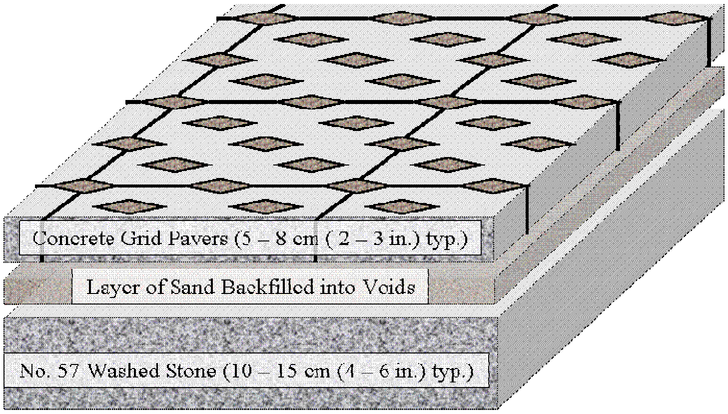
Run #1	Run #2	Run #3	Average	
1.1	9.0	2.6	4.2	cm/hr
0.4	3.5	1.0	1.7	in./hr

Post-Maintenance Condition Test::

Run #1	Run #2	Run #3	Average	
7.6	2.7	18.6	9.7	cm/hr
3.0	1.1	7.3	3.8	in./hr

Maintenance Performed: Removed top 1.3 cm (0.5 in.) of material from drainage pores.

Site:	Bainbridge Beach Access
Type of Surface:	Concrete Grid Pavers
Use:	Beach Access Parking



Pervious Area:	570	sq. m	16 stalls
	6100	sq. ft.	
Approximate Drainage Area:	910	sq. m	
	9800	sq. ft.	
Construction Date:	1984		

Address:	S. Virginia Dare Tr. & Bainbridge St., Nags Head, NC
Test Date:	July 16, 2003
Visual Assessment:	Very sandy with grass in 30% of voids.
Maintenance Practice:	As needed after large storms (a few times per year).

Existing Conditions Test:

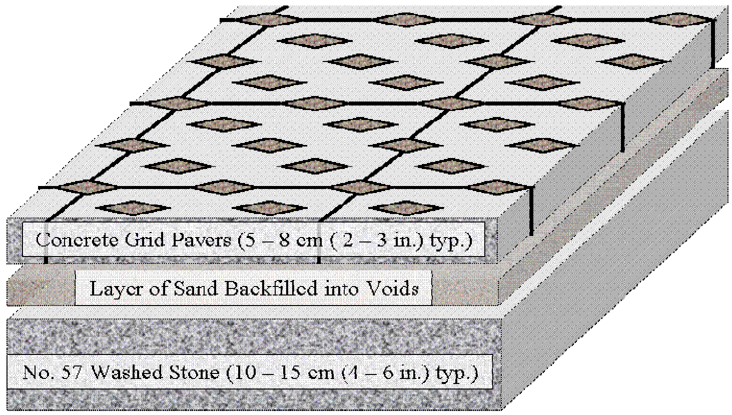
Run #1	Run #2	Run #3	Average	
4.0	2.7	5.9	4.2	cm/hr
1.6	1.1	2.3	1.6	in./hr

Post-Maintenance Condition Test:

Run #1	Run #2	Run #3	Average	
4.0	4.5	5.4	4.6	cm/hr
1.6	1.8	2.1	1.8	in./hr

Maintenance Performed: Removed top 1.9 cm (0.75 in.) of material from drainage pores.

Site:	Loggerhead Beach Access
Type of Surface:	Concrete Grid Pavers
Use:	Beach Access Parking



Pervious Area:	800 sq. m 8600 sq. ft.	15
Approximate Drainage Area:	1100 sq. m 12000 sq. ft.	
Construction Date:	1984	
Address:	S. Virginia Dare Rd., Nags Head, NC	
Test Date:	July 15, 2003	
Visual Assessment:	Fairly well maintained, some small sprigs in voids.	
Maintenance Practice:	As needed after large storms (a few times per year).	

Existing Conditions Test:

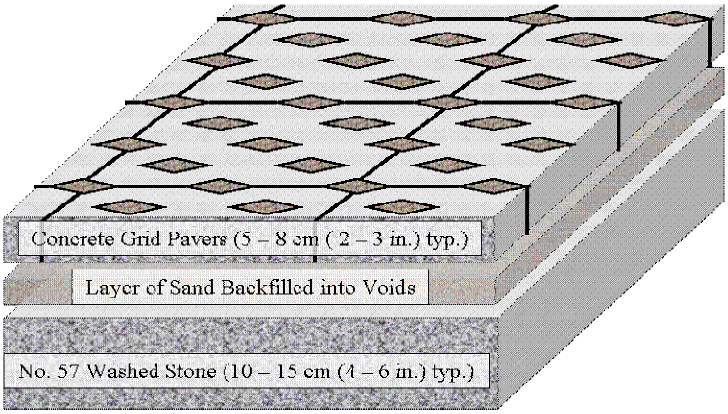
Run #1	Run #2	Run #3	Average	
4.3	4.0	2.5	3.6	cm/hr
1.7	1.6	1.0	1.4	in./hr

Post-Maintenance Condition Test:

Run #1	Run #2	Run #3	Average	
7.2	4.9	15.8	9.3	cm/hr
2.8	1.9	6.2	3.7	in./hr

Maintenance Performed: Removed top 1.3 cm (0.5 in.) of material from drainage pores.

Site:	Hargrove Beach Access
Type of Surface:	Concrete Grid Pavers
Use:	Beach Access Parking



Pervious Area:	820 sq. m 8800 sq. ft.	22 stalls	
Approximate Drainage Area:	820 sq. m 8800 sq. ft.		
Construction Date:	1984		
Address:	S. Virginia Dare Tr. & Hargrove St., Nags Head, NC		
Test Date:	July 17, 2003		
Visual Assessment:	Grass in 60% of voids and looks very sandy.		
Maintenance Practice:	As needed after large storms (a few times per year).		

Existing Conditions Test:

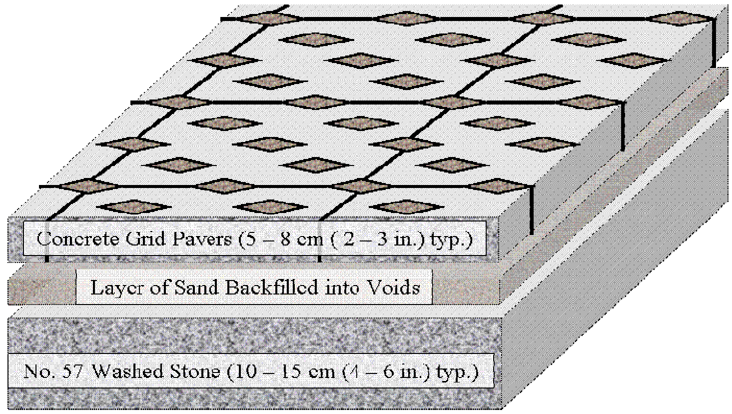
Run #1	Run #2	Run #3	Average	
2.0	1.8	1.2	1.7	cm/hr
0.8	0.7	0.5	0.7	in./hr

Post-Maintenance Condition Test:

Run #1	Run #2	Run #3	Average	
3.0	4.1	12.5	6.5	cm/hr
1.2	1.6	4.9	2.6	in./hr

Maintenance Performed: Removed top 1.3 cm (0.5 in.) of material from drainage pores.

Site:	Cary Public Works
Type of Surface:	Concrete Grid Pavers
Use:	Parking and main traffic areas



Pervious Area:	3000	sq. m
	32000	sq. ft.
Approximate Drainage Area:	4000	sq. m
	43000	sq. ft.
Construction Date:	1995	

Address:	400 James Jackson Rd., Cary, NC
Test Date:	August 7, 2003
Visual Assessment:	Small plants, sand, and trash in void spaces, slightly sloped.
Maintenance Practice:	None, only replaced broken blocks

Existing Conditions Test:

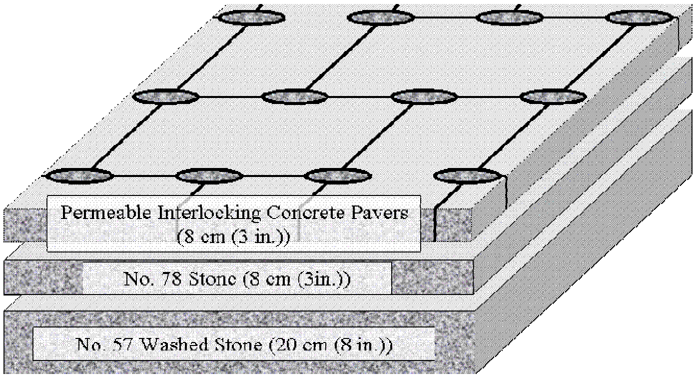
Run #1	Run #2	Run #3	Average	
0.8	0.7	1.5	1.0	cm/hr
0.3	0.3	0.6	0.4	in./hr

Post-Maintenance Condition Test:

Run #1	Run #2	Run #3	Average	
1.5	1.9	1.4	1.6	cm/hr
0.6	0.7	0.6	0.6	in./hr

Maintenance Performed: Removed top 1.3 cm (0.5 in.) of material from drainage pores.

Site:	Goldsbor Monitoring Site (Mickey's Pastry Shop)
Type of Surface:	Permeable Interlocking Concrete Pavers
Use:	Public parking for bakery



Pervious Area:	360 sq. m 3900 sq. ft.	30 stalls
Approximate Drainage Area:	540 sq. m. 5800 sq. ft.	
Construction Date:	2001	
Address:	2704 Graves Dr., Goldsboro, NC	
Test Date:	July 22, 2003	
Visual Assessment:	Looks very clean with minimal debri in voids.	
Maintenance Practice:	None	

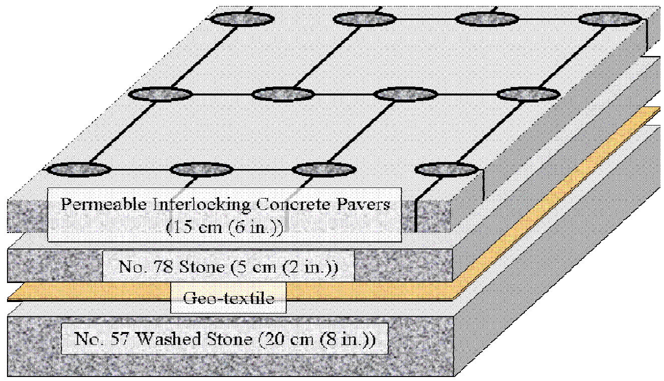
Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
3960	3913	4070	3981	cm/hr
1559.1	1540.6	1602.2	1567.3	in./hr

Site: CVS Pharmacy

Type of Surface: Permeable Interlocking Concrete Pavers

Use: Fringe parking



Pervious Area: 540 sq. m
5800 sq. ft.

Approximate Drainage Area: 540 sq. m
5800 sq. ft.

Construction Date: 2003

Address: 12001 Ocean Highway, Ocean City, MD

Test Date: July 29, 2003

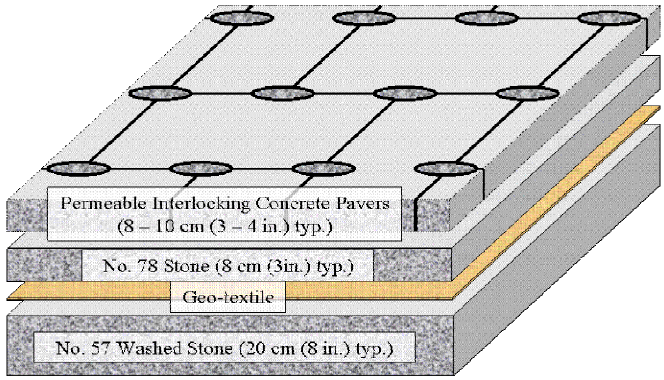
Visual Assessment: Looks clean working well.

Maintenance Practice: New

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
3600	4547	3436	3861	cm/hr
1417.3	1790.3	1352.9	1520.2	in./hr

Site:	Wal-Mart
Type of Surface:	Permeable Interlocking Concrete Pavers
Use:	Overflow parking

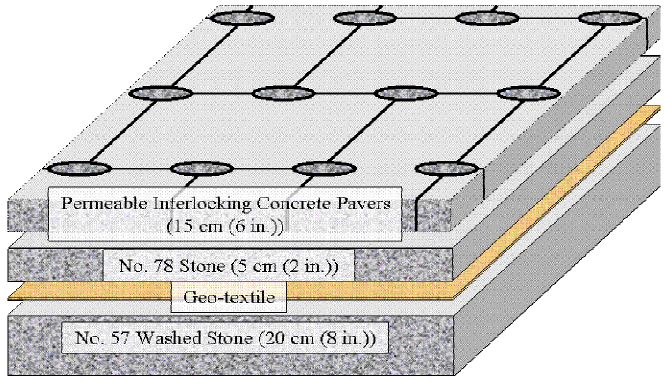


Pervious Area:	4000 sq. m 43000 sq. ft.
Approximate Drainage Area:	4000 sq. m 43000 sq. ft.
Construction Date:	2001
Address:	4493 Highway One, Rehoboth Beach, DE
Test Date:	July 29, 2003
Visual Assessment:	Lot is rarely used. May have underdrain.
Maintenance Practice:	New

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
2817	3024	3133	2992	cm/hr
1109.2	1190.6	1233.6	1177.8	in./hr

Site:	Dough Rollers
Type of Surface:	Permeable Interlocking Concrete Pavers
Use:	Resteraunt parking



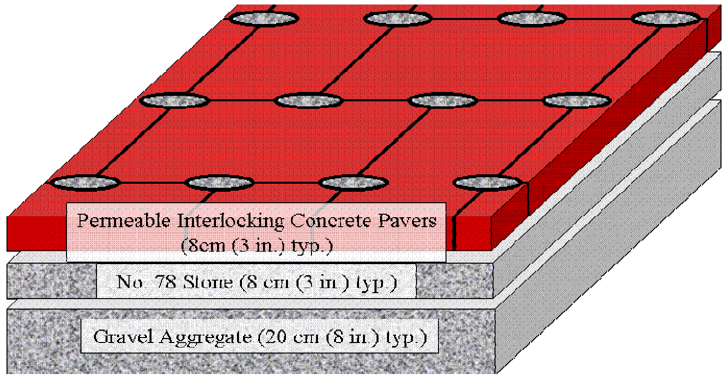
Pervious Area:	560	sq. m
	6000	sq. ft.
Approximate Drainage Area:	790	sq. m
	8500	sq. ft.
Construction Date:	2003	

Address:	41st St. & Coastal Highway, Ocean City, MD
Test Date:	July 29, 2003
Visual Assessment:	Clean and open pores.
Maintenance Practice:	New Site

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
2829	2068	2600	2499	cm/hr
1113.6	814.2	1023.6	983.8	in./hr

Site:	Swansboro Community Parking Lot
Type of Surface:	Permeable Interlocking Concrete Pavers
Use:	Public Parking



Pervious Area:	740 sq. m 8000 sq. ft.
Approximate Drainage Area:	740 sq. m 8000 sq. ft.
Construction Date:	2003
Address:	406 W. Corbett Ave. (NC 58), Swansboro, NC
Test Date:	June 23, 2004
Visual Assessment:	Free of fines or any other materials.
Maintenance Practice:	New

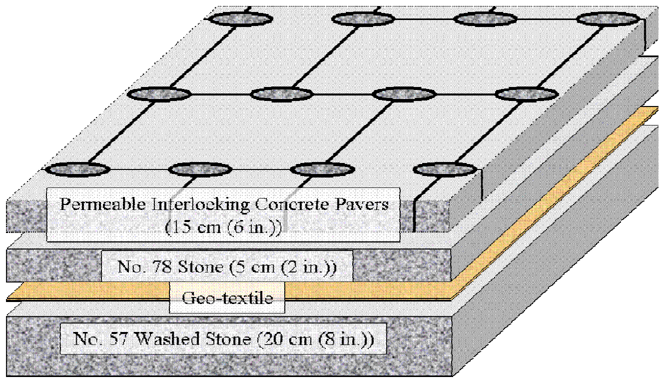
Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
2290	2130	2301	2240	cm/hr
901.8	838.4	905.9	882.0	in./hr

Site: Captiva Bay Condos

Type of Surface: Permeable Interlocking Concrete Pavers

Use: Residential Parking



Pervious Area: 310 sq. m
3300 sq. ft.

Approximate Drainage Area: 310 sq. m
3300 sq. ft.

Construction Date: 2003

Address: 85th St., Ocean City, MD

Test Date: July 29, 2003

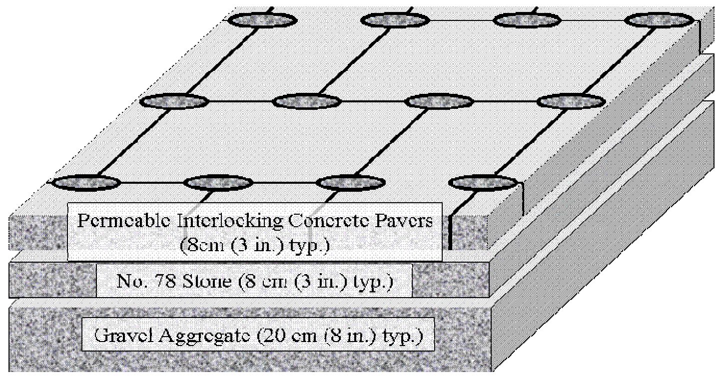
Visual Assessment: Condos under construction w/ hay bales around test site.

Maintenance Practice: New

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
2129	3423	851	2134	cm/hr
838.0	1347.7	335.1	840.3	in./hr

Site:	Naval Medical Center Walkway
Type of Surface:	Permeable Interlocking Concrete Pavers
Use:	Walking/Emergency Drive Path

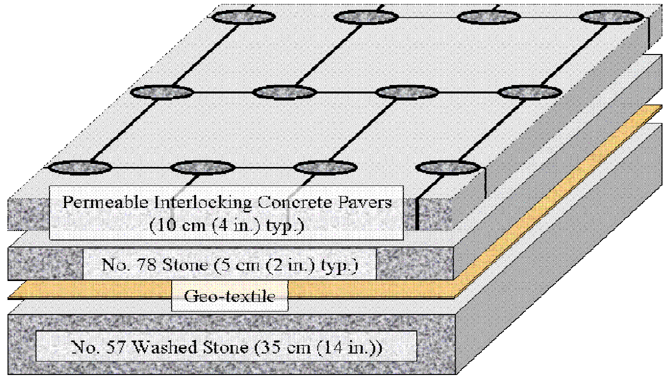


Pervious Area:	2000 sq. m 22000 sq. ft.
Approximate Drainage Area:	3000 sq. m 32000 sq. ft.
Construction Date:	2002
Address:	620 John Paul Jones Circle, Portsmouth, VA
Test Date:	June 15, 2004
Visual Assessment:	Some sand accumulation, but mostly free of fines
Maintenance Practice:	No maintenance.

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
1137	1734	1315	1395	cm/hr
447.5	682.5	517.9	549.3	in./hr

Site:	Baywoods of Annapolis Assisted Living Center
Type of Surface:	Permeable Interlocking Concrete Pavers
Use:	Parking and drive areas of assisted living center

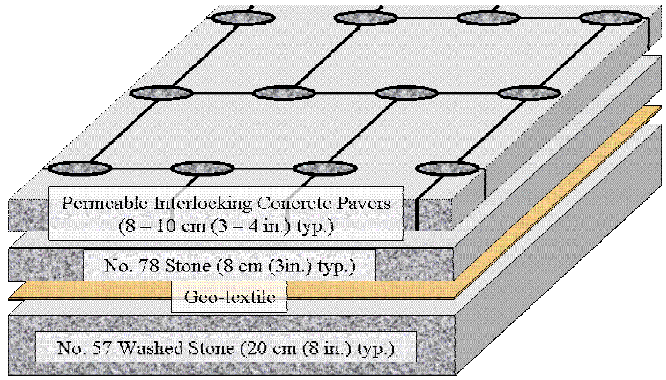


Pervious Area:	1300 sq. m 14000 sq. ft.
Approximate Drainage Area:	1300 sq. m 14000 sq. ft.
Construction Date:	2002
Address:	Bembe Beach Rd, Annapolis, MD
Test Date:	July 28, 2003
Visual Assessment:	Looks as though voids are permeable.
Maintenance Practice:	None

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
1072	672	1284	1009	cm/hr
422.1	264.4	505.7	397.4	in./hr

Site:	Harve de Grace
Type of Surface:	Permeable Interlocking Concrete Pavers
Use:	City street parking

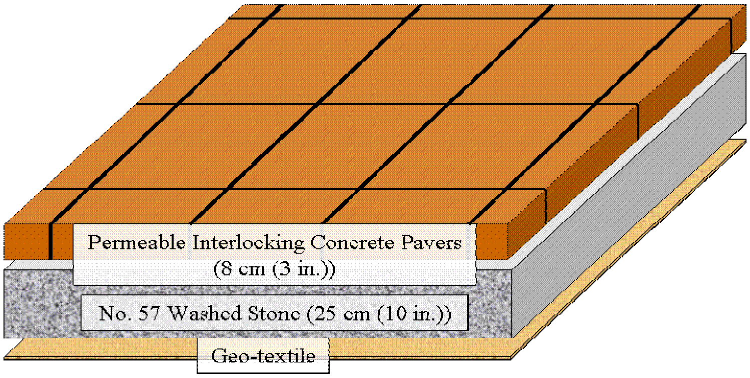


Pervious Area:	1100 sq. m 12000 sq. ft.
Approximate Drainage Area:	1100 sq. m 12000 sq. ft.
Construction Date:	1997
Address:	Concord Ave., Harve de' Grace, MD
Test Date:	July 28, 2003
Visual Assessment:	Needs maintenance and voids sealed by dirt, oil and debri.
Maintenance Practice:	Street sweeper every 1-2 weeks

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
76.9	85.5	138.0	100.1	cm/hr
30.3	33.7	54.3	39.4	in./hr

Site:	Penny Road PICP
Type of Surface:	Permeable Interlocking Concrete Pavers
Use:	Private Driveway

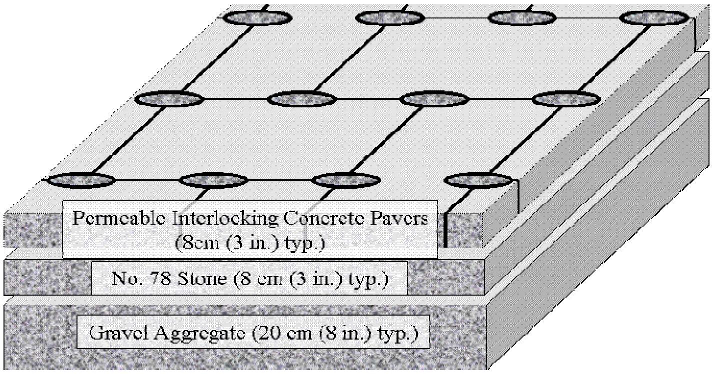


Pervious Area:	430 sq. m 4600 sq. ft.
Approximate Drainage Area:	430 sq. m 4600 sq. ft.
Construction Date:	2003
Address:	Birk Bluffs Ct., Cary, NC
Test Date:	October 15, 2003
Visual Assessment:	Clay and sediment accumulation in void spaces.
Maintenance Practice:	New

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
37	12	643	231	cm/hr
14.6	4.7	253.0	90.8	in./hr

Site:	Naval Medical Center Walkway
Type of Surface:	Permeable Interlocking Concrete Pavers
Use:	Walking/Emergency Drive Path

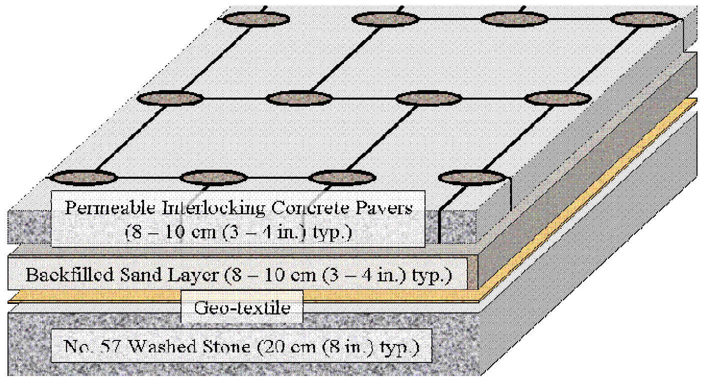


Pervious Area:	460 sq. m 5000 sq. ft.	38 stalls
Approximate Drainage Area:	460 sq. m 5000 sq. ft.	
Construction Date:	2002	
Address:	620 John Paul Jones Circle, Portsmouth, VA	
Test Date:	June 15, 2004	
Visual Assessment:	Sand and other fines have filled all voids.	
Maintenance Practice:	No maintenance.	

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
82.6	29.5	50.5	54.2	cm/hr
32.5	11.6	19.9	21.3	in./hr

Site:	River Bend
Type of Surface:	Permeable Interlocking Concrete Pavers
Use:	Parking for public works buildings



Pervious Area:	100 sq. m 1100 sq. ft.	8 stalls
Approximate Drainage Area:	100 sq. m 1100 sq. ft.	
Construction Date:	2000	
Address:	45 Shoreline Dr., River Bend, NC	
Test Date:	July 22, 2003	
Visual Assessment:	Compacted fines from gravel drive in voids.	
Maintenance Practice:	None	

Existing Conditions Test:

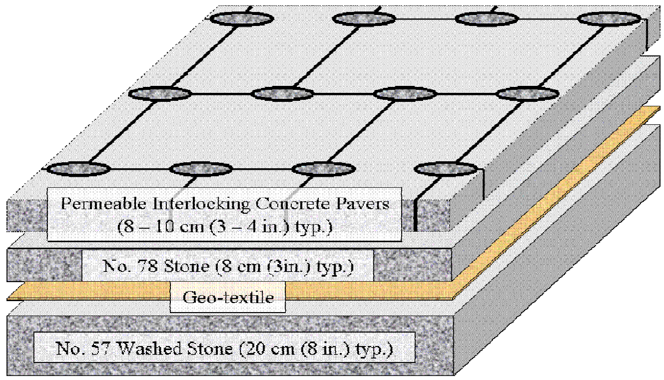
Run #1	Run #2	Run #3	Average	
3.5	3.3	17.2	8.0	cm/hr
1.4	1.3	6.8	3.1	in./hr

Post-Maintenance Condition Test:

Run #1	Run #2	Run #3	Average	
20.8	39.0	8.0	22.6	cm/hr
8.2	15.4	3.1	8.9	in./hr

Maintenance Performed: Removed top 1.9 cm (0.75 in.) of material from drainage pores.

Site:	Boat Ramp
Type of Surface:	Permeable Interlocking Concrete Pavers
Use:	Parking for boat launching.

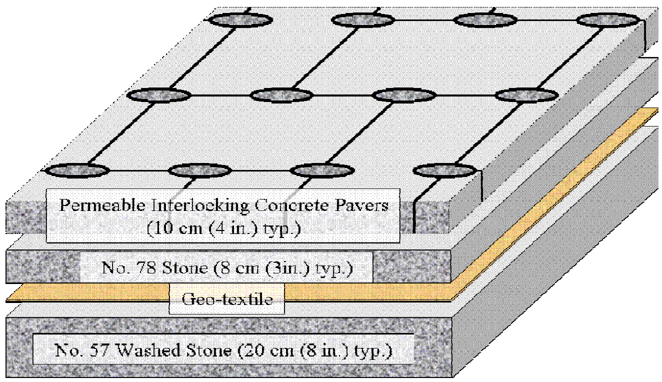


Pervious Area:	2000 sq. m 22000 sq. ft.
Approximate Drainage Area:	2000 sq. m 22000 sq. ft.
Construction Date:	2001
Address:	Route 231, Prince Frederick, MD
Test Date:	July 28, 2003
Visual Assessment:	Heavily used with lots of sand in void spaces.
Maintenance Practice:	none

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
4.3	2.0	2.5	2.9	cm/hr
1.7	0.8	1.0	1.2	in./hr

Site:	Somerset Drive, Ocean City, MD
Type of Surface:	Permeable Interlocking Concrete Pavers
Use:	Walkway to beach.

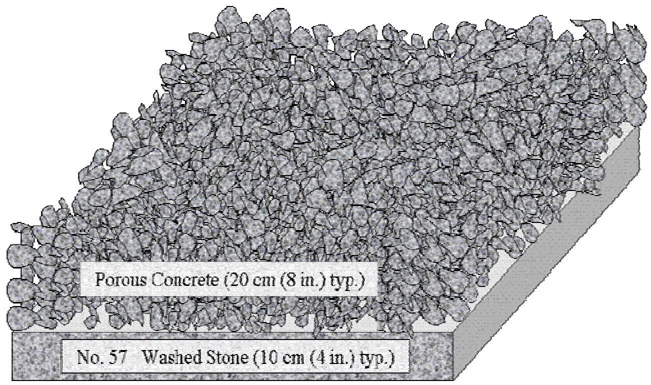


Pervious Area:	640 sq. m 6900 sq. ft.
Approximate Drainage Area:	640 sq. m 6900 sq. ft.
Construction Date:	Winter 2001
Address:	Somerset St., Ocean City, MD
Test Date:	July 29, 2003
Visual Assessment:	Lots of sand in voids.
Maintenance Practice:	Street Sweeper twice per week from June-Sept.

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
0.0	1.8	3.0	1.6	cm/hr
0.0	0.7	1.2	0.6	in./hr

Site:	Center for the Environment, Catawba College
Type of Surface:	Porous Concrete
Use:	Walkway



Pervious Area:	80 sq. m 860 sq. ft.
Approximate Drainage Area:	80 sq. m 860 sq. ft.
Construction Date:	2003
Address:	2300 W Innes St, Salisbury, NC
Test Date:	Juen 21, 2004
Visual Assessment:	substantial accumulation of pine straw.
Maintenance Practice:	Pressure washed once.

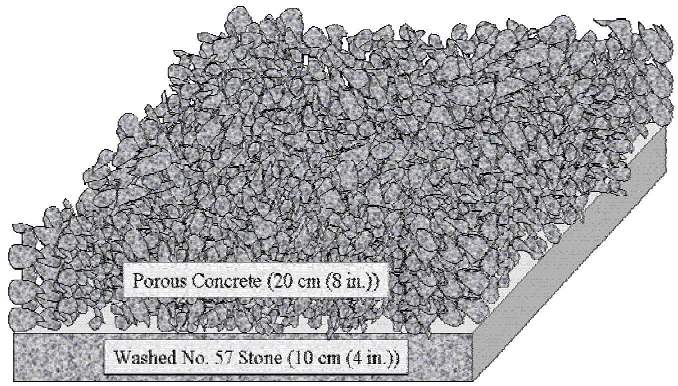
Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
1535	1207	3900	2214	cm/hr
604	475	1535	872	in./hr

Site: Loflin Concrete

Type of Surface: Porous Concrete

Use: Test plot; Entrance to maintenance garage



Pervious Area: 50 sq. m
540 sq. ft.

Approximate Drainage Area: 50 sq. m (plus unknown roof area)
540 sq. ft.

Construction Date: 2002

Address: 411 Valley Forge Rd., Hillsborough, NC

Test Date: June 22, 2004

Visual Assessment: Fine accumulation in places; oil spots; erroded

Maintenance Practice: None

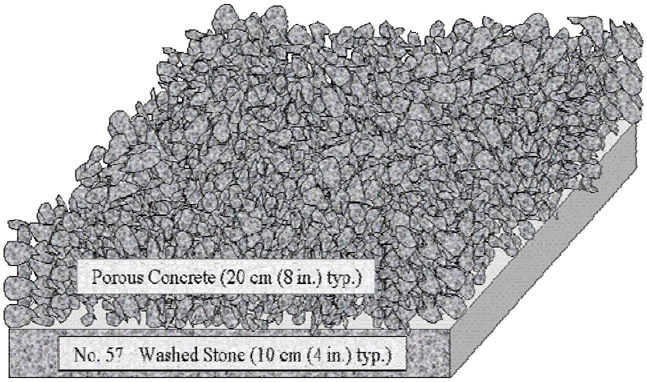
Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
304	6574	12343	6407	cm/hr
120	2588	4859	2522	in./hr

Site: Bailey's Landing II

Type of Surface: Porous Concrete

Use: Private Driveway



Pervious Area:	120	sq. m
	1300	sq. ft.
Approximate Drainage Area:	120	sq. m
	1300	sq. ft.

Construction Date: 2003

Address: 2425 Bailey's Landing Dr., Raleigh, NC

Test Date: June 4, 2004

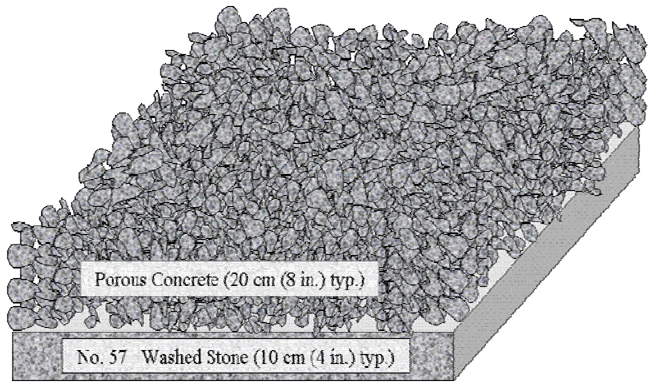
Visual Assessment: Some discoloration, but no major accumulation of fines.

Maintenance Practice: Pressure washed once.

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
1465	11250	5589	6101	cm/hr
577	4429	2200	2402	in./hr

Site:	Penny Road
Type of Surface:	Porous Concrete
Use:	Private Driveway



Pervious Area:	190 sq. m 2000 sq. ft.
Approximate Drainage Area:	190 sq. m 2000 sq. ft.
Construction Date:	2002
Address:	Birk Bluffs Ct., Cary, NC
Test Date:	May 4, 2004
Visual Assessment:	Some discoloration from fines.
Maintenance Practice:	New

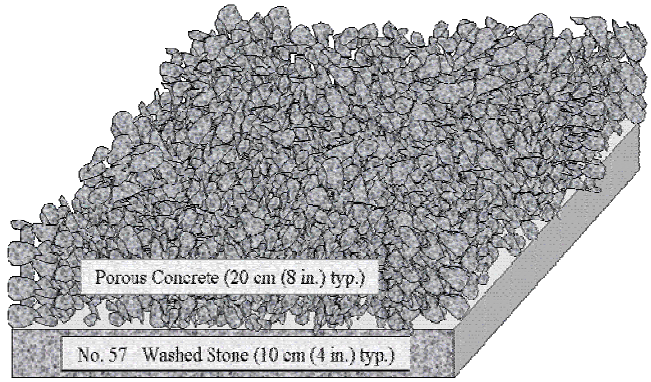
Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
6429	1330	3436	3732	cm/hr
2531	524	1353	1469	in./hr

Site: Friday Center Park and Ride Parking Lot

Type of Surface: Porous Concrete

Use: Parking Lot



Pervious Area: 3600 sq. m 200 stalls
39000 sq. ft.

Approximate Drainage Area: 3600 sq. m
39000 sq. ft.

Construction Date: 2002

Address: Hwy. 54, Chapel Hill, NC

Test Date: August 18,2003

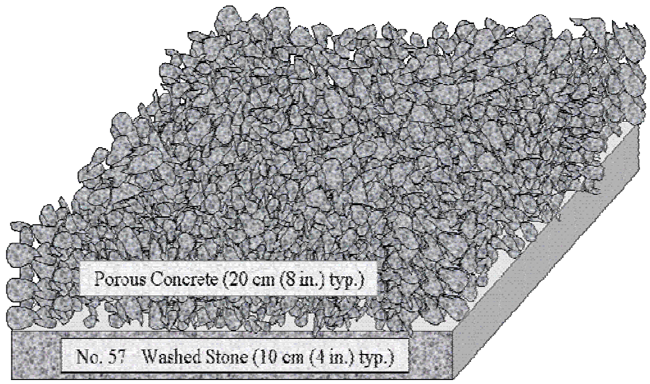
Visual Assessment: Clean and well maintained. Minor discoloration from fines.

Maintenance Practice: New

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
1535	1207	3900	2214	cm/hr
604.4	475.3	1535.4	871.7	in./hr

Site:	Ready Mix Concrete Lab
Type of Surface:	Porous Concrete
Use:	Test plot/Loading area



Pervious Area:	42 sq. m 450 sq. ft.
Approximate Drainage Area:	42 sq. m 450 sq. ft.
Construction Date:	2002
Address:	6112 Westgate Rd., Raleigh, NC
Test Date:	June 22, 2004
Visual Assessment:	Large aggregate accumulation.
Maintenance Practice:	None

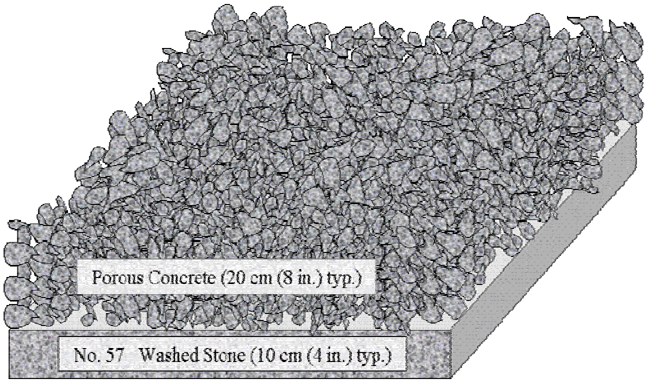
Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
14	328	3531	1291	cm/hr
6	129	1390	508	in./hr

Site: Bailey's Landing II

Type of Surface: Porous Concrete

Use: Private Driveway



Pervious Area: 190 sq. m
2000 sq. ft.

Approximate Drainage Area: 200 sq. m
2200 sq. ft.

Construction Date: 2003

Address: 2301 Bailey's Landing Dr., Raleigh, NC

Test Date: June 22, 2004

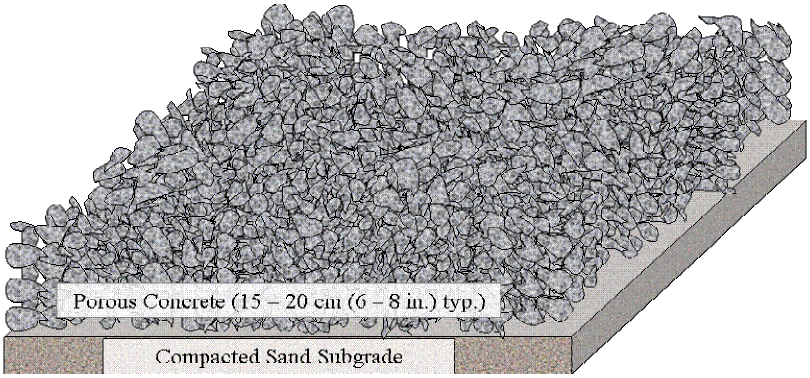
Visual Assessment: Sediment deposition from surrounding watershed.

Maintenance Practice: None

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
0	22	1880	634	cm/hr
0	9	740	250	in./hr

Site:	McCrary Park
Type of Surface:	Porous Concrete
Use:	Parking for city park

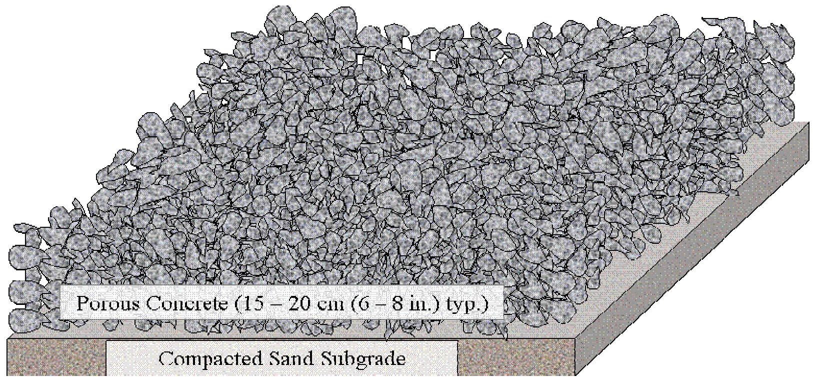


Pervious Area:	320 sq. m 3400 sq. ft.	18 stalls
Approximate Drainage Area:	320 sq. m 3400 sq. ft.	
Construction Date:	2002	
Address:	Randall Parkway, Wilmington, NC	
Test Date:	July 24, 2003	
Visual Assessment:	Some fines are present in voids, but looks clean.	
Maintenance Practice:	Vaccum swept once per year.	

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
10.6	12.6	44.5	22.6	cm/hr
4.2	5.0	17.5	8.9	in./hr

Site:	Atlantic Beach Drive
Type of Surface:	Porous Concrete
Use:	Town street (installed for flood reduction and infiltration)



Pervious Area:	84	sq. m
	900	sq. ft.
Approximate Drainage Area:	17000	sq. m
	180000	sq. ft.
Construction Date:	2000	

Address:	114 Boardwalk, Atlantic Beach, NC
Test Date:	July 24, 2003
Visual Assessment:	Substantial sand and silt deposits.
Maintenance Practice:	None

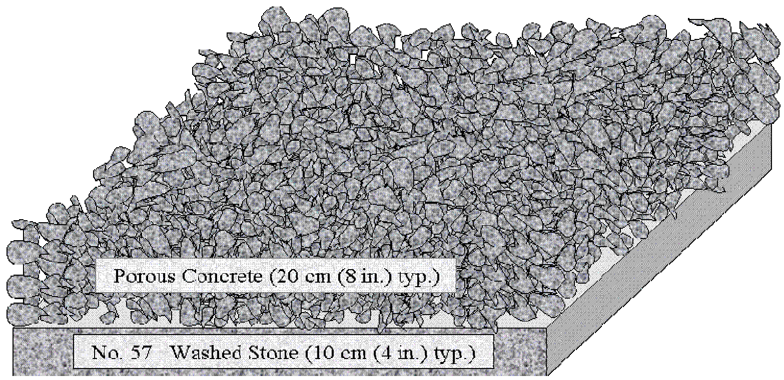
Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
14.1	17.7	9.2	13.7	cm/hr
5.6	6.9	3.6	5.4	in./hr

Site: Bryarton I

Type of Surface: Porous Concrete

Use: Private Driveway



Pervious Area: 80 sq. m
860 sq. ft.

Approximate Drainage Area: 80 sq. m
860 sq. ft.

Construction Date: 2000

Address: Southern Cross Ave., Raleigh, NC

Test Date: June 25, 2004

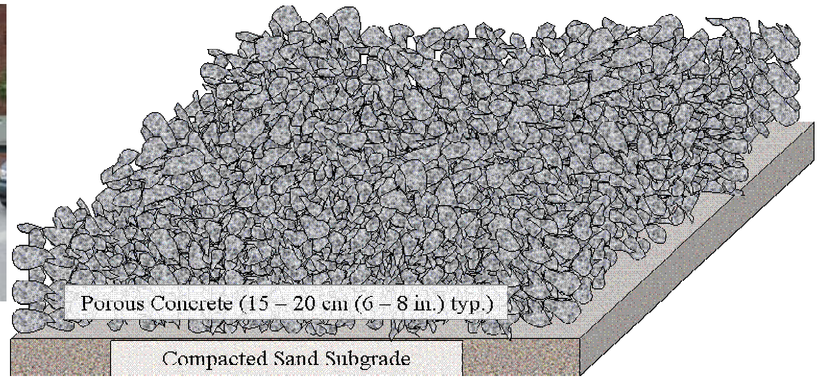
Visual Assessment: Fine sediment accumulation and smearing during construction.

Maintenance Practice: None

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
38.4	0.4	0.0	12.9	cm/hr
15.1	0.2	0.0	5.1	in./hr

Site:	Wrightsville Beach Catholic Church
Type of Surface:	Porous Concrete
Use:	Parking Lot

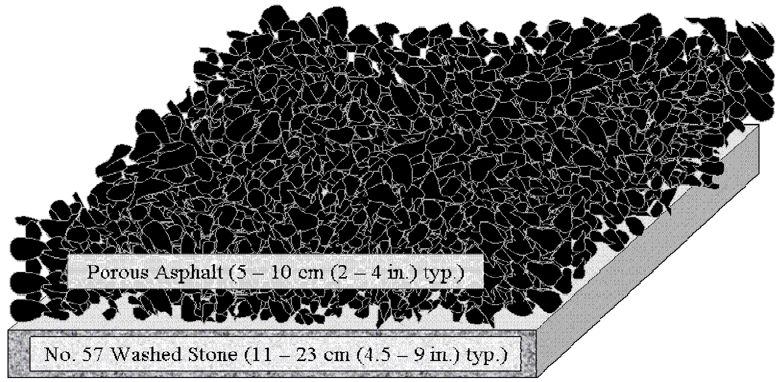


Pervious Area:	300 sq. m 3200 sq. ft.	16 stalls
Approximate Drainage Area:	300 sq. m 3200 sq. ft.	
Construction Date:	2003	
Address:	209 South Lumina Avenue, Wrightsville Beach, NC	
Test Date:	June 16, 2004	
Visual Assessment:	Heavy silt and sand deposits in surface voids.	
Maintenance Practice:	None	

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
5.5	19.4	9.2	11.4	cm/hr
2.2	7.6	3.6	4.5	in./hr

Site:	The Friday Center Park and Ride (porous asphalt)
Type of Surface:	Porous Asphalt
Use:	Park and Ride

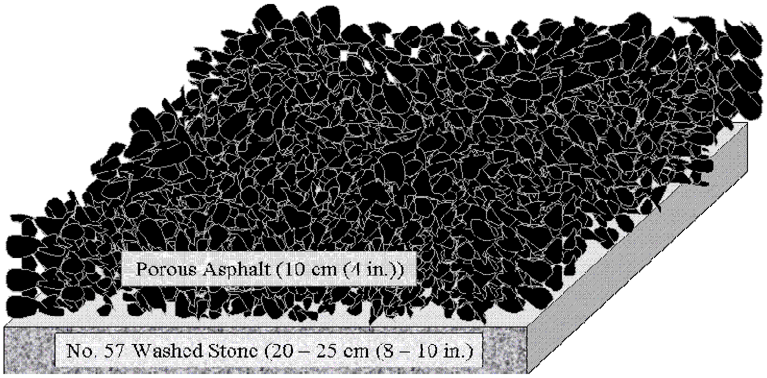


Pervious Area:	11000 sq. m 120000 sq. ft.	600 stalls
Approximate Drainage Area:	11000 sq. m 120000 sq. ft.	
Construction Date:	2002	
Address:	Hwy. 54, Chapel Hill, NC	
Test Date:	August 18, 2003	
Visual Assessment:	Very clean and permeability has been maintained.	
Maintenance Practice:	New	

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
7615	3528	7855	6333	cm/hr
2998	1389	3092	2493	in./hr

Site:	Atlantic Beach PA I
Type of Surface:	Porous Asphalt
Use:	Town street (installed for flood reduction and infiltration)

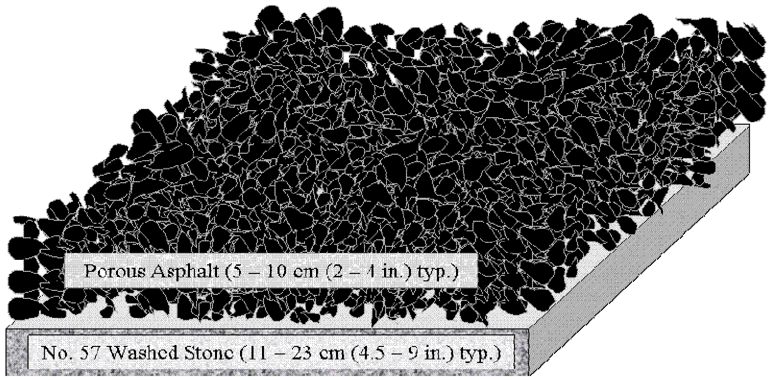


Pervious Area:	110	sq. m
	1200	sq. ft.
Approximate Drainage Area:	1200	sq. m
	13000	sq. ft.
Construction Date:	2002	
Address:	200 Block of Greenville Ave., Atlantic Beach, NC	
Test Date:	June 17, 2004	
Visual Assessment:	Looks like impermeable asphalt; sand and silt accumulation.	
Maintenance Practice:	None	

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
9.4	12.0	18.6	13.3	cm/hr
3.7	4.7	7.3	5.2	in./hr

Site:	Fayetteville Technical Community College I
Type of Surface:	Porous Asphalt
Use:	Parking for student center



Pervious Area:	5500 sq. m 59000 sq. ft.	308 stalls
Approximate Drainage Area:	5500 sq. m 59000 sq. ft.	
Construction Date:	1986	
Address:	2201 Hull Rd., Fayetteville, NC	
Test Date:	August 14, 2003	
Visual Assessment:	Seems to be sealed and impermeable.	
Maintenance Practice:	Vaccum swept 4x per year.	

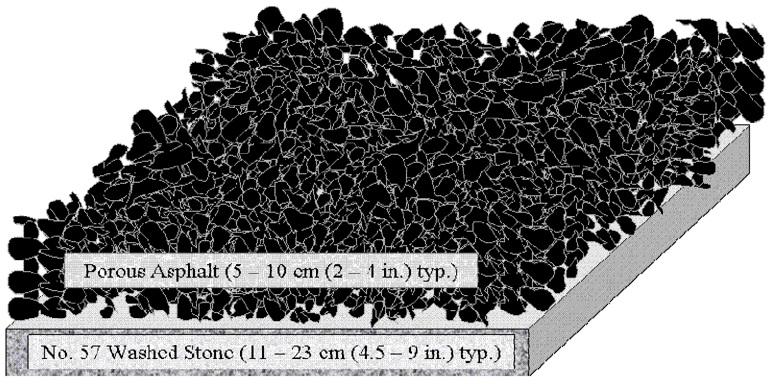
Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
5.8	1.8	8.5	5.4	cm/hr
2.3	0.7	3.3	2.1	in./hr

Site: Fayetteville Technical Community College II

Type of Surface: Porous Asphalt

Use: Student parking



Pervious Area: 2000 sq. m
22000 sq. ft.

Approximate Drainage Area: 2000 sq. m
22000 sq. ft.

Construction Date: 1996

Address: 2201 Hull Rd., Fayetteville, NC

Test Date: August 14, 2003

Visual Assessment: Looks fairly well maintained. Minimal sediment on surface.

Maintenance Practice: Contractor vaccum sweeps lot quarterly.

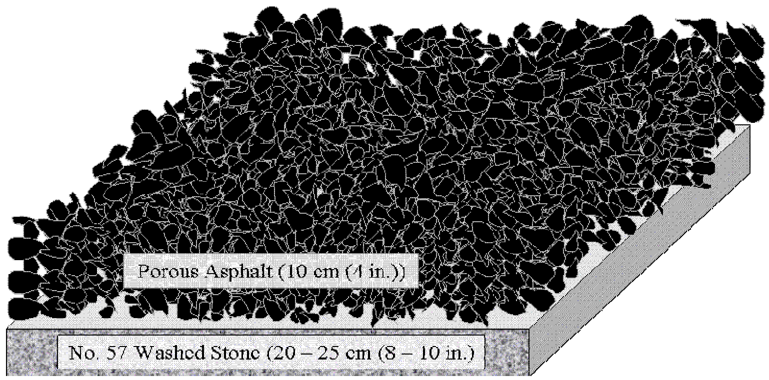
Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
7.8	3.8	5.6	5.7	cm/hr
3.1	1.5	2.2	2.3	in./hr

Site: Atlantic Beach PA II

Type of Surface: Porous Asphalt

Use: Town street (installed for flood reduction and infiltration)



Pervious Area: 130 sq. m
1400 sq. ft.

Approximate Drainage Area: 1300 sq. m
14000 sq. ft.

Construction Date: 2002

Address: 126 Mobile Dr., Atlantic Beach, NC

Test Date: June 23, 2004

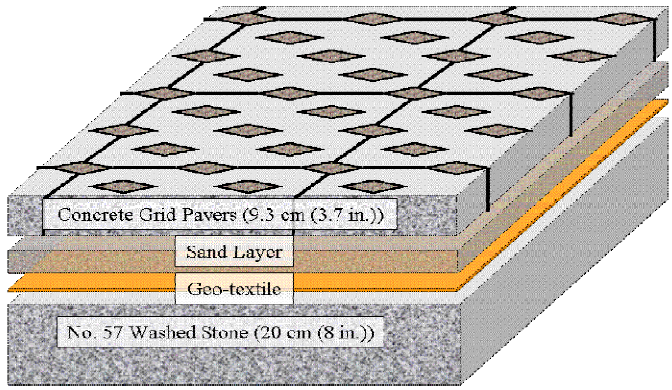
Visual Assessment: Fines evident and present on the surface.

Maintenance Practice: None

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
3.5	5.4	2.8	3.9	cm/hr
1.4	2.1	1.1	1.5	in./hr

Site:	Kinston CGP
Type of Surface:	Concrete Grid Pavers
Use:	Police vehicle parking

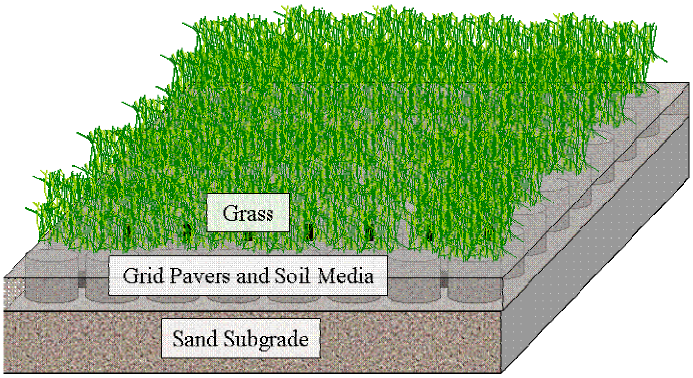


Pervious Area:	590 sq. m 6400 sq. ft.	20 stalls
Approximate Drainage Area:	590 sq. m 6400 sq. ft.	
Construction Date:	1999	
Address:	207 E. King St., Kinston, NC	
Test Date:	July 21, 2003	
Visual Assessment:	Very sandy with less than 10% grass in void spaces.	
Maintenance Practice:	One sweep per year with a street sweeper.	

Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
83.5	31.9	57.3	57.6	cm/hr
32.9	12.6	22.6	22.7	in./hr

Site:	Kinston GP
Type of Surface:	Grass Pavers
Use:	Police Vehicle Parking



Pervious Area:	180 sq. m 1900 sq. ft.	6 stalls
Approximate Drainage Area:	180 sq. m 1900 sq. ft.	
Construction Date:	1999	
Address:	207 E. King St., Kinston, NC	
Test Date:	July 21, 2003	
Visual Assessment:	Some areas with no grass, but grass seems to do well.	
Maintenance Practice:	Mowing when needed.	

Bare Area:

Run #1	Run #2	Run #3	Average	
7.8	5.3	14.3	9.1	cm/hr
3.1	2.1	5.6	3.6	in./hr

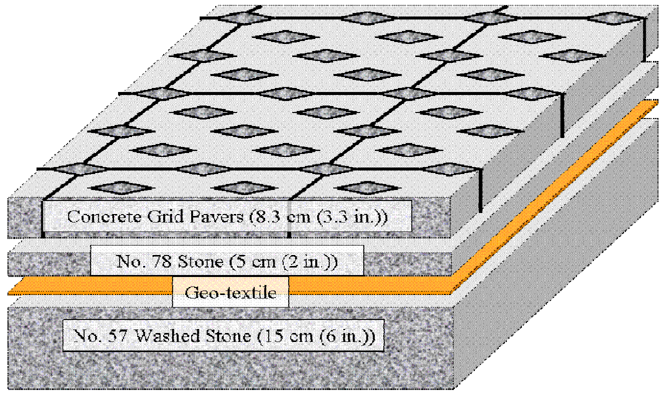
Grassy Area:

Run #1	Run #2	Run #3	Average	
27.6	33.9	32.1	31.2	cm/hr
10.9	13.3	12.6	12.3	in./hr

Site: Wynn Plaza

Type of Surface: Concrete Grid Pavers

Use: Park and beach parking



Pervious Area: 100 sq. m / 1100 sq. ft. — 8 stalls

Approximate Drainage Area: 100 sq. m / 1100 sq. ft.

Construction Date: 2001

Address: 101 S. Lumina Ave., Wrightsville Beach, NC

Test Date: July 23, 2003

Visual Assessment: Some sand filled in voids, and negligible grass growth.

Maintenance Practice: Once every two weeks with a street sweeper.
(Tracy Dail, Wrightsville Beach)

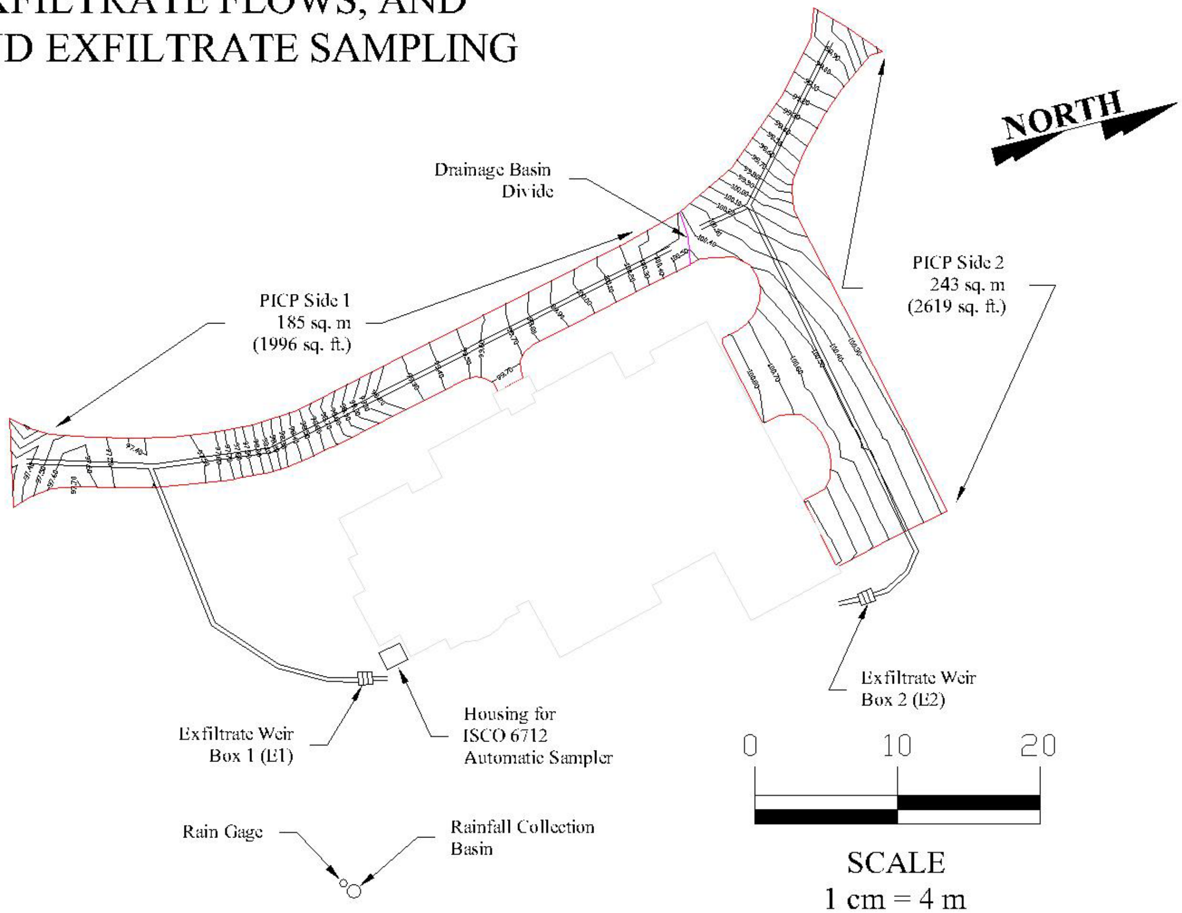
Existing Conditions Test:

Run #1	Run #2	Run #3	Average	
13.1	8.3	12.9	11.4	cm/hr
5.1	3.3	5.1	4.5	in./hr

APPENDIX B

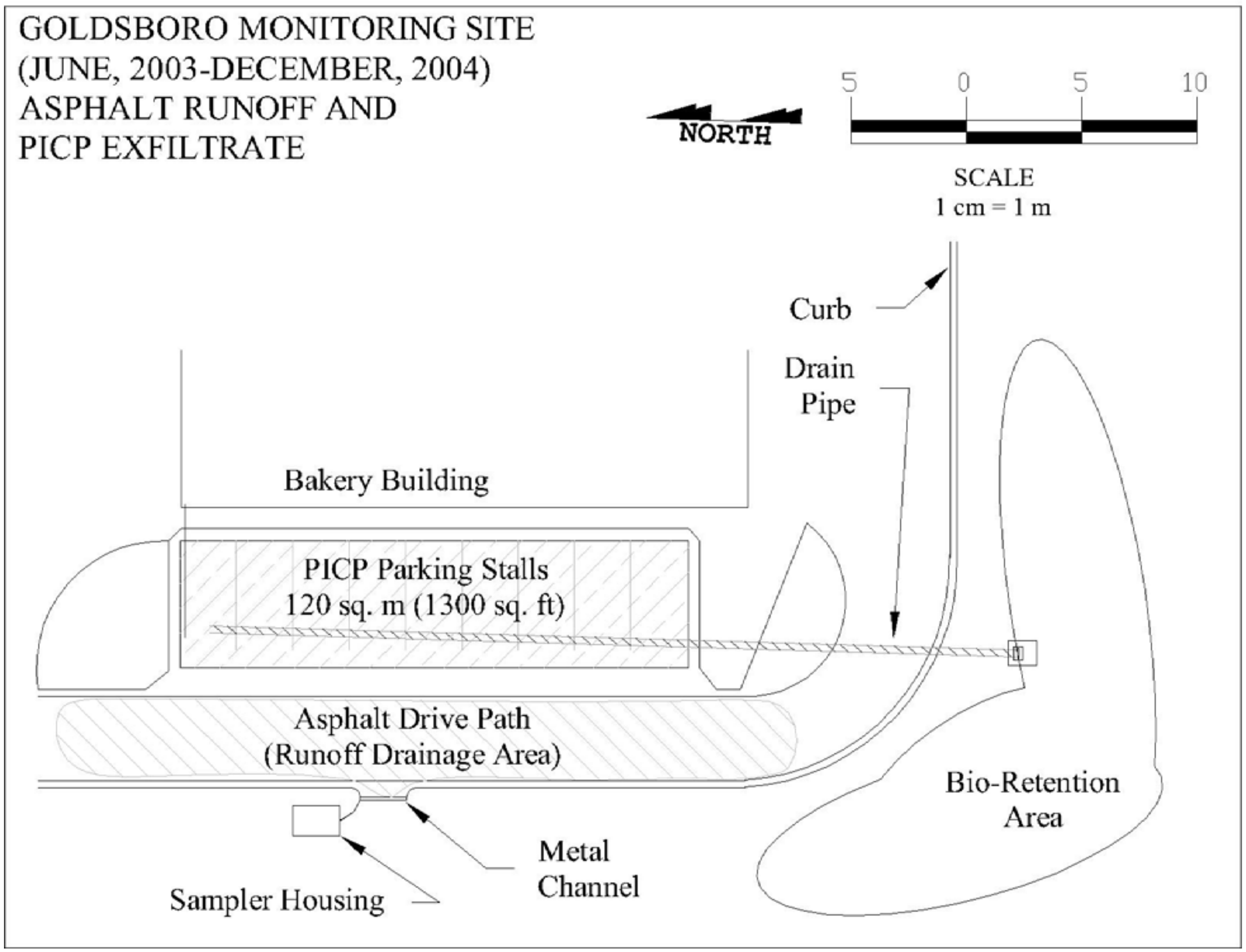
CARY MONITORING SITE DIAGRAM

CARY MONITORING SITE
(JANUARY-NOVEMBER, 2004)
RAINFALL, EXFILTRATE FLOWS, AND
RAINFALL AND EXFILTRATE SAMPLING



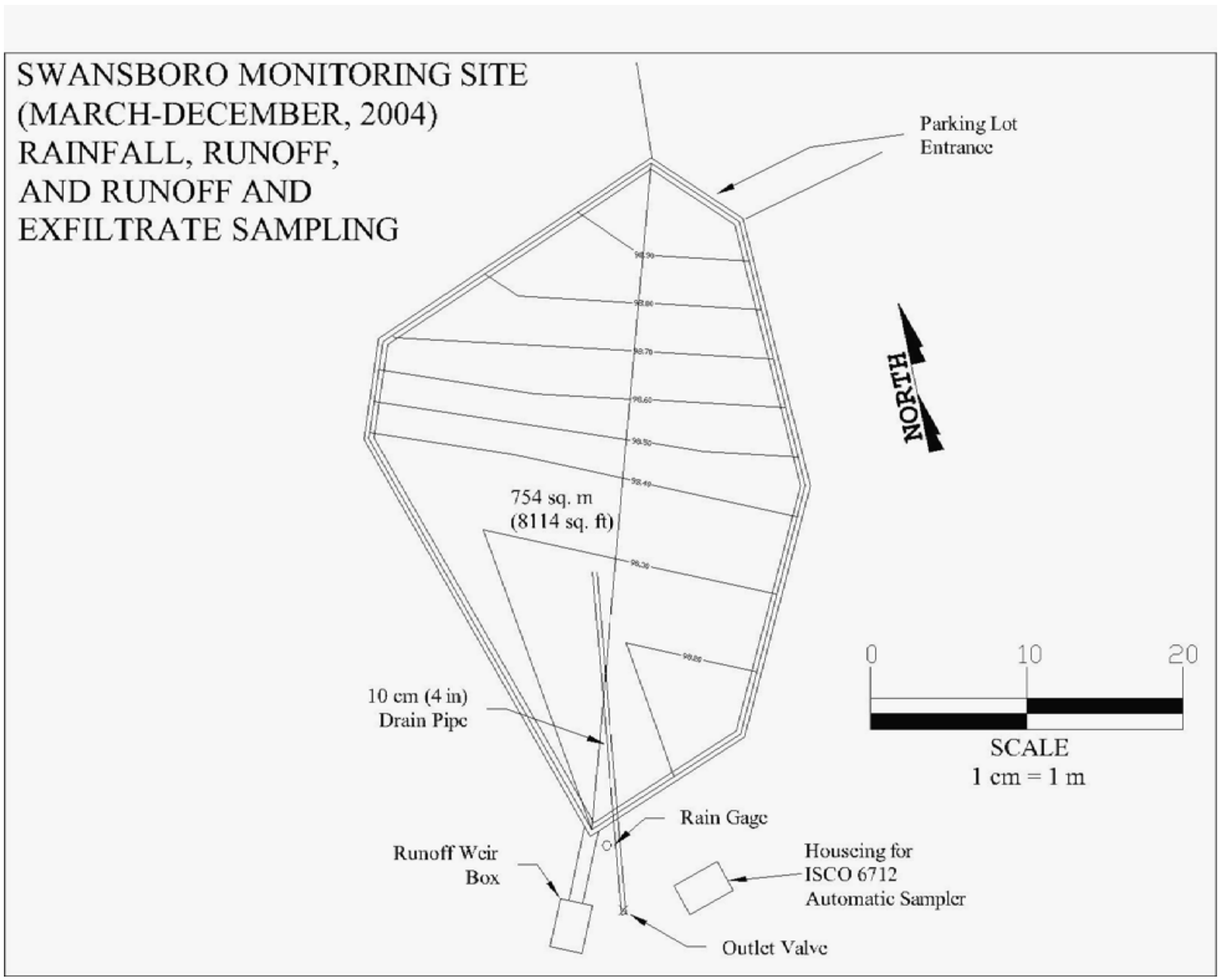
APPENDIX C

GOLDSBORO MONITORING SITE DIAGRAM



APPENDIX D

SWANSBORO MONITORING SITE DIAGRAM



APPENDIX E

FLOW DATA COLLECTED FROM CARY MONITORING SITE

ABSTRACT

A permeable pavement site in Cary, North Carolina, was instrumented with flow and rainfall monitoring equipment to determine the hydrologic performance of a lined Permeable Interlocking Concrete Pavements (PICP) cell in a clay loam soil. Data from the Cary flow monitoring site was used to determine that partial infiltration occurred. The site showed the potential to infiltrate runoff; 57% of rainfall on average infiltrated. On average, between 30 and 56% of peak rainfall rates infiltrated the site. It was also determined that on average, the delay to peak between rainfall and exfiltrate flows was 1.14 hrs, for storms over 1.3 cm (0.5 in). However, delays to peak may be dependent on the rainfall intensity, with a potential maximum delay of 1.4 hrs at minimal intensities for this site.

INTRODUCTION

The Cary permeable interlocking concrete pavement (PICP) sites, was equipped with monitoring equipment to monitor exfiltrate outflow and rainfall rates. Any water that passed through the PICP pavers, until it exited the storage basin is referred to as infiltrate flow, while water that left the storage basin through a drainpipe is referred to as exfiltrate flow. The site was instrumented to determine the hydrologic performance of a lined PICP cell in a clay loam soil. However, data quality was marginal due to numerous instrument failures and misinformation. Although conclusive results were not determined from this site, collected data with analysis are presented here.

MATERIALS AND METHODS

The Cary site was constructed on clay loam soil in the fall of 2003 with a surface area of 480 m^2 (4200 ft^2) (Appendix B). The SF Rima™ [2] pavers were 8 cm (3 in) thick and were laid over a compacted layer of at least 25 cm (10 in) of washed No. 57 stone (ASTM D448), with a 5 cm (2 in) layer of No. 78 stone between the two layers (Appendix A 25). Photo-analysis was used to determine that open space of PICP was approximately four percent (4%) of the surface area. The storage basin under the pavers was divided into two separately drained basins of 185 m^2 (1996 ft^2) and 243 m^2 (2619 ft^2). Clay loam soil infiltration rates are typically too low for drainage basins to recurrently drain between. Therefore, one 10 cm (4 in) corrugated plastic pipe drained each of the drainage basins. An ISCO 674® tipping bucket rain gauge measured precipitation. In addition, the storage basin, or pavement support layer, was lined with an impermeable geo-textile to prevent deep seepage and shrink swell associated with clay insitu soils.

The Cary site was equipped with flow monitoring equipment to determine exfiltrate flows. Runoff and infiltrate flow volumes were determined from rainfall, watershed areas, and flow monitoring data. The two exfiltrate drainage pipes each flowed into a weir box with a baffle and a 90^o V-notch weir shown in Appendix I. The weir boxes were 0.56 m (1.8 ft) wide by 0.56 m (1.8 ft) deep by 0.46 m (1.5 ft) tall. Baffles were 0.15 m (0.5 ft) in front of the inflow opening to

[2] The use of trade names is for project information only and does not constitute an endorsement by North Carolina State University.

still the flow and weirs were 0.15 m (0.5 ft) in front of the baffles. For one box, Exfiltrate 2 (E2), a WIKA LH-10 Submersible Liquid Level Transmitter® (SLLT) was placed between the baffle and weir to record the water depth. The transmitter produced a current relative to the water level and was powered by two 6-volt lantern batteries. Current output was logged on an Omega Data Logger® (OM-DLAC) every five minutes. Equation 1 was used to calculate flowrates, Q (l/s), from water level above the weir inverts, H (m), for 90° V-notch weirs.

$$Q = 1380 * H^{2.5} \qquad \text{(Equation 1)}$$

The other weir box, Exfiltrate 1 (E1), was used for both water level recording and sample collection. Initially the same equipment and setup as E2 was used solely for water level recording. In February, an ISCO 6712® with a 730 Flow Bubbler Module® replaced the WIKA Submersible Liquid Level Transmitter® and Omega Data Logger® system. The bubbler tube was placed on the floor of the weir box, approximately 0.08 m (0.25 ft) behind the baffle. The sampler was powered by a Free Energy America 20W Solar Panel® and 12-volt marine battery.

An ISCO 674® tipping bucket rain gage was installed at the Cary site to measure rainfall intensities. By quantifying the volume of water entering the site, and measuring exfiltration rate, the volume of runoff was calculated using Equation 2.

$$\text{Runoff Volume} = \text{Rainfall} * \text{Watershed Area} - \text{Exfiltrate Volume} \qquad \text{(Equation 2)}$$

The Cary rain gauge was connected to three different data loggers during monitoring. Each rain gauge collected 0.025 mm (0.01 in) of rainfall per tip and was backed up with a generic manual rain gauge. The rain gage was separated from the sampler by a grassed lawn. The site was also used as a model home, thus, the owners would not approve burying the connector cable below the manicured lawn. Therefore, the rain gage was connected to a series of independent data loggers. Data loggers were interchanged due to performance issues, until a Hobo Event Data Logger® was installed. The Hobo Event Data Logger reliably and consistently recorded tips from the rain gage.

RESULTS AND ANALYSIS

Exfiltrate and rainfall data were analyzed at the Cary site for data collected over ten months (February to November, 2004). However, numerous technical difficulties arose during data collection, weakening any conclusions drawn from this study.

At the Cary site, several technical difficulties arose. Initially an Omega Data Logger® was installed to collect rainfall data. However, after a few weeks, the system design was determined to be incompatible with the equipment being used for data collection. Global Water's GL 400-1-1®, 2 Channel, data logger was then installed, on February 9, to record precipitation intensities on a five minute interval. It was observed that the data logger ceased recording data an arbitrary length of time after downloading, ranging from three days to two weeks. After three months of repeated attempts to remedy this problem, the Global Water Data Logger was removed from the site (May 15). Next, a rain gauge data logger was developed using a BasicX BX-24® processor chip and programming board (Appendix J), but this device never became operational. Finally, a HOBO Event Data Logger® was purchased and installed at the site (May 27) and performed without incident through the remainder of the study. Table 1 lists when rainfall data were collected at the site. Gaps in data were filled by those measured at the Lake Wheeler Research Farms (LWRF) (~4 km (~2.5 mi) from the site). The LWRF weather station collected hourly rainfall data, in lieu of on-site five minute intervals. Rainfall totals between on-site data and LWRF were typically within 0.25 cm (0.1 in) of each other. Peak rainfall rates and times were not determined at the site with LWRF data. Additionally, leakage remained an unresolved problem until May. Inflow pipes at both E1 and E2 constantly leaked where the inflow pipe joined the weir box. Additional weight of water in the pipe during flow caused flexing; therefore, a tight seal between the weir box and pipe did not form. Support stakes were installed under the inflow pipe for support, which ended the leakages. Leaks were also present within the weir boxes where the weir was welded to the box itself. In May, the boxes were drained and the welds were caulked. The repairs closed noticeable leakage. It is unclear what effect theses leaks had on flow measurement; however the leaks were noted.

Table 1. Summary of rain gauge data logger recordings from the Cary site from February 2004 until November 2004.

Rainfall Collection Periods	
Start	**End**
2/9	2/19
3/13	3/20
3/30	4/5
4/10	4/13
4/22	4/23
5/27	11/30

Flow data from E2 was collected nearly continuously during the monitoring period. Two significant gaps exist in the data: first from March 18 through March 21, 2004, and June 2 through July 7, 2004. The initial gap in data occurred when “old” data was overwritten. The second gap resulted when the data logger was not re-initialized after downloading.

Though the WIKA SLLT® is a highly sensitive instrument, irregularly high currents and a daily oscillation in the current were regularly recorded shortly after installation. Figure 1 displays relatively normal flow data for three events that occurred in late May. Initially, oscillations were minor, with fluctuations of 0.21 amps (a difference in depth of 1.7 cm (0.67 in)). However, over time they increased to 2.4 amps (a difference in depth of 20 cm (7.6 in)), perhaps resulting from low battery power. The oscillation could also have been due to a daily heating effect, although the WIKA SLLT® is equipped with a temperature correcting function. The temperature sensor may have been damaged during freeze events early in the monitoring period and become less accurate as temperatures warmed over the summer. In an attempt to rectify the daily oscillation, a 24 hour moving average was calculated in place of the original data (Figure 2). However, the data depicts the water level rising with or without rainfall.

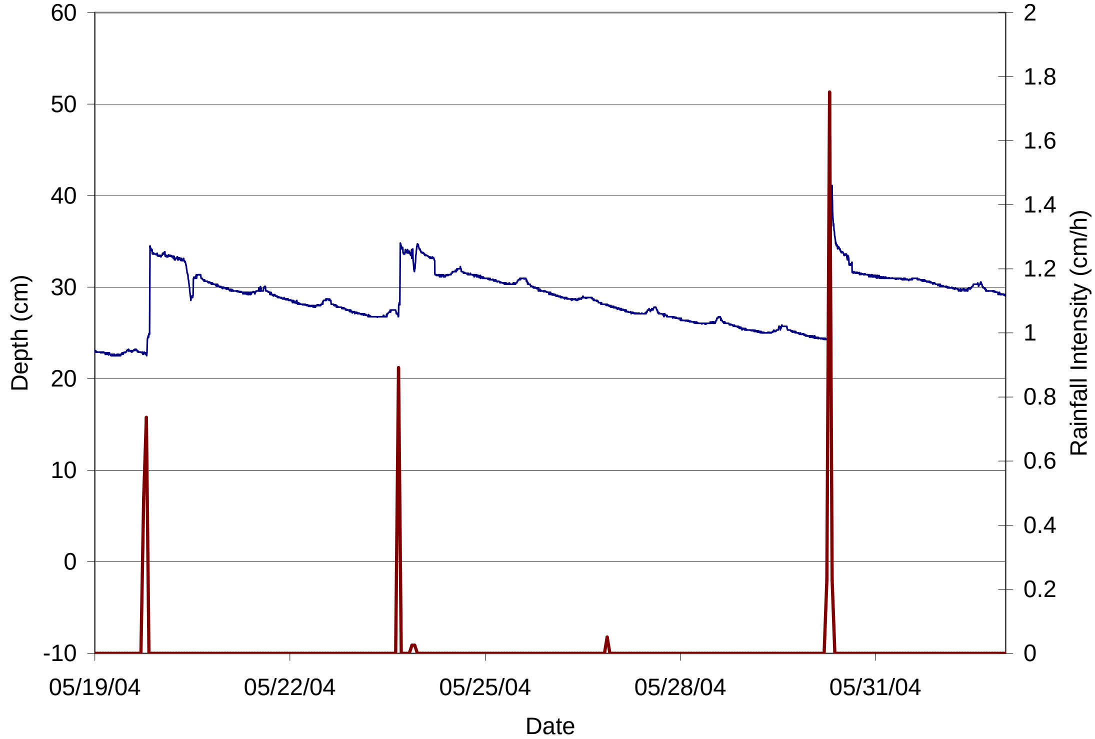


Figure 1. LWRF hourly rainfall and normal water level patterns for exfiltrate water depths from E2 at the Cary monitoring site during May 2004.

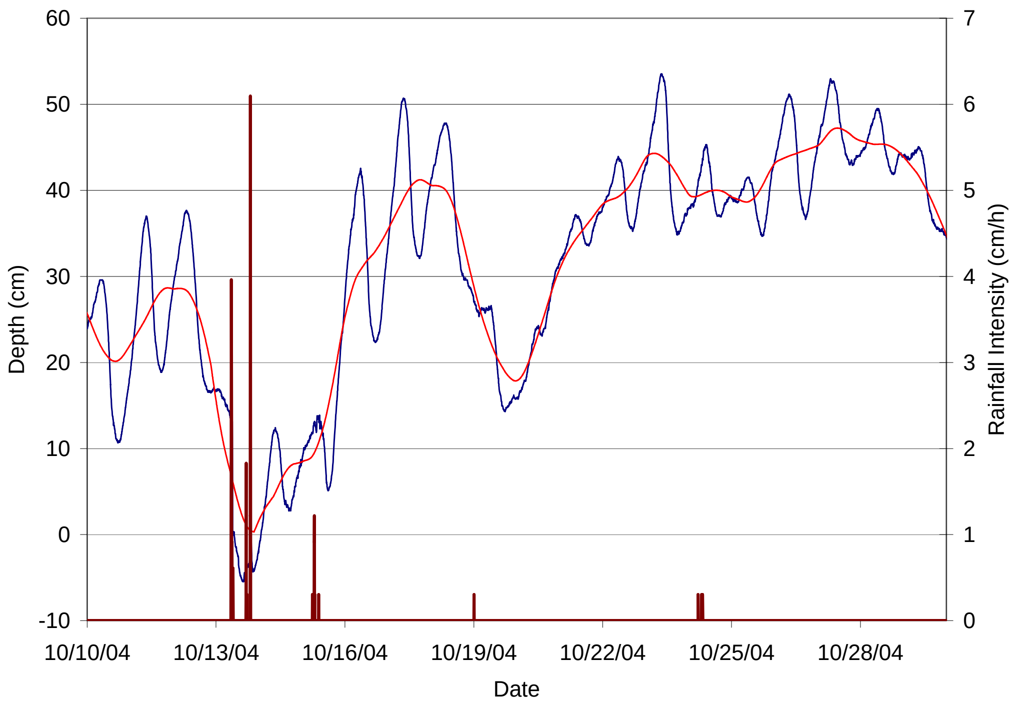


Figure 2. Raw flow data with 24 hr normalization for E2 water depths and on-site five-minute rainfall from the Cary site during October 2004.

Exfiltrate 1 (E1) was initially installed with a WIKA SLLT® and Omega Data Logger® (January 3 through March 18, 2004). Upon installation of the ISCO 6712 automatic sampler, the WIKA SLLT® and Omega Data Logger® were removed. The WIKA SLLT® and Omega system collected consistently reasonable data. The ISCO worked well from March until July, when numerous power failures started occurring. During the remainder of the monitoring period (July through November, 2004) power failures were a common occurrence. The ISCO used a 720 bubbler module to determine water depths. The ISCO bubbler module failed on June 4, 2004. The bubbler module was repaired and reinstalled on July 21, 2004. Several tests were completed, attempting to determine the cause for the frequent power drains. Many attempts were made to remedy the power failures (including replacement of the bubbler module, batteries, and solar panel), but none were successful. To accurately determine the effectiveness of the Cary site for infiltrating rainfall and reducing runoff, each of these three instruments must be operational and accurate; very rarely did this happen during the course of the study. However, with limited data, several rainfall events were analyzed to characterize the performance of the Cary site.

As mentioned earlier, to determine runoff rates the rainfall volumes and exfiltrate flow volumes must determined. Exfiltrate flow volumes were determined from flow rates that were based on flow depths. Currents recorded by the Omega Data Logger® from the WIKA SLLT® were converted to water level data by using Equation 1, where H is the depth of water behind the weir in meters, and A is the recorded current in milliamps from the WIKA SLLT®.

$$H = 0.0806 * A - 0.312 \qquad \text{(Equation 1)}$$

Equation 6 was given by literature from WIKA (2000) and then confirmed through calibration (Appendix I). With depth determined, flow was calculated. Equation 7 was provided by Grant and Dawson (2001) for flows (Q) in l/s over a 90^{o} V-notch weir with flow depths (H) over the weir in meters and was also confirmed through calibration (Appendix I). However, after considering the testing conditions, and relatively low flows used for calibration, it was determined that the supplied ISCO equation would be more reliable (Equation 2).

$$Q = 1380 * H^{2.5} \qquad \text{(Equation 2)}$$

The driveway was constructed with a liner underneath the gravel layer to prevent deep seepage and infiltration into the native soil below. The site was designed so that all infiltrate flow would leave via the exfiltrate flow pipes. The site was constructed so that no runoff from areas other than the driveway would infiltrate through the PICP. Any rainfall not exfiltrating the driveway was calculated as runoff. Small pockets of infiltrate most likely remained within the gravel storage basin; however, the volumes were assumed to be negligible.

It is important to note that for permeable pavements with lined gravel storage basins, like the site in Cary, no significant runoff volume attenuation occurs. Unless an additional BMP is employed, exfiltrate flow volumes will be similar to runoff volume leaving impervious surfaces of comparable size. Therefore, lined permeable pavements will offer relatively small reductions of water volume leaving a site.

Since event data for Cary were typically only available for rainfall and E1 data collection devices, and the site was divided into two gravel storage basins, the basins were analyzed both separately and together. Often no outflow data from E2 were measured during rainfall events, which was highly unlikely. Instead of presenting only combined data using an incorrect E2 volumes of 0 m^3, the data was split between the two basins and then combined to more accurately characterize the total exfiltrate volumes.

Table 2 shows runoff attenuation results from nine storms that occurred between February, 2004, and August, 2004, at the Cary site. Runoff attenuation percentages were the percent of calculated rainfall volumes for a drainage basin watershed area (either for E1 or E2) that passed through a corresponding exfiltrate pipe. For all but 2 events, E1 exfiltrate flow volumes exceeded the predicted rainfall inflow for its drainage area. More than 100% of the rainfall exfiltrated side 1 for seven of the nine events. This may be the result of runoff or exfiltrate from side 2 flowing into side 1, which effectively increased the watershed area for drainage basin 1 beyond its initial design.

Table 2. Runoff attenuation percents from Cary site between February 2004 and August 2004.

	Rainfall Total		**Runoff Attenuation %**			
Date	**(cm)**	**(in)**	**E1**	**E2**	**Total**	**Asphalt**
02/27/2004	1.07	(0.42)	146	1	64	59
04/12/2004	1.78	(0.70)	15	0	7	72
05/02/2004	2.87	(1.13)	73	0	31	81
07/22/2004	1.45	(0.57)	152	58	99	67
07/29/2004	1.55	(0.61)	105	24	59	69
08/06/2004	1.70	(0.67)	124	0	54	71
08/12/2004	2.97	(1.17)	120	0	52	82
08/13/2004	2.92	(1.15)	189	0	82	81
08/15/2004	2.46	(0.97)	148	0	64	79
Average	**2.09**	**(0.82)**	**119**	**9**	**56**	**75**

Three rainfall events occurred during a four-day period in August, totaling almost 8.5 cm (3 in). For each event, E1 had exfiltrate flow volumes exceeding rainfall volumes. For the two events in which E1 runoff attenuation percentages were less than 100 (April 12, 2004, and May 2, 2004), LWRF rainfall data was used to calculate rainfall infiltrate flows. Not including these two events, total runoff attenuation for individual storms ranged from 52 to 99%, compared to a range of 52 to 82% for the same rainfall on an asphalt surface using a CN of 98 (SCS, 1986). Assuming that higher than expected E1 outflows accounted for lower than expected E2 outflows, on average, 57% of all calculated rainfall inflows exfiltrated the PICP drainage basins.

In 2003, the driveway was noted to have localized partial clogging of pavers while testing the surface infiltration rate (Chapter 1, Appendix A 25), probably due to construction traffic. Since the clogging was localized, and not uniform, rainfall could have entirely ranoff from certain areas while completely infiltrated in others. This could explain why exfiltration volumes were not higher. However, it seems highly unlikely that exfiltrate from E2 would only flow during two of the three smallest events that occurred during the monitoring period.

Typically rational coefficients are used to describe perviousness of an area to predict peak runoff rates. However, by integration to determine total volumes (Equation 3), a rational coefficient was determined for entire events based on rainfall and calculated runoff volumes.

$$C = \frac{A * \int_0^t I}{\int_0^t R} \qquad \text{(Equation 3)}$$

Equation 3 is the integration of the Rational equation, solved for C using volumes instead of flowrates. Equation 3 integrates rainfall (I) and runoff (R) rates over their respective times of flow; A remains the assumed contributing area. Table 3, below, shows Rational Coefficients determined from exfiltrate volumes and rainfall using Equation 3. Typically, the SCS Curve Number method would be used to determine runoff volumes; however, when watersheds are extremely small, the modified Rational equation is used.

Table 3. Calculated Rational Coefficients for Cary PICP site for events greater than 1 cm depth occurring between February 2004 and August 2004

Date	Rainfall Total Depth (cm):	(in)	E1 Rational C	E2 Rational C	Total Rational C
2/27/2004	1.07	(0.42)	-0.46	0.99	0.36
4/12/2004	1.78	(0.70)	0.85	1.00	0.93
5/2/2004	2.87	(1.13)	0.27	1.00	0.69
7/22/2004	1.45	(0.57)	-0.52	0.42	0.01
7/29/2004	1.55	(0.61)	-0.05	0.76	0.41
8/6/2004	1.70	(0.67)	-0.24	1.00	0.46
8/12/2004	2.97	(1.17)	-0.20	1.00	0.48
8/13/2004	2.92	(1.15)	-0.89	1.00	0.18
8/15/2004	2.46	(0.97)	-0.49	1.00	0.36
Average	**2.09**	**(0.82)**	**-0.16**	**0.90**	**0.44**

The rational coefficients of E1 are less than zero, which corresponds with results in Table 2 of attenuations greater than 100%. However, Rational Coefficients are typically greater than zero, since infiltration volumes typically do not exceed rainfall volumes. Correspondingly, E2 coefficients were approximately equal to 1.00, suggesting no runoff attenuation which again corresponds to Table 2 where the average runoff attenuation percentage was less than 10%. Combined, data from E1 and E2 suggests an average Rational Coefficient of 0.44, much lower than typical impervious surfaces **(**0.96) (Malcom, 1989).

Peak runoff rates cannot be determined for a split flow system, with a gravel storage layer, by measuring rainfall and only one outflow, unless that storage volume is measured as well. Due to storage, exfiltrate flow periods are much longer than corresponding rainfall events, or infiltrate flows. Thus, exfiltrate flows, compared to infiltrate flows, are extended and reduced. This is depicted by illustrations (Figures 3 and 4), not actual data, shown below; times and flows were arbitrarily selected. Therefore, the peak runoff rate is not equal to the difference between the peak exfiltrate and rainfall rates, peak exfiltrate flow rates are also a function of the storage layer. If exfiltrate flows were equal to infiltrate flows, then the peak runoff rate would have been equal to the difference between peak rainfall rates and peak exfiltrate flow rates. This situation is the maximum possible peak runoff. In contrast, the minimum possible peak runoff rate is based on the rational coefficient for entire events. In this situation, runoff reduction occurs uniformly throughout events. Peak runoff rates are between these two, maximum and minimum, possible runoff flow rates. However, for conservative analysis of peak infiltrated rainfall, runoff flow rates were determined based on the difference between peak rainfall rates and peak exfiltrate flow rates. Using the maximum possible runoff flow rate results in determining the minimum possible infiltrate flow rate.

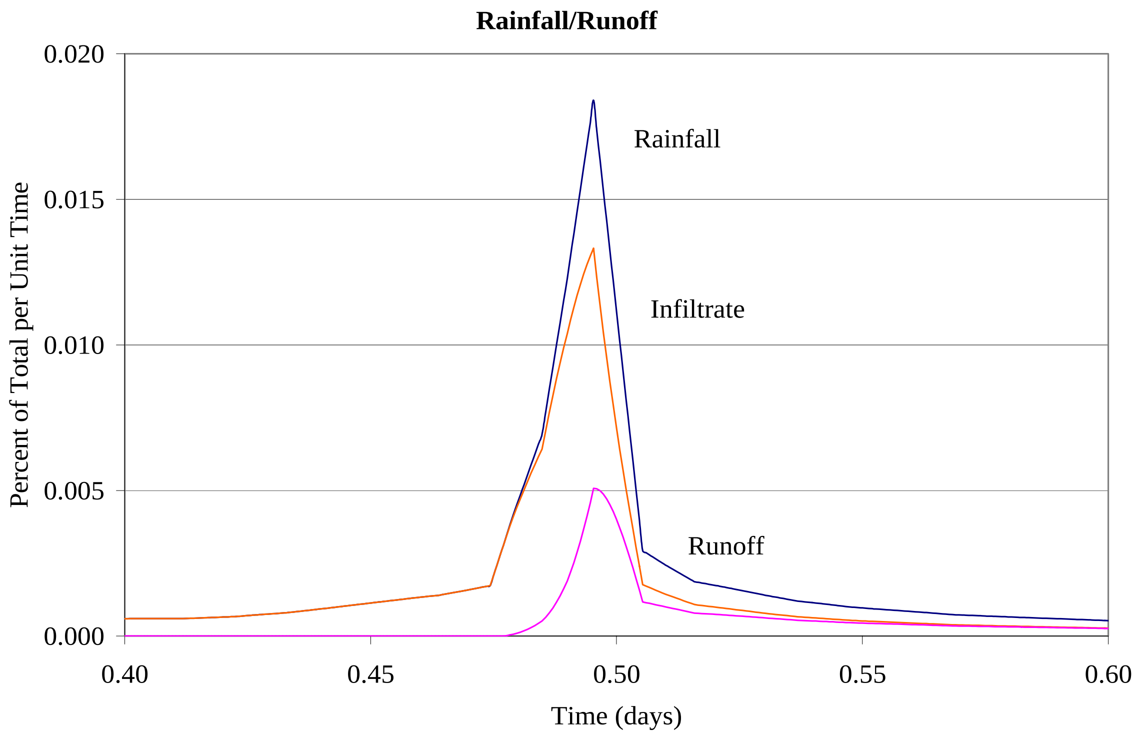


Figure 3. Representation of a possible relationship between rainfall, infiltrate and runoff from a PICP site.

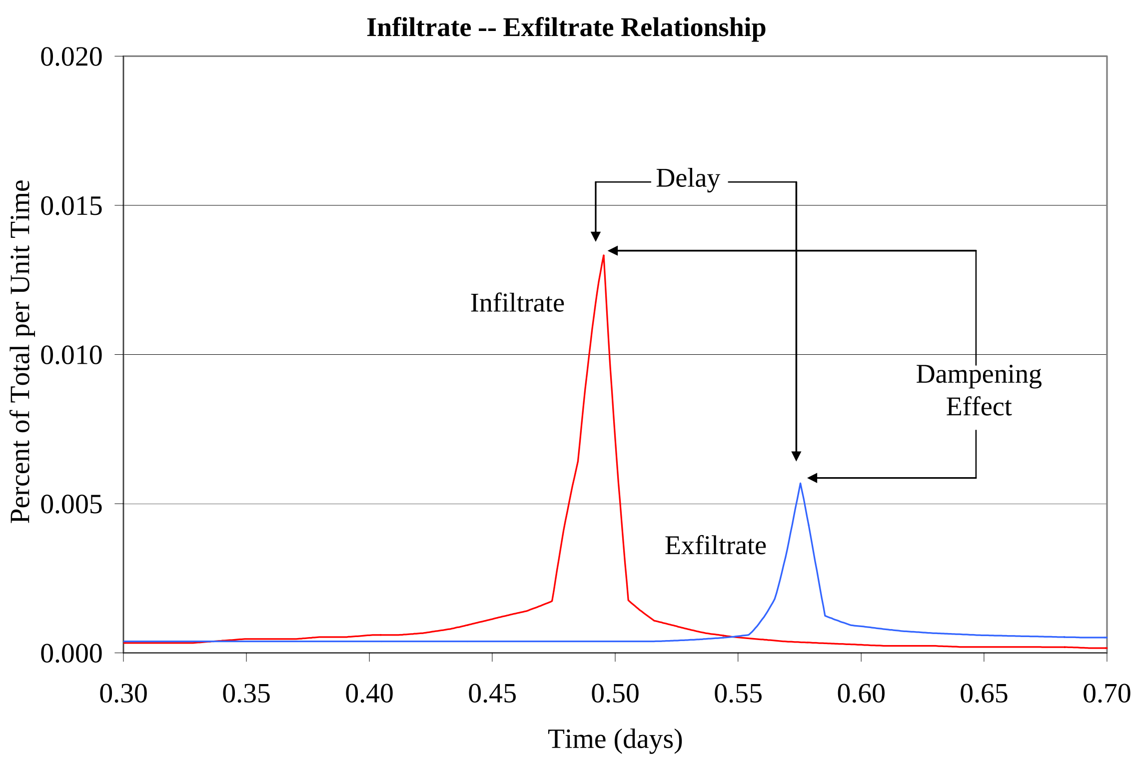


Figure 4. Representation of a possible flow relationship between infiltrate and exfiltrate.

Table 4 summarizes peak runoff flow rate attenuations and delays to peaks. Rainfall data from the April 12 and May 2, 2004, events were omitted from this analysis, because on-site five-minute data were not recorded, and therefore specific peak times could not be identified.

Table 4. Rainfall and outflow intensity data from Cary site for events that were greater than 1 cm total depth between February 2004 and August 2004.

Date	Maximum Five Minute Rainfall Intensity (cm/h)	(in/h)	% Peak Exfiltrate Rate: Peak Rainfall Rate (%)	Delay to Peak (h)
2/27/2004	0.36	(0.14)	33	0.33
7/22/2004	1.22	(0.48)	18	1.33
7/29/2004	2.74	(1.08)	43	1.50
8/6/2004	6.71	(2.64)	18	1.08
8/12/2004	7.32	(2.88)	21	1.17
8/13/2004	2.74	(1.08)	45	1.25
8/15/2004	1.52	(0.60)	33	1.33
Average:			**30**	**1.14**

Peak exfiltrate to peak rainfall ratios were calculated to characterize peak runoff attenuations. Ratios were calculated by dividing peak exfiltrate flow rates by peak rainfall flow rates. It can be seen that, on average, 30% of peak rainfalls infiltrated and ranged from 18 to 45%, for conservative runoff reduction calculations. Alternatively, the maximum runoff attenuation, based on the integrated Rational Coefficient (0.44) would be 56%. Therefore, the actual percent of the peak rainfall rate converted to runoff was, on average, between 44% and 70%, substantially higher than runoff rates reported by Hunt et al. (2002).

The delay to peak exfiltrate outflow, for events exceeding 1 cm/h (0.4 in/h), ranged from 1 to 1.5 hrs, with an average of 1.14 hrs. The delay to peak for these six events may be related to the rainfall intensity (Figure 5). The best-fit equation for this relationship, a slight exponential decay, predicts the delay to peak to be approximately 1.4 hrs with a rainfall intensity of 1 cm/h (0.4 in/h). The true exponential decay relationship most likely has an infinitely high delay to peak for extremely low intensities and an infinitely small, possibly 0, delay to peak extremely high intensities.

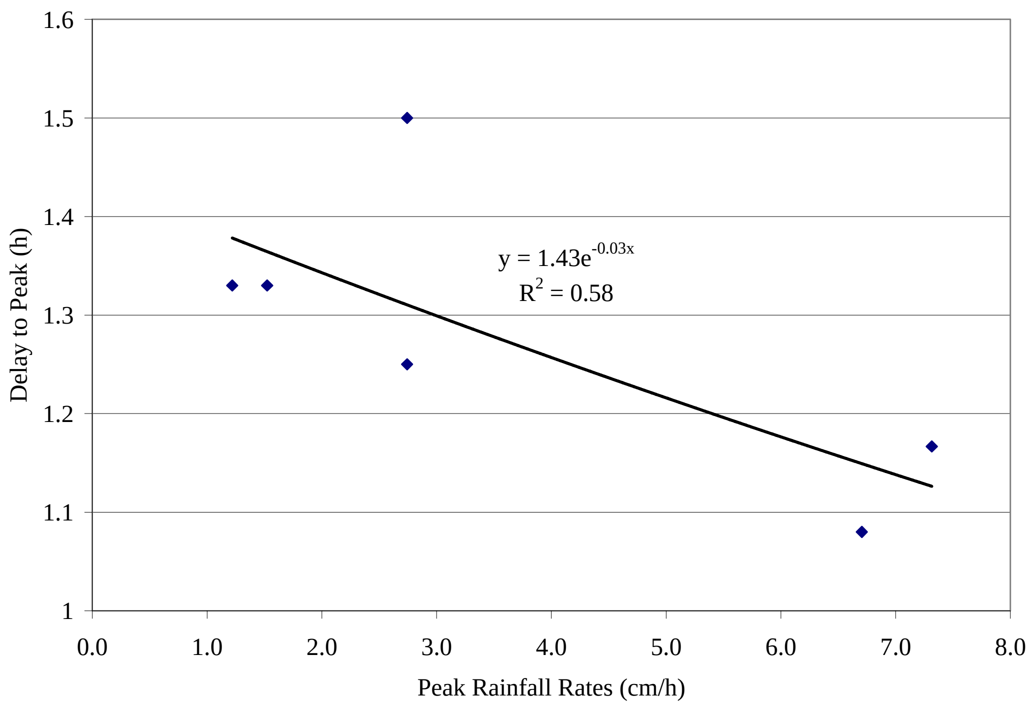


Figure 5. Delay to exfiltrate outflow peak versus peak rainfall rates relationship for the Cary PICP site for events with peak intensities greater than 1 cm/h between February 2004 and August 2004.

CONCLUSIONS

Data from the Cary flow monitoring site was used to determine that partial infiltration occurred. More rainfall event data sets need to be collected at the Cary site before general conclusions can be made about the effectiveness of the site. However, for the storms analyzed, the site showed the potential to infiltrate runoff, infiltrating 57% of rainfall on average. Peak runoff rates from the site were not accurately determined due to the effect of storage and lack of functioning flow monitoring equipment. Therefore, amount of peak rainfall infiltrated was between the long term infiltration percentage (56%) and the percent of peak exfiltrate rate to peak rainfall (30%). On average, the delay to peak between rainfall and exfiltrate flows was 1.14 hrs, for storms over 1.3 cm (0.5 in). Delays to peak may be dependent on the rainfall intensity, with a potential maximum delay of 1.4 hrs at minimal intensities for this site. Finally, lined PICP sites may not attenuate total volumes of water moving offsite, however, they may reduce and delay peak flows.

ACKNOWLEDGEMENTS

Thanks go to the North Carolina Department of Environmental Health and Natural Resources, which administers the EPA 319(h) grants, and helped fund this study. Thanks goes to Jonathan Smith and David Bidelspach for their assistance and guidance throughout this study. Thanks go to the Birklands and Rufty Homes for allowing monitoring of their site.

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WIKA Instrument Corporation (WIKA). (2000) High Performance Submersible Liquid Level Transmitters. Type: LH-10 Datasheet. Available at: http://www.wika.com/Content/Market%5C Support%5CSupport%5Ctech%20ref%20library%5CTronic%20Datasheets%5C LH-10.pdf

APPENDIX F

WATER QUALITY RESULTS

Goldsboro

Table 1. Total nitrogen concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and December 2004.

Event Date:	Exfiltrate (mg N/l)	Runoff (mg N/l)
6/11/2003	0.62	4.06
7/24/2003	1.98	1.25
8/15/2003	1.14	1.31
9/19/2003	1.63	0.49
9/23/2003	0.27	1.24
10/9/2003	1.09	1.03
2/4/2004	0.36	1.23
2/13/2004	0.19	0.53
8/3/2004	1.78	1.26
8/4/2004	1.63	1.90
8/6/2004	1.39	1.45
8/16/2004	0.78	1.20
10/14/2004	0.30	1.96
12/12/2004	0.57	2.33
Average:	0.98	1.52

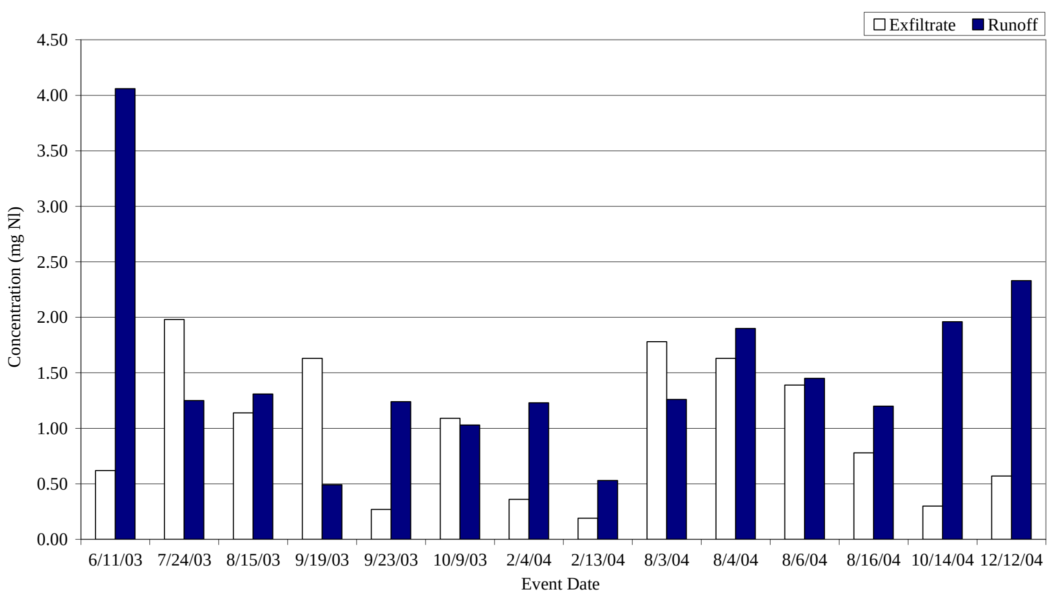

Figure 1. Total nitrogen concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and December 2004.

Table 2. Nitrate-nitrite concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and December 2004.

Event Date:	Exfiltrate (mg N/l)	Runoff (mg N/l)
6/11/2003	0.37	0.01
7/24/2003	0.10	0.27
8/15/2003	0.37	0.80
9/19/2003	0.91	0.02
9/23/2003	0.27	0.01
10/9/2003	0.78	0.07
2/4/2004	0.36	0.17
2/13/2004	0.19	0.05
8/3/2004	0.78	0.28
8/4/2004	0.83	0.80
8/6/2004	0.83	0.50
8/16/2004	0.28	0.20
10/14/2004	0.05	0.66
12/12/2004	0.05	0.33
Average:	0.44	0.30

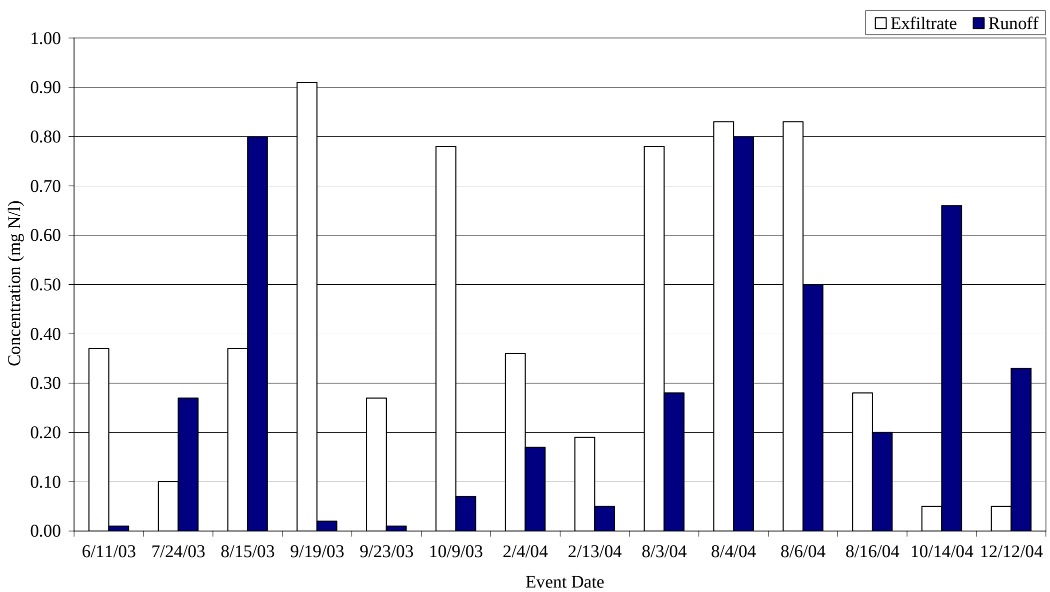

Figure 2. Nitrate-nitrite concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and December 2004.

Table 3. Total Kjeldahl nitrogen concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and December 2004.

Event Date:	Exfiltrate (mg N/l)	Runoff (mg N/l)
6/11/2003	0.25	4.06
7/24/2003	1.88	0.98
8/15/2003	0.77	0.51
9/19/2003	0.72	0.47
9/23/2003	0.13	1.24
10/9/2003	0.31	0.96
2/4/2004	0.13	1.06
2/13/2004	0.05	0.48
8/3/2004	1.00	0.98
8/4/2004	0.80	1.10
8/6/2004	0.56	0.95
8/16/2004	0.50	1.00
10/14/2004	0.20	1.30
12/12/2004	0.47	2.00
Average:	0.55	1.22

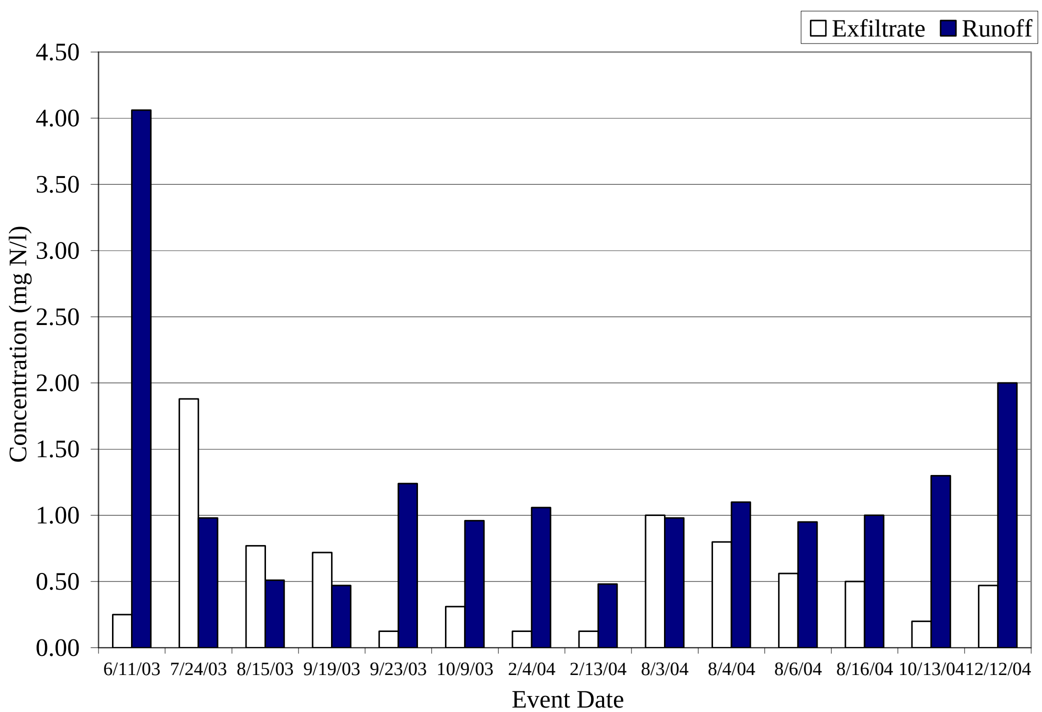

Figure 3. Total Kjeldahl nitrogen concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and December 2004.

Table 4. Ammonia concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between August 2004 and December 2004.

Event Date	Exfiltrate (mg N/l)	Runoff (mg N/l)
8/3/2004	0.05	0.17
8/4/2004	0.05	0.41
8/6/2004	0.05	0.35
8/16/2004	0.05	0.18
10/14/2004	0.05	0.65
12/12/2004	0.05	0.31
Average:	0.05	0.35

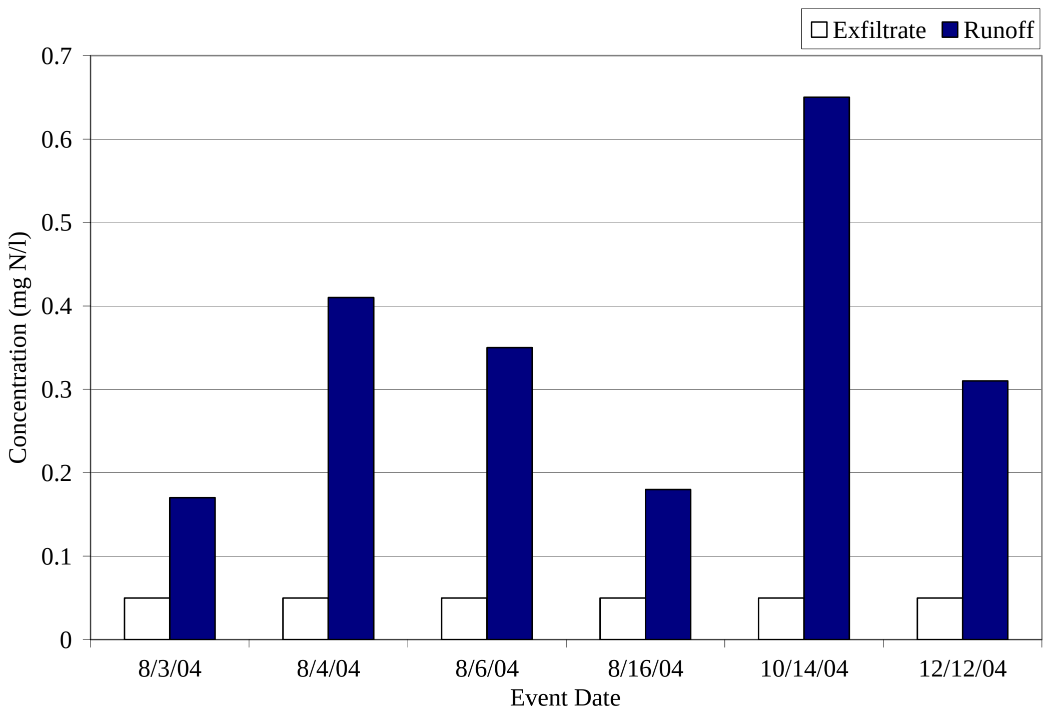

Figure 4. Ammonia in water concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between August 2004 and December 2004.

Table 5. Organic nitrogen concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between August 2004 and December 2004.

Event Date:	Exfiltrate (mg N/l)	Runoff (mg N/l)
8/3/2004	0.95	0.81
8/4/2004	0.75	0.69
8/6/2004	0.51	0.60
8/16/2004	0.45	0.82
10/14/2004	0.15	0.65
12/12/2004	0.42	1.69

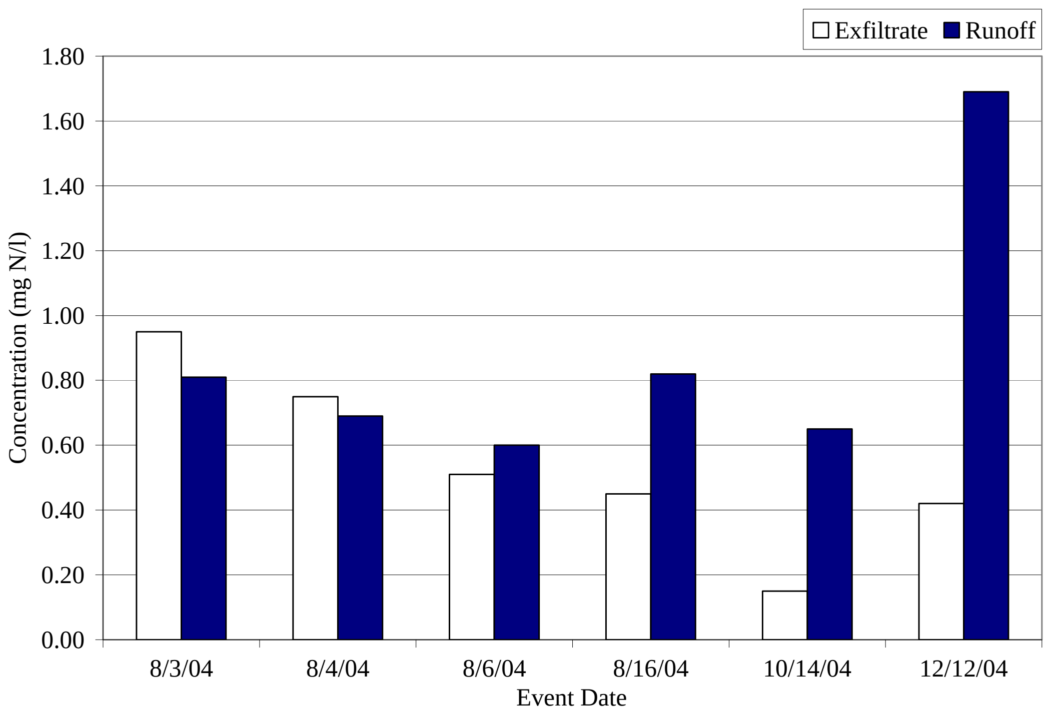

Figure 5. Organic nitrogen concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between August 2004 and December 2004.

Table 6. Total Phosphorus in water concentrations of PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and December 2004.

Event Date:	Exfiltrate (mg P/l)	Runoff (mg P/l)
6/11/2003	0.025	0.36
7/24/2003	0.025	0.05
8/15/2003	0.08	0.06
9/19/2003	0.025	0.12
9/23/2003	0.28	0.98
10/9/2003	0.025	0.2
2/4/2004	0.025	0.21
2/13/2004	0.01	0.03
8/3/2004	0.07	0.09
8/4/2004	0.12	0.06
8/6/2004	0.09	0.12
8/16/2004	0.09	0.14
10/14/2004	0.03	0.16
12/12/2004	0.09	0.25
Average:	0.07	0.20

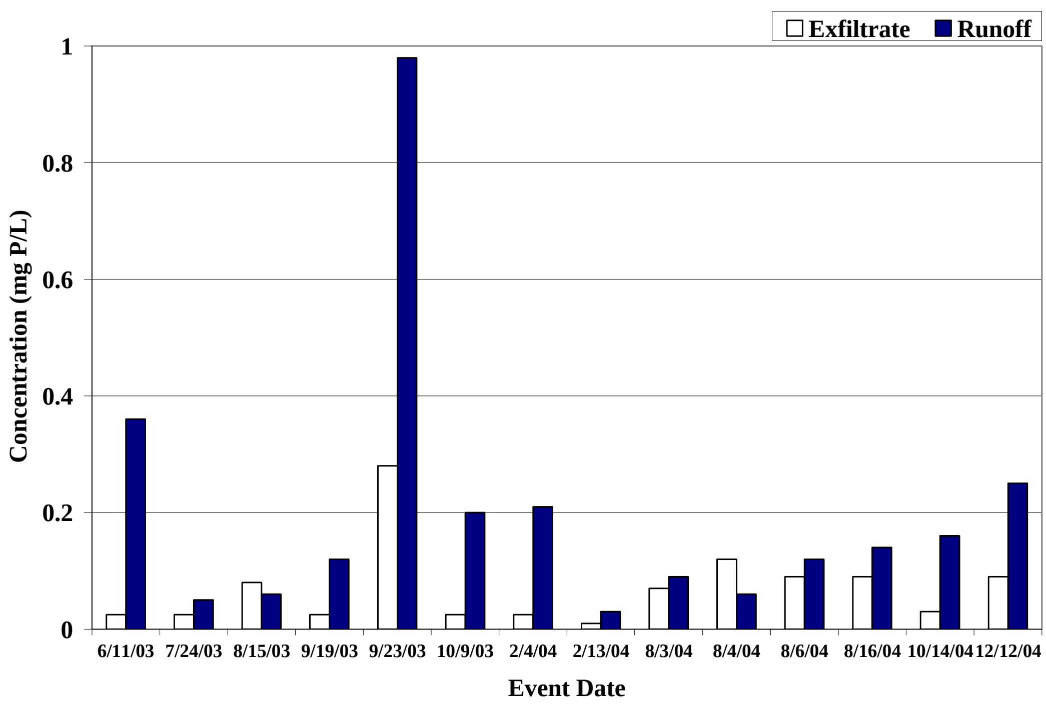

Figure 6. Total phosphorus concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and December 2004.

Table 7. Orthophosphate concentrations of PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between August 2004 and December 2004.

Event Date:	Exfiltrate (mg P/l)	Runoff (mg P/l)
8/3/2004	0.03	0.02
8/4/2004	0.02	0.02
8/6/2004	0.02	0.02
8/16/2004	0.02	0.02
10/14/2004	0.01	0.11
12/12/2004	0.05	0.17
Average:	0.03	0.06

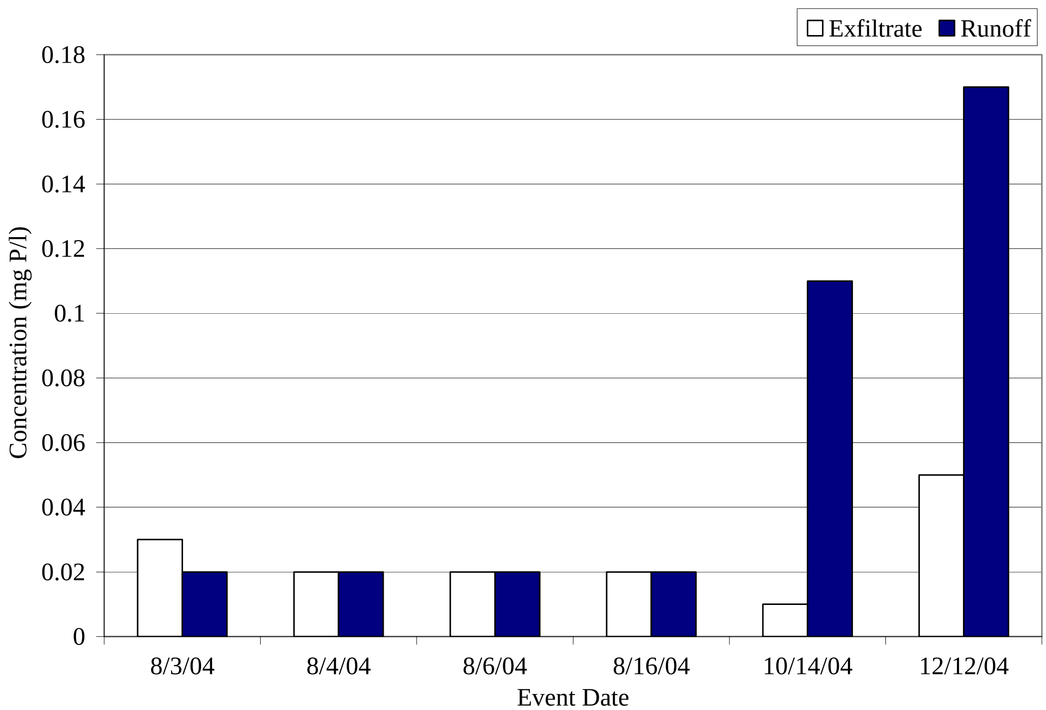

Figure 7. Orthophosphate concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between August 2004 and December 2004.

Table 8. Bound phosphorus concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between August 2004 and December 2004.

Event Date:	Exfiltrate (mg P/l)	Runoff (mg P/l)
8/3/2004	0.040	0.070
8/4/2004	0.100	0.040
8/6/2004	0.070	0.100
8/16/2004	0.070	0.120
10/14/2004	0.020	0.050
12/12/2004	0.040	0.080

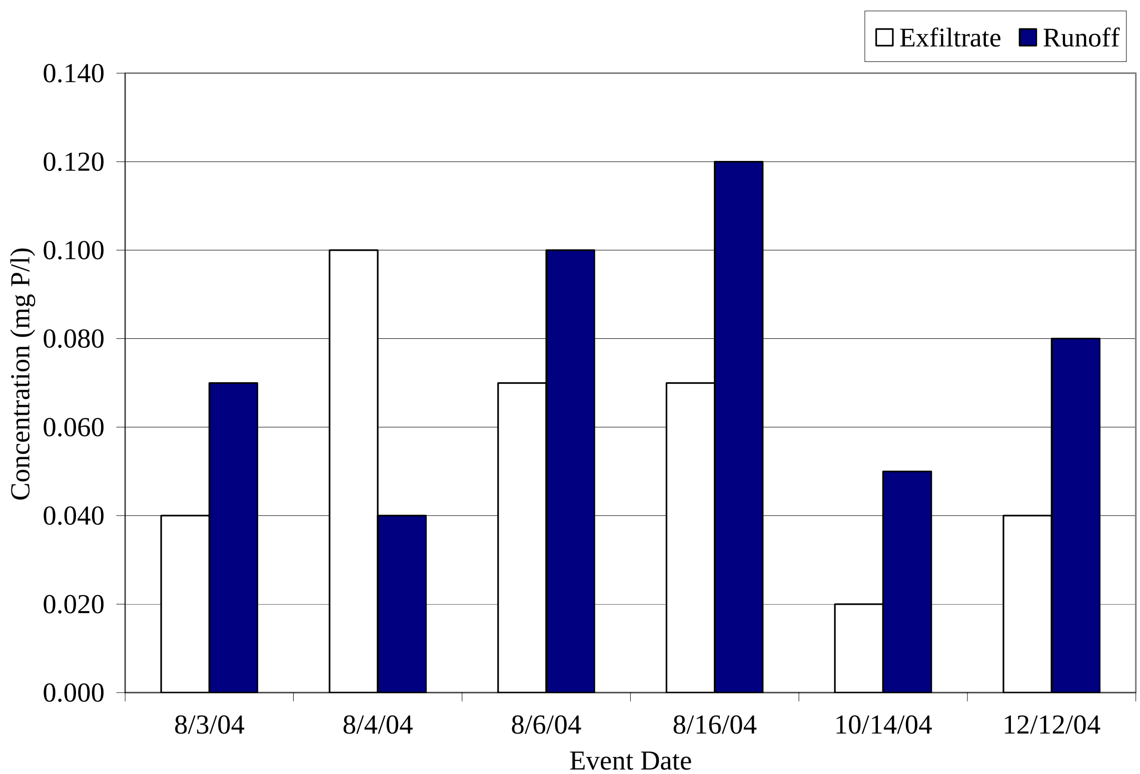

Figure 8. Bound phosphorus concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between August 2004 and December 2004.

Table 9. Total suspended solids concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and December 2004.

Event Date:	Exfiltrate (mg/l)	Runoff (mg/l)
6/11/2003	7	6
7/24/2003	15	8
8/15/2003	16	119
9/19/2003	15	7
9/23/2003	5	18
10/9/2003	3	12
2/4/2004	0	96
2/13/2004	8	10
8/3/2004	7	10
8/4/2004	10	236
8/6/2004	4	21
8/16/2004	9	5
12/12/2004	63	22
Average:	12	44

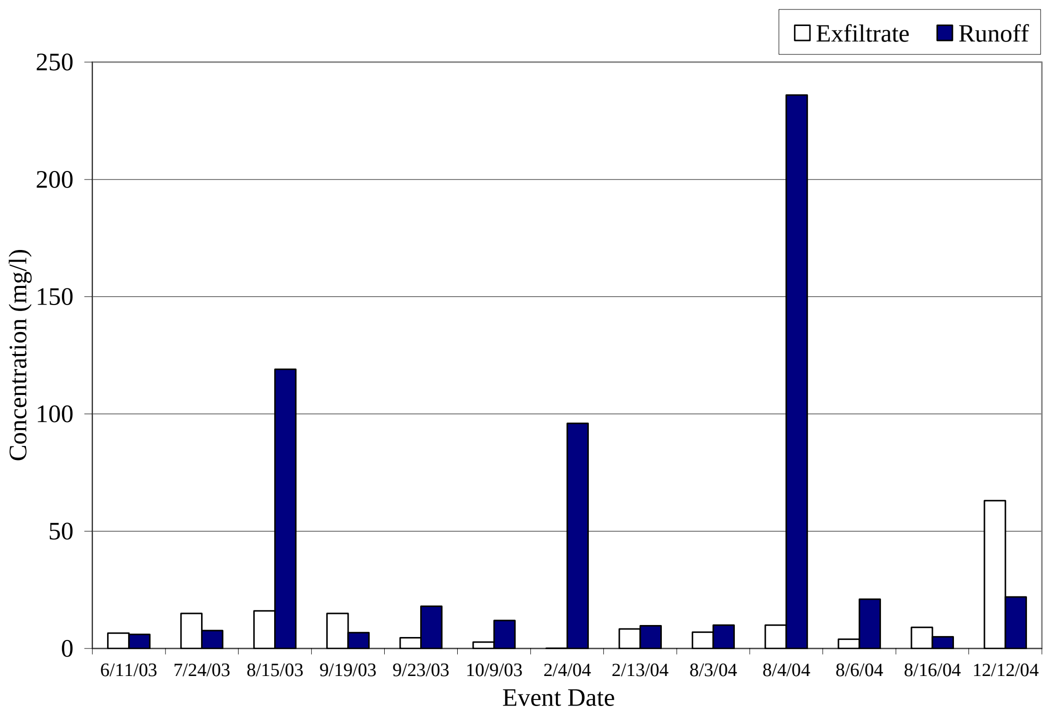

Figure 9. Total suspended solids concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and December 2004.

Table 10. Copper concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and February 2004.

Event Date:	Exfiltrate (mg/l)	Runoff (mg/l)
6/11/03	0.012	0.005
7/24/03	0.005	0.005
8/15/03	0.005	0.018
9/19/03	0.005	0.014
9/23/03	0.005	0.042
10/9/03	0.005	0.011
2/4/04	0.005	0.027
2/13/04	0.005	0.005
Average:	0.006	0.016

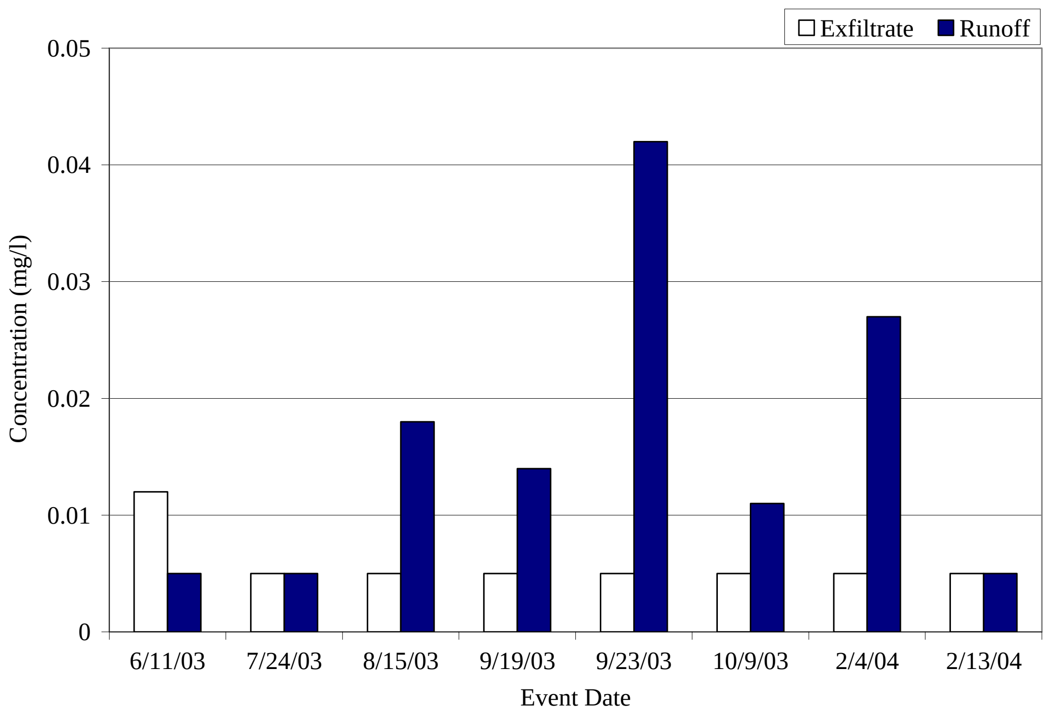

Figure 10. Copper concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and February 2004.

Table 11. Zinc concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and February 2004.

Event Date:	Exfiltrate (mg/l)	Runoff (mg/l)
6/11/03	0.005	0.066
7/24/03	0.005	0.036
8/15/03	0.005	0.053
9/19/03	0.012	0.052
9/23/03	0.012	0.107
10/9/03	0.012	0.060
2/4/04	0.011	0.093
2/13/04	0.005	0.068
Average:	0.008	0.067

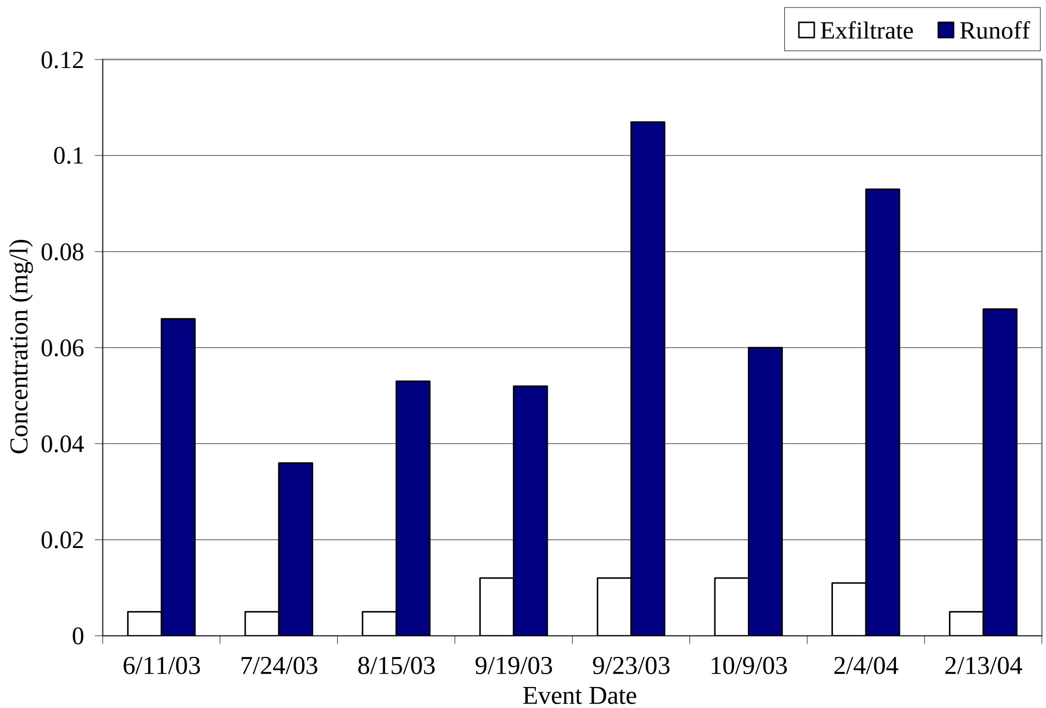

Figure 11. Zinc concentrations for PICP exfiltrate and asphalt runoff from the Goldsboro site for collected samples from events between June 2003 and February 2004.

Cary

Table 12. Total nitrogen concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Event Date:	Rain (mg N/l)	Exfiltrate (mg N/l)
2/9/04	0.49	3.05
2/13/04	0.45	2.10
4/13/04	1.45	2.99
4/28/04	3.51	3.90
5/4/04	1.48	1.15
5/27/04	9.39	2.48
5/31/04	1.37	5.50
6/4/04	2.13	9.00
8/3/04	1.86	2.59
8/13/04	1.45	2.70
8/16/04	2.06	2.80
10/25/04	12.70	0.64
11/24/04	1.01	0.68
12/1/04	0.49	0.90
12/24/04	0.86	1.01
Average:	2.71	2.77

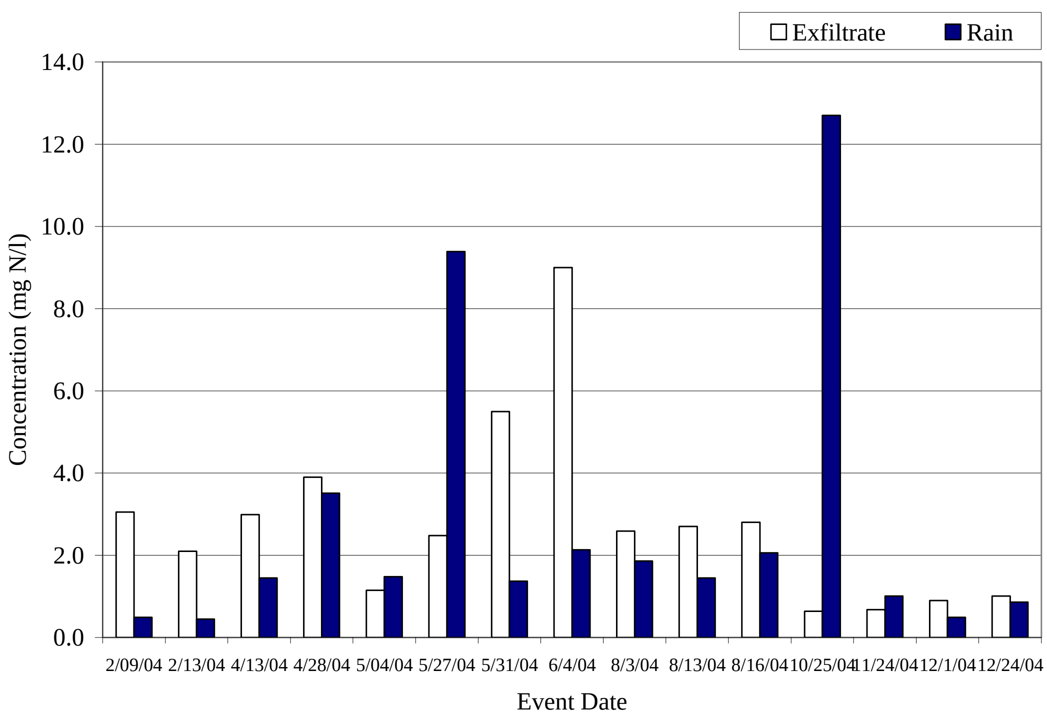

Figure 12. Total nitrogen concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Table 13. Nitrate-nitrite concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Event Date:	Rain (mg N/l)	Exfiltrate (mg N/l)
2/9/04	0.14	2.20
2/13/04	0.27	1.50
4/13/04	0.05	2.00
4/28/04	0.11	2.80
5/4/04	0.18	0.05
5/27/04	0.29	1.50
5/31/04	0.17	3.50
6/4/04	0.33	7.60
8/3/04	0.26	0.89
8/13/04	0.15	1.30
8/16/04	0.16	1.20
10/25/04	2.90	0.05
11/24/04	0.25	0.05
12/1/04	0.36	0.05
12/24/04	0.20	0.14
Average:	0.39	1.66

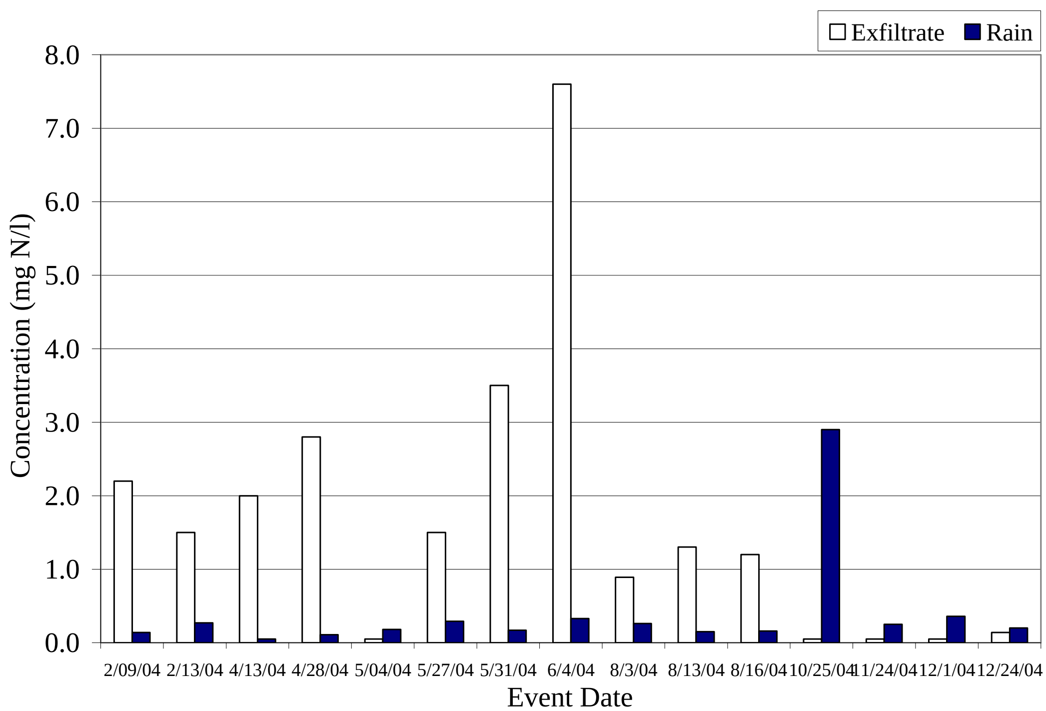

Figure 13. Nitrate-nitrite concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Table 14. Total Kjeldahl nitrogen concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Event Date:	Rain (mg N/l)	Exfiltrate (mg N/l)
2/9/04	0.35	0.85
2/13/04	0.18	0.60
4/13/04	1.40	0.99
4/28/04	3.40	1.10
5/4/04	1.30	1.10
5/27/04	9.10	0.98
5/31/04	1.20	2.00
6/4/04	1.80	1.40
8/3/04	1.60	1.70
8/13/04	1.30	1.40
8/16/04	1.90	1.60
10/25/04	9.80	0.59
11/24/04	0.76	0.63
12/1/04	0.50	0.96
12/24/04	0.29	0.76
Average:	2.33	1.11

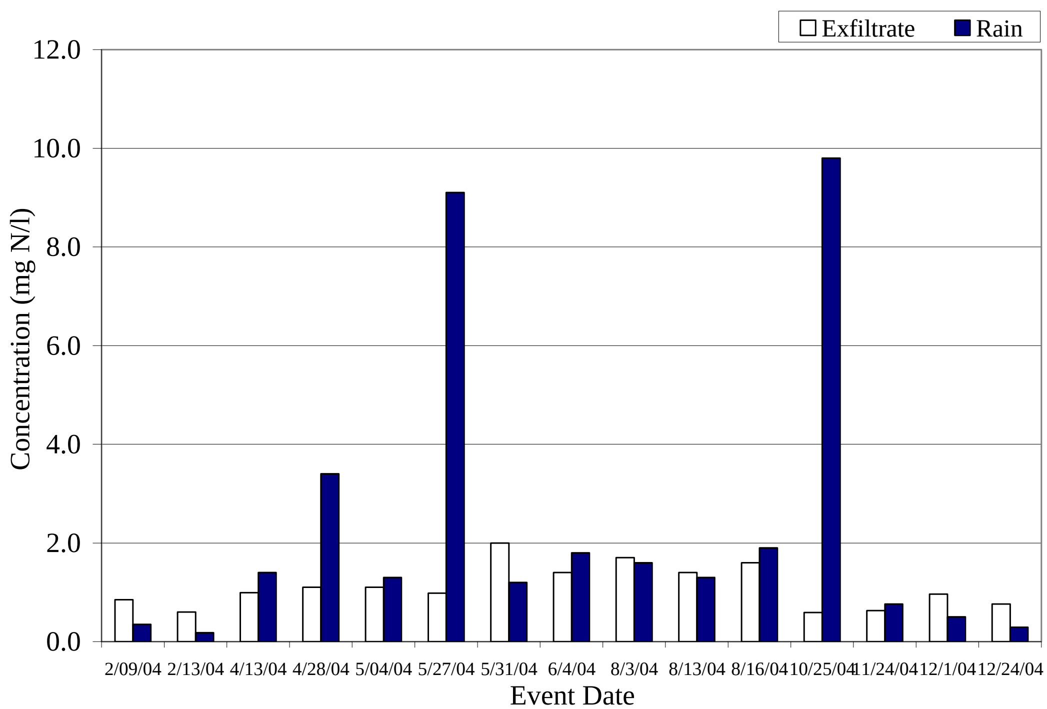

Figure 14. Total Kjeldahl nitrogen concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Table 15. Ammonia in water concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Event Date:	Rain (mg N/l)	Exfiltrate (mg N/l)
2/9/04	0.12	0.05
2/13/04	0.05	0.05
4/13/04	0.72	0.05
4/28/04	1.30	0.05
5/4/04	0.05	0.05
5/27/04	2.80	0.05
5/31/04	0.05	0.05
6/4/04	0.33	0.05
8/3/04	0.30	0.05
8/13/04	0.12	0.05
8/16/04	0.11	0.05
10/25/04	2.90	0.05
11/24/04	0.23	0.05
12/1/04	0.35	0.05
12/24/04	0.19	0.13
Average:	0.64	0.06

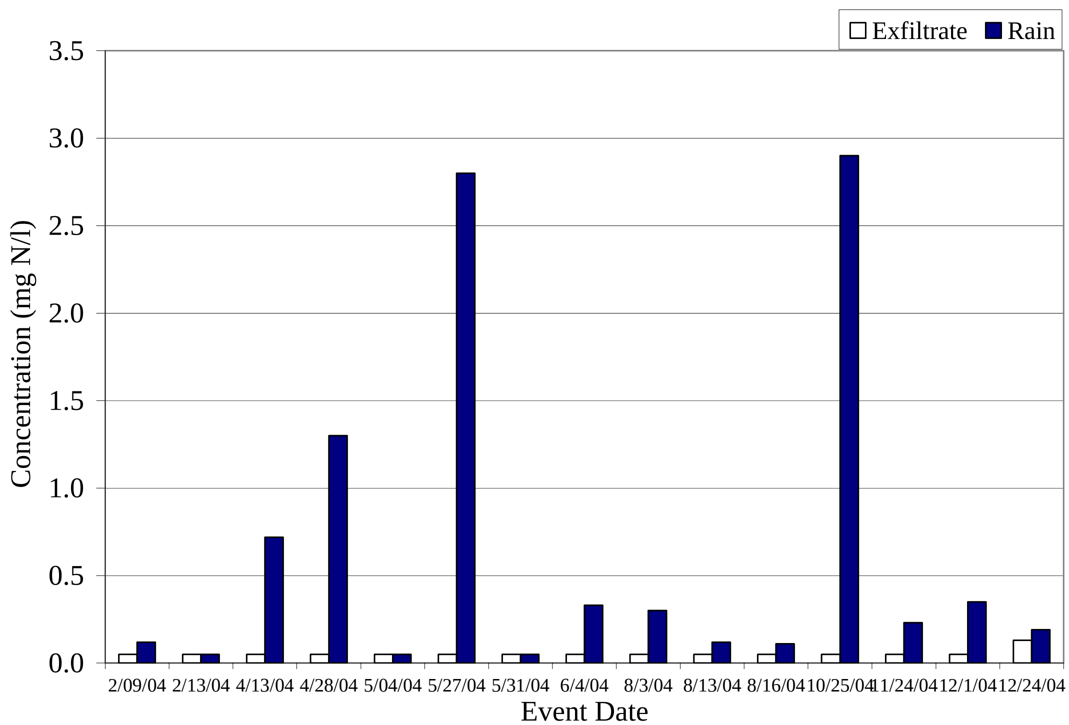

Figure 15. Ammonia in water concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Table 16. Organic nitrogen concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Event Date:	Rain (mg N/l)	Exfiltrate (mg N/l)
2/9/04	0.23	0.80
2/13/04	0.13	0.55
4/13/04	0.68	0.94
4/28/04	2.10	1.05
5/4/04	1.25	1.05
5/27/04	6.30	0.93
5/31/04	1.15	1.95
6/4/04	1.47	1.35
8/3/04	1.30	1.65
8/13/04	1.18	1.35
8/16/04	1.79	1.55
10/25/04	6.90	0.54
11/24/04	0.53	0.58
12/1/04	0.15	0.91
12/24/04	0.10	0.63

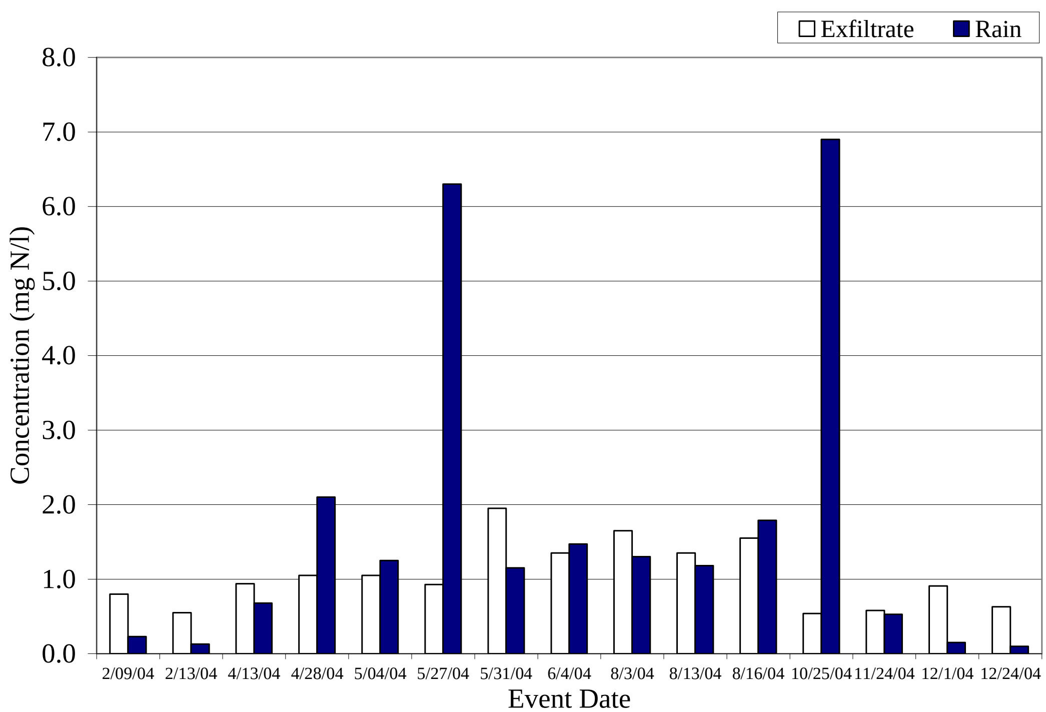

Figure 16. Organic nitrogen concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Table 17. Total phosphorus concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Event Date:	Rain (mg P/l)	Exfiltrate (mg P/l)
2/9/04	0.01	0.16
2/13/04	0.05	0.18
4/13/04	0.16	0.3
4/28/04	0.62	0.18
5/4/04	0.21	0.3
5/27/04	0.77	0.12
5/31/04	0.22	0.27
6/4/04	0.16	2.6
8/3/04	0.37	0.35
8/13/04	0.23	0.33
8/16/04	0.1	0.27
10/25/04	0.75	0.17
11/24/04	0.03	0.26
12/1/04	0.09	0.35
12/24/04	0.06	0.22
Average:	0.26	0.40

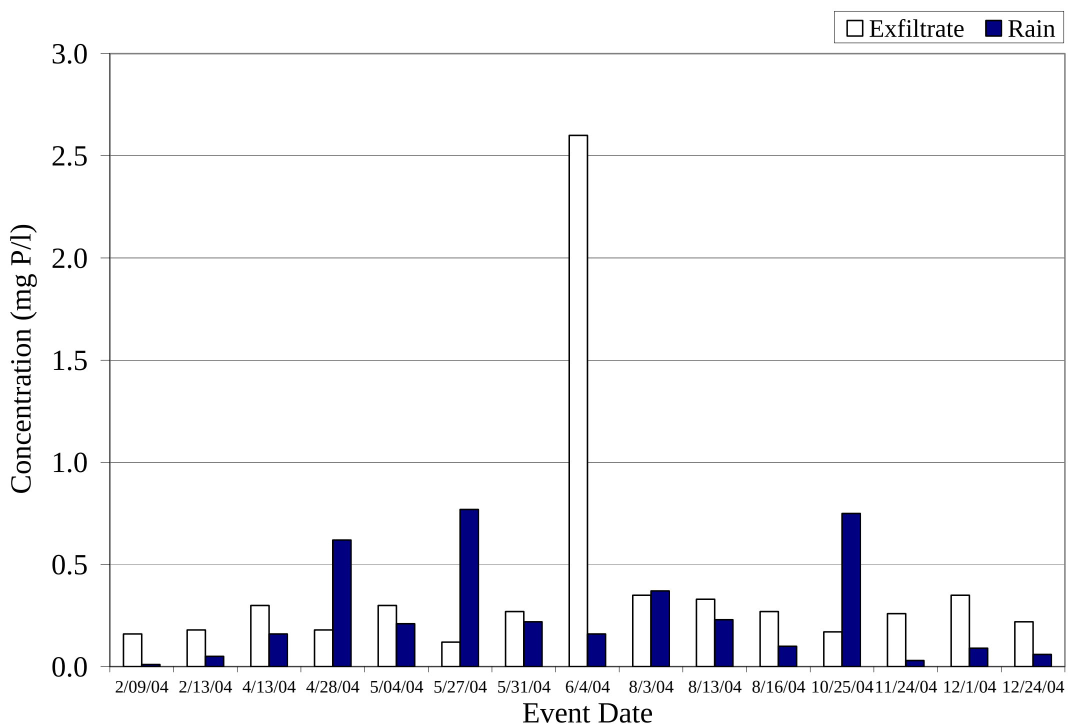

Figure 17. Total phosphorus concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Table 18. Orthophosphate concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Event Date:	Rain (mg P/l)	Exfiltrate (mg P/l)
2/9/04	0.005	0.160
2/13/04	0.005	0.080
4/13/04	0.050	0.140
4/28/04	0.180	0.110
5/4/04	0.010	0.240
5/27/04	0.150	0.100
5/31/04	0.005	0.250
6/4/04	0.020	2.400
8/3/04	0.060	0.280
8/13/04	0.010	0.220
8/16/04	0.005	0.180
10/25/04	0.720	0.160
11/24/04	0.005	0.240
12/1/04	0.020	0.330
12/24/04	0.005	0.220
Average:	0.083	0.341

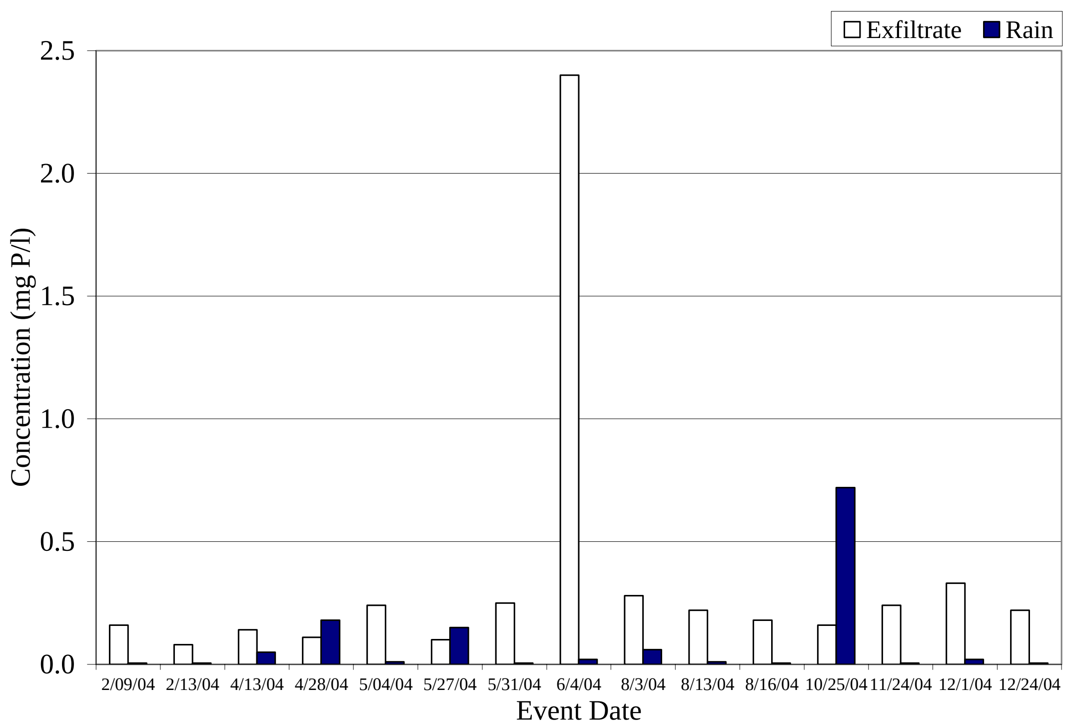

Figure 18. Orthophosphate concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Table 19. Bound phosphorus concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Event Date:	Rain (mg P/l)	Exfiltrate (mg P/l)
2/9/04	0.005	0.010
2/13/04	0.045	0.100
4/13/04	0.110	0.160
4/28/04	0.440	0.070
5/4/04	0.200	0.060
5/27/04	0.620	0.020
5/31/04	0.215	0.020
6/4/04	0.140	0.200
8/3/04	0.310	0.070
8/13/04	0.220	0.110
8/16/04	0.095	0.090
10/25/04	0.030	0.010
11/24/04	0.025	0.020
12/1/04	0.070	0.020
12/24/04	0.055	0.010

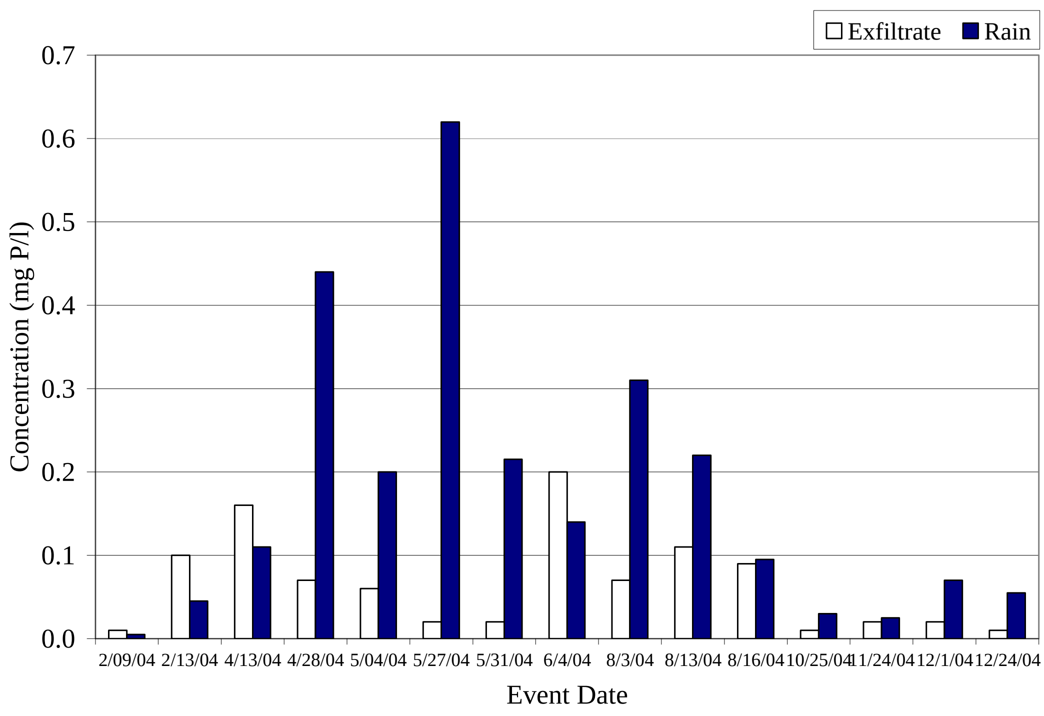

Figure 19. Bound phosphorus concentrations for PICP exfiltrate and rain from the Cary site for collected samples from events between February 2004 and December 2004.

Table 20. Total suspended solids concentrations for PICP exfiltrate from the Cary site for collected samples from events between February 2004 and December 2004.

Event Date:	Exfiltrate (mg/l)
2/9/04	16.5
2/13/04	7.0
4/13/04	16.0
4/28/04	4.5
5/4/04	11.0
5/27/04	10.0
5/31/04	2.5
6/4/04	28.0
8/3/04	1.0
8/13/04	31.0
8/16/04	8.0
10/25/04	6.0
11/24/04	14.0
12/1/04	16.0
Average:	12.3

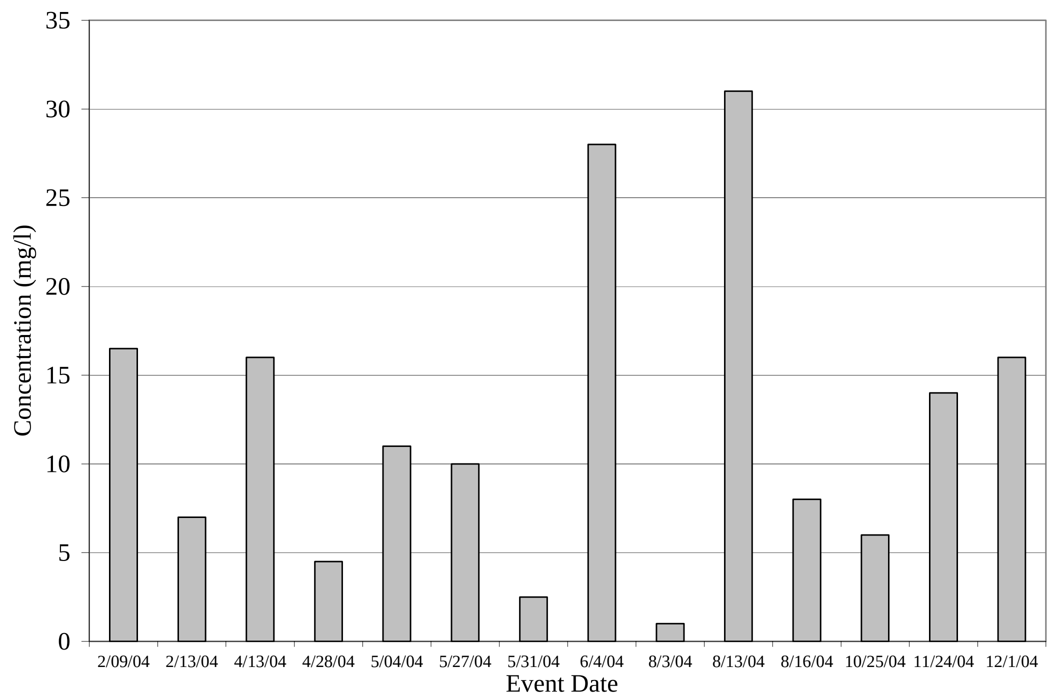

Figure 20. Total suspended solids concentrations for PICP exfiltrate from the Cary site for collected samples from events between February 2004 and December 2004.

Swansboro

Table 21. All concentrations for PICP exfiltrate collected from the Swansboro site for event samples collected between March 2004 and November 2004.

Event Date:	TN (mg N/l)	NO_{2+3}-N (mg N/l)	TKN (mg N/l)	NH_4-N (mg N/l)	ON (mg N/l)
3/2/04	0.15	0.10	0.05	0.05	0.00
3/16/04	0.35	0.20	0.15	0.05	0.10
3/29/04	0.37	0.17	0.2	0.05	0.15
4/1/04	0.41	0.22	0.19	0.05	0.14
4/12/04	0.56	0.36	0.2	0.05	0.15
4/13/04	0.47	0.29	0.18	0.05	0.13
5/3/04	0.40	0.19	0.21	0.05	0.16
6/1/04	0.65	0.36	0.29	0.05	0.24
6/11/04	0.74	0.32	0.42	0.05	0.37
8/3/04	0.93	0.28	0.65	0.05	0.60
8/13/04	0.10	0.05	0.05	0.05	0.00
8/19/04	0.10	0.05	0.05	0.05	0.00
9/2/04	0.10	0.05	0.05	0.05	0.00
9/9/04	0.19	0.05	0.14	0.05	0.09
11/13/04	0.10	0.05	0.05	0.05	0.00
11/29/04	0.10	0.05	0.05	0.05	0.00
Average:	0.36	0.17	0.18	0.05	0.13

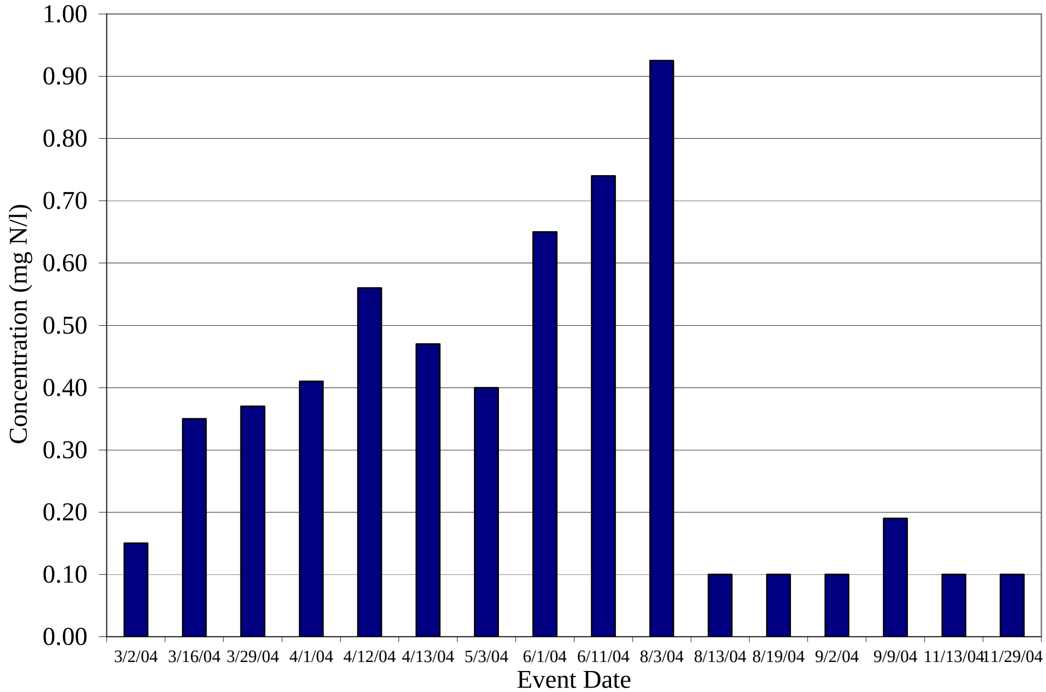

Figure 21. Total nitrogen concentrations for PICP exfiltrate collected from the Swansboro site for event samples collected between March 2004 and November 2004.

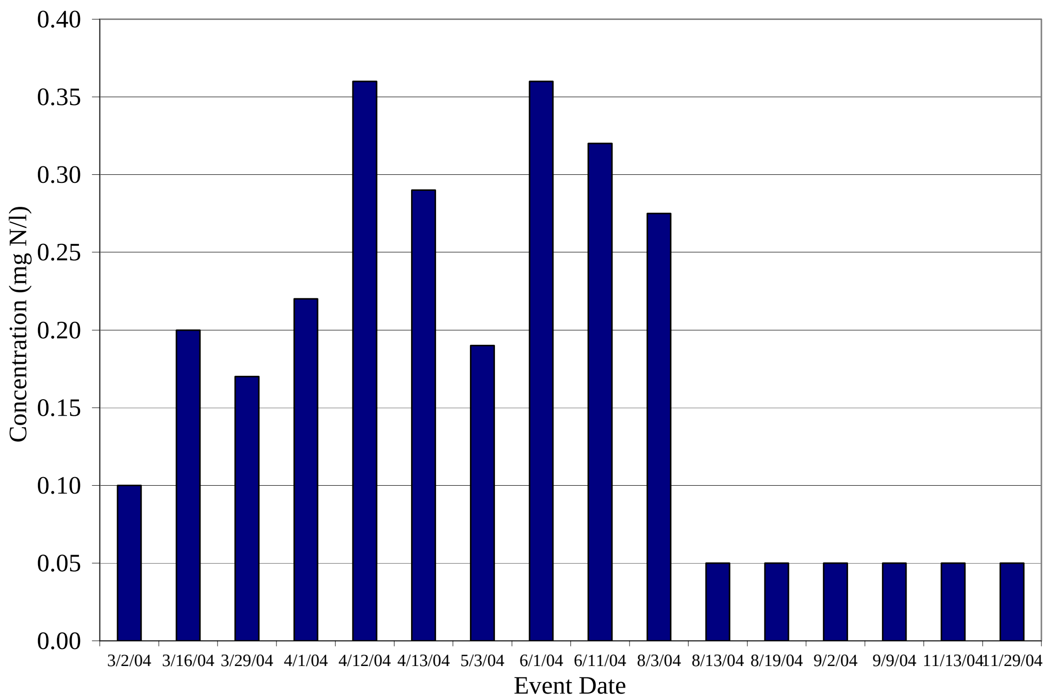

Figure 22. Nitrate-nitrite concentrations for PICP exfiltrate collected from the Swansboro site for event samples collected between March 2004 and November 2004.

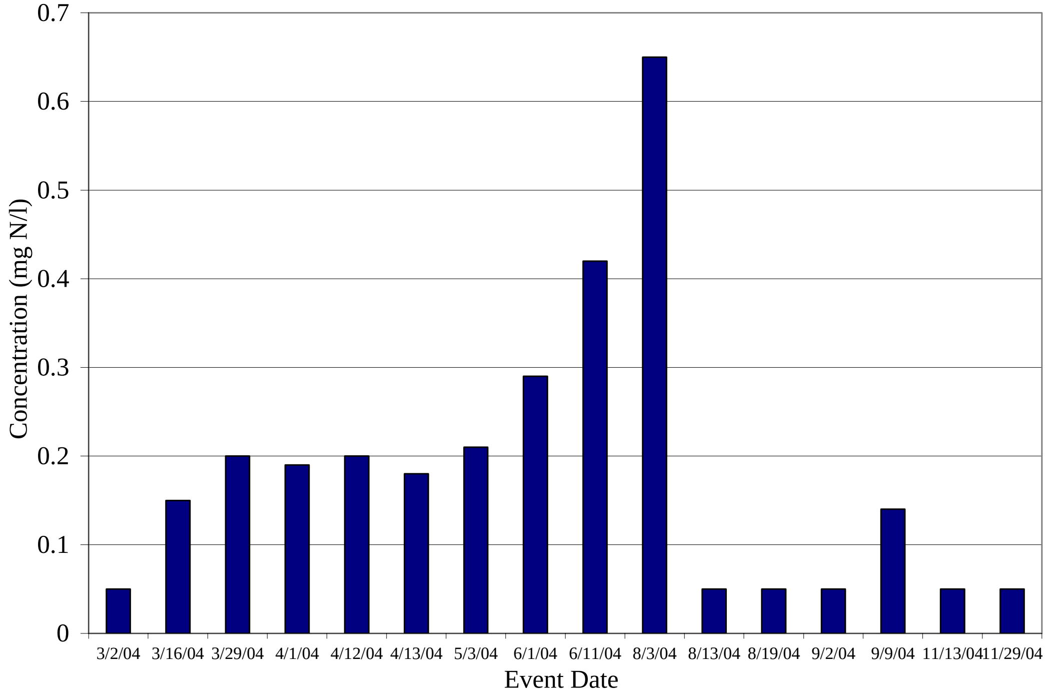

Figure 23. Total Kjeldahl nitrogen concentrations for PICP exfiltrate collected from the Swansboro site for event samples collected between March 2004 and November 2004.

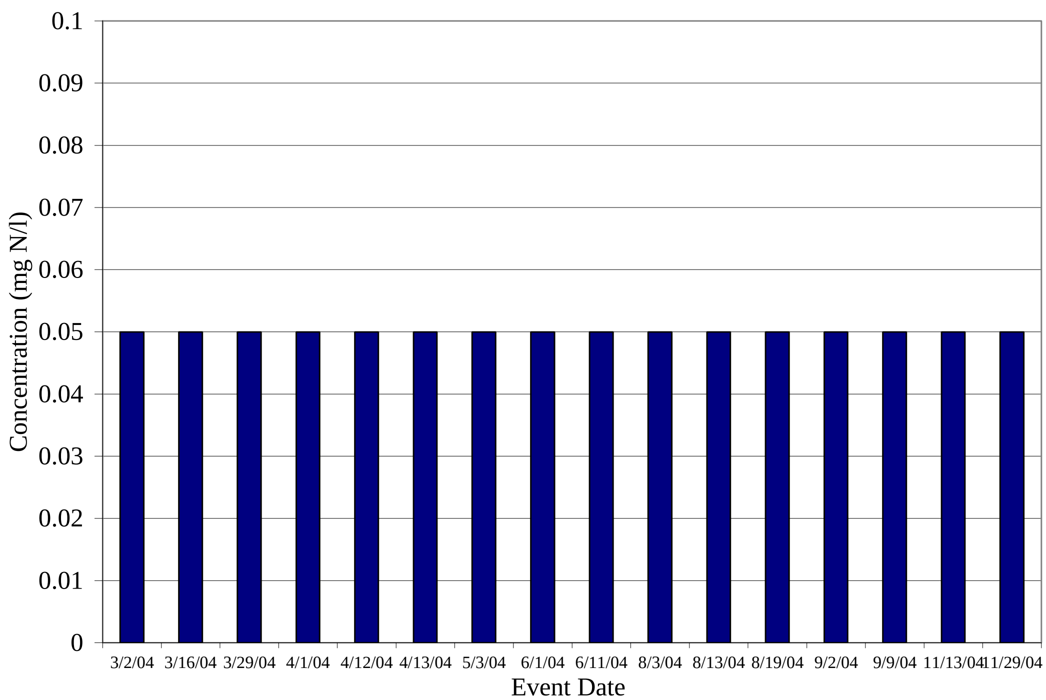

Figure 24. Ammonia in water concentrations for PICP exfiltrate collected from the Swansboro site for event samples collected between March 2004 and November 2004.

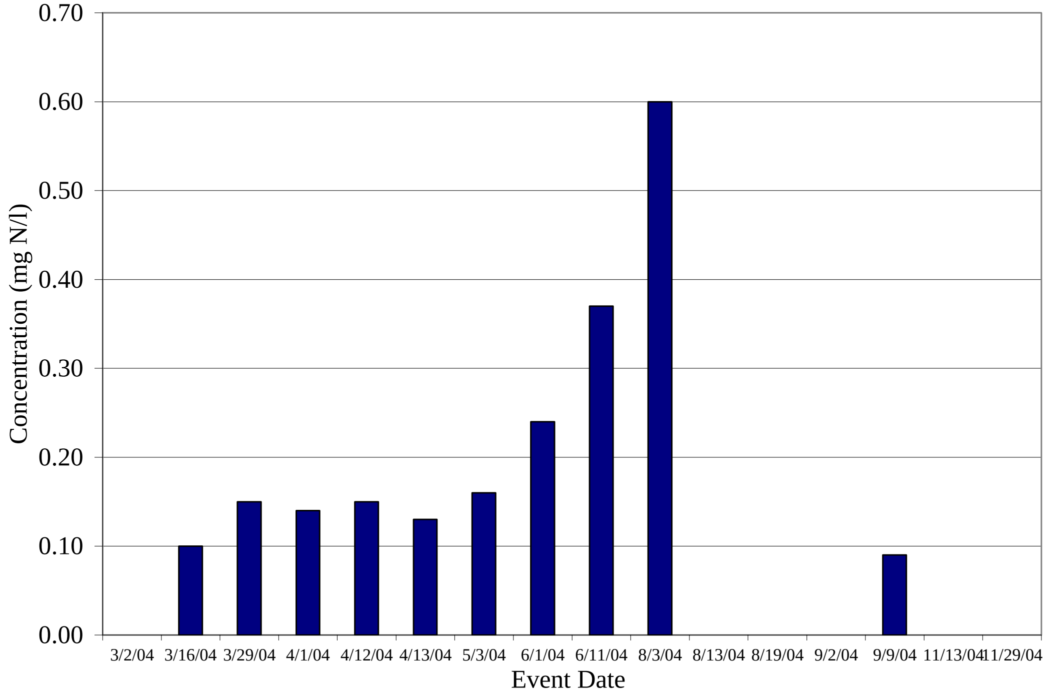

Figure 25. Organic nitrogen concentrations for PICP exfiltrate collected from the Swansboro site for event samples collected between March 2004 and November 2004.

Table 22. Phosphorus PICP exfiltrate species concentrations from the Swansboro site for event samples collected between March 2004 and November 2004.

Event Date:	**TP (mg P/l)**	**PO_4 (mg P/l)**	**BP (mg P/l)**
3/2/04	0.140	0.005	0.135
3/16/04	0.020	0.005	0.015
3/29/04	0.005	0.005	0.000
4/1/04	0.020	0.005	0.015
4/12/04	0.020	0.005	0.015
4/13/04	0.005	0.005	0.000
5/3/04	0.005	0.005	0.000
6/1/04	0.050	0.005	0.045
6/11/04	0.050	0.005	0.045
8/3/04	0.115	0.023	0.093
8/13/04	0.080	0.050	0.030
8/19/04	0.060	0.040	0.020
9/2/04	0.070	0.070	0.000
9/9/04	0.080	0.050	0.030
11/13/04	0.080	0.050	0.030
11/29/04	0.110	0.080	0.030
Average:	0.057	0.025	0.031

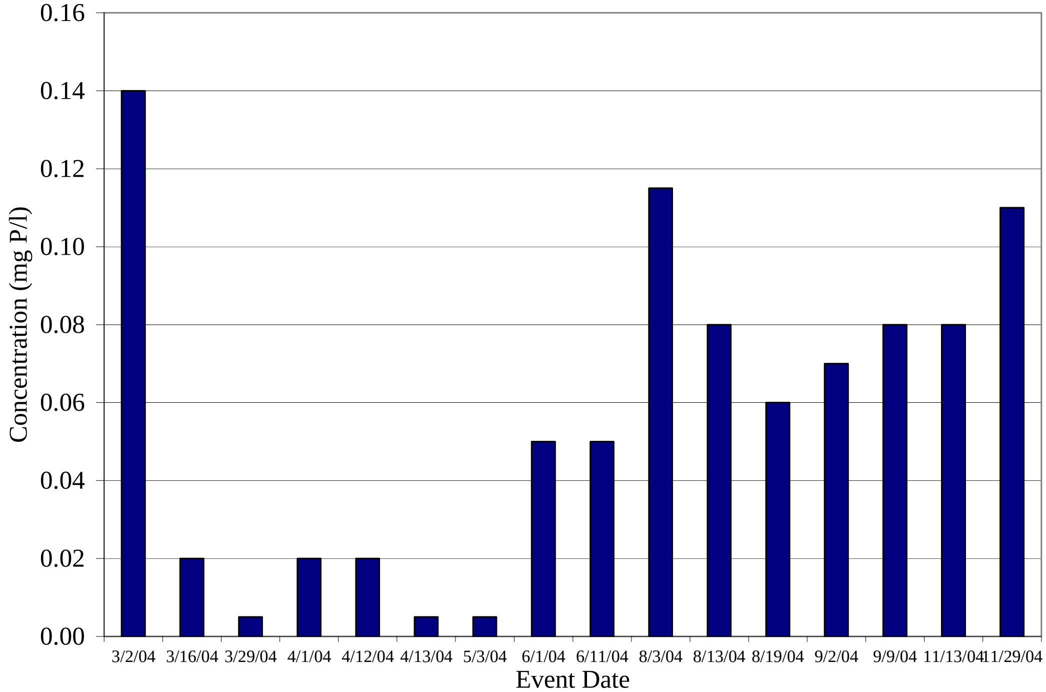

Figure 26. Total phosphorus concentrations for PICP exfiltrate collected from the Swansboro site for event samples collected between March 2004 and November 2004.

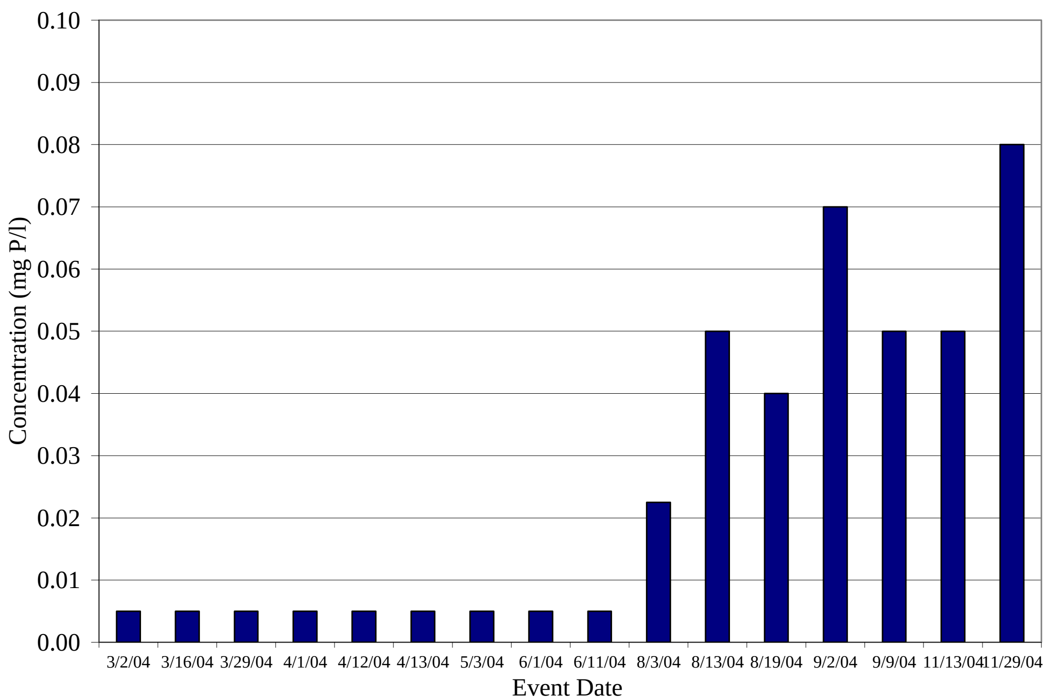

Figure 27. Orthophosphate concentrations for PICP exfiltrate collected from the Swansboro site for event samples collected between March 2004 and November 2004.

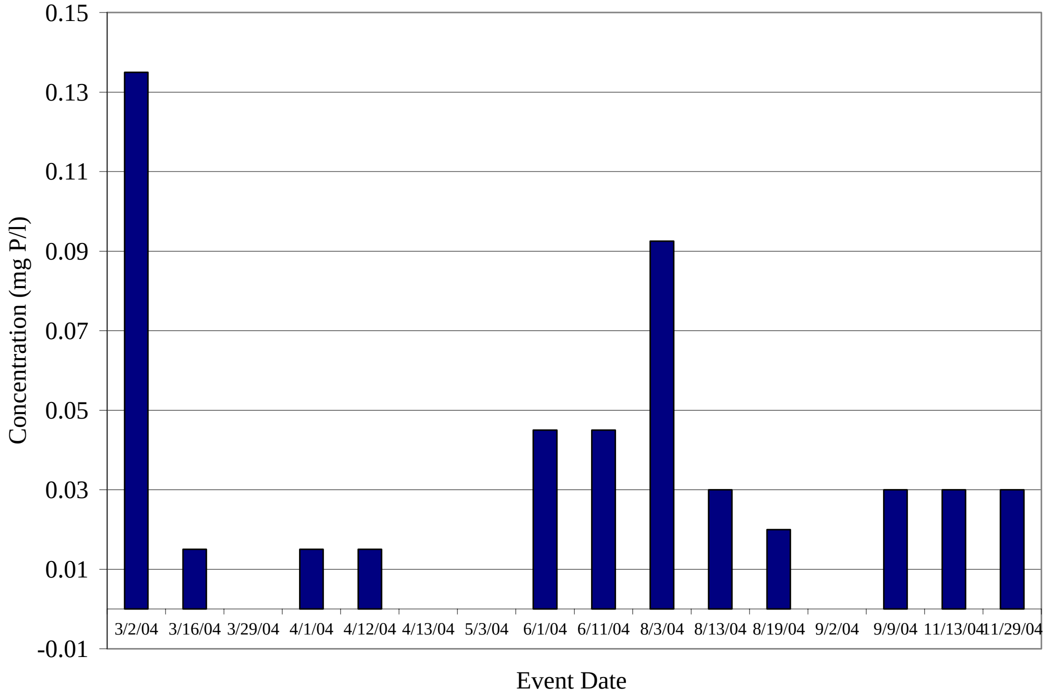

Figure 28. Bound phosphorus concentrations for PICP exfiltrate collected from the Swansboro site for event samples collected between March 2004 and November 2004.

APPENDIX G

STATISTICAL RESULTS FROM WATER QUALITY DATA

Goldsboro

Table 1. Skewness for distributions and test for significance for TN concentrations from the Goldsboro site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Exfiltrate	0.22	-0.47
Runoff	1.92	-0.05
p-value		0.0511

Table 2. Skewness for distributions and test for significance for NO_{2+3}-N concentrations from the Goldsboro site.

	Skewness
Sample	**Normal**
Exfiltrate	0.32
Runoff	0.81
p-value	0.1668

Table 3. Skewness for distributions and test for significance for TKN concentrations from the Goldsboro site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Exfiltrate	1.74	-0.04
Runoff	2.68	0.80
p-value		0.0075

Table 4. Skewness for distributions and test for significance for NH_4-N concentrations from the Goldsboro site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Exfiltrate	1.37	N/A
Runoff	1.02	0.12
p-value		0.0003

Table 5. Skewness for distributions and test for significance for ON concentrations from the Goldsboro site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Exfiltrate	2.20	1.88
Runoff	0.25	-1.15

Table 6. Summary table from SAS for the Goldsboro site ON concentrations test for significance.

Test		-Statistic-	-----p Value------	
Student's t	t	1.598306	Pr > |t|	0.1709
Sign	M	1	Pr >= |M|	0.6875
Signed Rank	S	6.5	Pr >= |S|	0.2188

Table 7. Skewness for distributions and test for significance for TP concentrations from the Goldsboro site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Exfiltrate	2.28	0.19
Runoff	2.92	0.49
p-value		0.0017

Table 8. Skewness for distributions and test for significance for PO_4 concentrations from the Goldsboro site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Exfiltrate	1.37	0.10
Runoff	1.32	1.05

Table 9. Summary table from SAS for the Goldsboro site PO_4 concentrations test for significance.

Test		-Statistic-	-----p Value------	
Student's t	t	1.595411	Pr > |t|	0.1329
Sign	M	4.5	Pr >= |M|	0.0352
Signed Rank	S	43	Pr >= |S|	0.0125

Table 10. Skewness for distributions and test for significance for BP concentrations from the Goldsboro site.

	Skewness
Sample	**Normal**
Exfiltrate	0.28
Runoff	0.35
p-value	0.27522

Table 11. Skewness for distributions and test for significance for TSS concentrations from the Goldsboro site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Exfiltrate	3.03	-2.16
Runoff	2.26	1.04

Table 12. Summary table from SAS for the Goldsboro site TSS concentrations test for significance.

Test	-Statistic-		-----p Value------	
Student's t	t	1.59422	Pr > |t|	0.1369
Sign	M	1.5	Pr >= |M|	0.5811
Signed Rank	S	19.5	Pr >= |S|	0.1909

Table 13. Skewness for distributions and test for significance for Cu concentrations from the Goldsboro site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Exfiltrate	2.83	2.83
Runoff	1.30	0.25

Table 14. Summary table from SAS for the Goldsboro site Cu concentrations test for significance.

Test	-Statistic-		-----p Value------	
Student's t	t	2.008627	Pr > |t|	0.0845
Sign	M	2	Pr >= |M|	0.2188
Signed Rank	S	8.5	Pr >= |S|	0.0938

Table 15. Skewness for distributions and test for significance for Zn concentrations from the Goldsboro site.

	Skewness
Sample	Normal
Exfiltrate	0.03
Runoff	0.73
p-value	0.0001

Cary

Table 16. Skewness for distributions and test for significance for TN concentrations from the Cary site.

	Skewness	
Sample	Normal	Log-Normal
Rainfall	2.33	0.79
Exfiltrate	1.83	-0.05
p-value		0.4036

Table 17. Skewness for distributions and test for significance for NO_{2+3}-N concentrations from the Cary site.

	Skewness	
Sample	Normal	Log-Normal
Rainfall	3.78	1.64
Exfiltrate	2.15	-0.55

Table 18. Summary table from SAS for the Cary site NO_{2+3}-N concentrations test for significance.

Test	-Statistic-		-----p Value------	
Student's t	t	2.207912	Pr > |t|	0.0444
Sign	M	2.5	Pr >= |M|	0.3018
Signed Rank	S	37	Pr >= |S|	0.0353

Table 19. Skewness for distributions and test for significance for TKN concentrations from the Cary site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Rainfall	2.09	0.21
Exfiltrate	0.66	0.05
p-value		0.5107

Table 20. Skewness for distributions and test for significance for NH_4-N concentrations from the Cary site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Rainfall	1.95	0.54
Exfiltrate	3.87	3.87

Table 21. Summary table from SAS for the Cary site NH_4-N concentrations test for significance.

Test	-Statistic-		-----p Value------	
Student's t	t	-2.37119	Pr > |t|	0.0326
Sign	M	-6	Pr >= |M|	0.0005
Signed Rank	S	-39	Pr >= |S|	0.0005

Table 22. Skewness for distributions and test for significance for ON concentrations from the Cary site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Rainfall	2.00	-0.19
Exfiltrate	0.64	0.02
p-value		0.6673

Table 23. Skewness for distributions and test for significance for TP concentrations from the Cary site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Rainfall	1.25	-0.56
Exfiltrate	3.78	2.48

Table 24. Summary table from SAS for the Cary site TP concentrations test for significance.

Test	-Statistic-		-----p Value------	
Student's t	t	0.824378	Pr > |t|	0.4235
Sign	M	3.5	Pr >= |M|	0.1185
Signed Rank	S	20	Pr >= |S|	0.2769

Table 25. Skewness for distributions and test for significance for PO_4 concentrations from the Cary site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Rainfall	3.35	0.98
Exfiltrate	3.77	2.16

Table 26. Summary table from SAS for the Cary site PO_4 concentrations test for significance.

Test	-Statistic-		-----p Value------	
Student's t	t	1.595411	Pr > |t|	0.1329
Sign	M	4.5	Pr >= |M|	0.0352
Signed Rank	S	43	Pr >= |S|	0.0125

Table 27. Skewness for distributions and test for significance for BP concentrations from the Cary site.

	Skewness	
Sample	**Normal**	**Log-Normal**
Rainfall	1.54	-0.78
Exfiltrate	1.00	-0.05
p-value		0.0142

APPENDIX H

MODELING STUDY OF PERCENT EVENT DEPTHS AND CORRESPONDING DETENTION POND SIZES FOR CITIES IN NORTH CAROLINA

ABSTRACT

Typically, detention ponds have been sized to capture runoff from the first flush event. Some states have looked at sizing them to capture the 90% storm. Rainfall event depths were determined so that stormwater structures were sized to capture that percent runoff, on 10% increments, plus 85%, for cities across North Carolina. The first flush event was found to be approximately between the 78 and 88% event. Using a spreadsheet model, detention ponds sizes were then determined for each city based on 90% rainfall event depth, watershed size, watershed land use, and soil type. Finally, using the same spreadsheet model, peak discharges from 2-yr 24 hr events routed through ponds designed for the 90% event were determined not to exceed pre-development peak runoff rates from the 2-yr 24-hr event.

INTRODUCTION

In North Carolina, stormwater structures are typically sized to capture runoff from the first 2.5 cm (1 in) – 3.8 cm (1.5 in) of rainfall. This depth is commonly known as the “first flush” and typically transports the majority of pollution in stormwater (Gupta and Saul, 1996). Stormwater structures are designed to allow excess water to bypass the treatment systems. However, some states, such as New York, have used the 90% storm for sizing detention ponds (NYSDEC, 2003). The 90% Storm is a term referring to the stormwater structure sizing event depth needed to capture 90% of all runoff. Some in the stormwater community believe the 90% storm to be nearly equal to 2.5 cm (1.0 in) in the Piedmont of NC.

This study determined the 90% storm for cities across North Carolina and then modeled the rainfall to determine the size detention ponds. Finally the 2-yr, 24 h, storm was modeled to determine whether post-development outflow from the detention pond would exceed pre-development peak flows.

NINETY PERCENT EVENT

First, event totals needed to be determined. Initially 30 years (1974 – 2003) of daily rainfall data were collected for the cities in Table 1 and the weather station IDs. Each day’s rainfall was assumed to be one isolated event. Event totals were then isolated and ranked. The total rainfall over 30 years was summed and then 10% of the total rainfall was calculated. The theoretical capture depth was subtracted from each storm and the differences summed. If the capture depth was greater than the event depth, then the difference was set to 0. The capture depth was manipulated until the summed differences, equaled 10% of the total rainfall; thus if only 10% of all rainfall volume exceeded the capture volume, then 90% would have been captured. The capture depth was then determined to be the 90% event. It should be noted that, in the end, events on a 10% interval, were determined from 10 – 80 along with 85 as well. From here on, it should be assumed that the full range of percent events are identified whenever the “90% event” term is used, since it was the benchmark of this study.

It was determined later that the assumption that each day was an individual rainfall event was inaccurate. A rainfall event was defined as lasting from the beginning of rainfall until at least six hrs elapses without rainfall. Therefore, rainfall events frequently ended a day or two later than they began. As a result, it was determined hourly rainfall data would need to be used to accurately determine the 90% event. Therefore, 30 years of hourly rainfall was collected and event totals were isolated. 90% events were determined methods as stated previously.

Table 1. Municipalities, COOPIDs, and site names for data files used in this study.

City	COOPID	Site Name
Asheville	310300	Asheville Regional Airport
Brevard	311055	Brevard
	316805	Pisgah National Forest
Charlotte	311690	Charlotte Douglas International Airport
Elizabeth City	312719	Elizabeth City
Fayetteville	313017	Fayetteville Pwc
Greensboro	313630	Greensboro Piedmont Triad International Airport
Greenville	313638	Greenville
Raleigh	317069	Raleigh Durham International Airport
Wilmington	319457	Wilmington International Airport

Only daily data was available for Brevard, which had significant gaps. However, when the data was requested from the North Carolina State Climate Office, supplementary daily rainfall data from the Pisgah Forest weather station was sent as well. These two data sets were combined to form a very complete data set, missing only one day in the entire 30-year period. Gaps in the Brevard data were replaced with Pisgah Forest data.

The quantity and quality of hourly data was generally good. However, Elizabeth City, Fayetteville, and Greenville had significant gaps, often missing entire months. To utilize the data, using the principle that rainfall occurs in a yearly cycle, years missing more than one contiguous month were eliminated from the data set used to determine % events: Elizabeth City (6): ’75, ’84, ’90, ’91, ’92, ’99; Fayetteville (4): ’79, ’84, ’92, ’00; Greenville (15): ’74-’77, ’79, ’80-’84, ’95, ’96, ’98, ’99, ’01. Table 2 lists hurricanes and their respective occurrence year for eliminated years.

Table 2. Major hurricanes occurring during years of hourly data that were either missing or removed.

Year	Hurricanes
1984	Diana
1985	Gloria
1986	Charley
1996	Arthur, Bertha, Fran
1998	Bonnie, Dennis
1999	Floyd
2003	Isabel

To remedy the substantial gaps in the Greenville and Elizabeth City data, a daily-to-hourly conversion factor was determined for Wilmington, since daily data was consistent. Daily data, instead of hourly data, was then used to determine the 90% event for Greenville and Elizabeth City and then scaled up using the daily-to-hourly conversion factor. The conversion factor ranged from 1.10 for the 20% event to 1.19 for the 90% event.

After reviewing preliminary results, it was determined that the 85% storm was of interest for possibly usage as a detention pond sizing benchmark. Therefore, Table 3 shows both 90% and 85% events from each city. Percent event depths for all cities and charts plotting the full range of these depths can be found in Appendix K.

Table 3. Calculated 90% and 85% rainfall depths for cities across NC.

	90% storm		85% Storm	
City	**(cm)**	**(in.)**	**(cm)**	**(in.)**
Asheville	3.26	1.28	2.57	1.01
Brevard	3.94	1.55	3.25	1.28
Charlotte	4.07	1.60	2.69	1.06
Elizabeth City	4.05	1.59	3.12	1.23
Fayetteville	3.93	1.55	3.15	1.24
Greensboro	3.96	1.56	3.13	1.23
Greenville	4.71	1.85	3.59	1.41
Raleigh	3.65	1.44	2.94	1.16
Wilmington	5.69	2.24	4.37	1.72

DETENTION POND SIZING

Once event depths were determined, only the 90% event depths were used to design detention ponds to capture all of runoff for each city, except for Brevard, and to determine the area needed for these ponds. This part of the study largely built off a previous study by Williams et al (2004). The detention ponds were all sized with a b-value of 1.2, which corresponds to the angle of the sides of the pond. A minimum value of one (1) corresponds to vertical sides and as values increase, the sides become more horizontal. Ponds were designed so that all runoff was stored in 38 cm (15 in) of rise in the water surface of the pond. Ponds were sized for watersheds of 4, 20, and 61 ha (10, 50, and 150 ac) with A, B, C, and D type soils. Imperviousnesses of modeled watersheds were 12, 25, 38, 65, and 85% (NRCS, 1986); corresponding to land uses for 2, 1/2, 1/4, and 1/8 acre residential and commercial/industrial, respectively (Malcom, 1989). After a few model runs it became apparent that the pond sizes for differing watersheds were directly related to the watershed areas. Therefore, 61 ha (150 ac) watersheds were modeled and then scaled down to the 20 ha (50 ac) and 4 ha (10 ac) watersheds. In addition to modeling the 90% daily and hourly events, ponds were sized to capture runoff and runoff from rainfall events of 2.5 cm (1 in);3.8 cm (1.5 in) for cities with Type III SCS rainfall distributions (NRCS, 1986).

Models were run on 1-minute intervals. Rainfall input into the model was determined by scaling a Type II (Asheville, Brevard, Charlotte, Greensboro/Winston-Salem and Raleigh/Durham) or Type III (Elizabeth City, Fayetteville, Greenville and Wilmington) SCS rainfall event (NRCS, 1986) to the depth storm to be modeled. Curve numbers were derived from the percent imperviousnesses and soil group to determine the amount of runoff (NRCS, 1986). Runoff was determined by using the SCS Small Watershed Method with Equations 1 and 2 (NRCS 1986).

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad \text{(Equation 1)}$$

$$S = \frac{1000}{CN} - 10 \qquad \text{(Equation 2)}$$

P is equal to the total precipitation, while S is the soil storage volume, and Q is the runoff for that minute interval from the watershed (NRCS, 1986). Runoff was converted to cubic meters by multiplying the runoff depth by the watershed area. This volume was added to the storage volume.

The initial storage volume (SV_0) was calculated by Equation 3 (Malcom, 1989).

$$SV_0 = Ks*(Stage)^b \qquad \text{(Equation 3)}$$

Then runoff volumes were added to the previous minute's volume. The initial stage was set for all ponds at 1.4 m (4.8 ft) so that the average depth of the pond would be 1.2 m (4.0 ft). Stages were calculated from corresponding storage volumes from Equation 4 (Malcom, 1989).

$$Stage = (\frac{S * V_N}{K_S})^{1/b} \qquad \text{(Equation 4)}$$

To determine required area for ponds, the maximum stage was determined for each storm, then that stage was used to calculate surface area (Equation 5) (Malcom, 1989), and then converted from square meters to hectares. Resulting detention pond sizes can be found in Appendix L.

$$A = K_S * b * Stage^{b-1} \qquad \text{(Equation 5)}$$

PEAK OUTFLOW MODEL

After determining detention pond sizes for various land covers, soil conditions, and sizes for different cities, the question was posed as to whether ponds designed to capture the 90% event would mitigate flow from the 2-yr event. Watersheds converted from forested and agricultural land uses to different residential and commercial uses in Charlotte and Wilmington were modeled to determine whether the 2-yr 24 h event, routed through a 90% event sized detention pond, had outflows that exceeded pre-developed peak runoff. Pre-development flow conditions are listed below for Charlotte and Wilmington in Tables 17 and 18, respectively.

Table 4. Pre-development peak runoff and total runoff volumes form 2-year 24-hour rainfall event for simulated watersheds in Charlotte, NC.

Charlotte	2 year Storm		8.9 cm	(3.5 in.)
61 ha (150 ac)	SCS Type II Storm			
	Forested		Ag Soil	
Soil	A	B	A	B
CN	36	60	65	75
Peak Flow [m^3/s]	0	4.0	5.7	9.1
[ft^3/s]	0	140	200	320
Runoff Volume [m^3]	0	8000	12000	20000
[ft^3]	0	290000	410000	710000

Table 5. Pre-development peak runoff and total runoff volumes form 2-year 24-hour rainfall event for simulated watersheds in Wilmington, NC.

Wilmington	2 year Storm		11 cm	(4.5 in.)
61 ha (150 ac)	SCS Type III Storm			
	Forested		Ag Soil	
Soil	A	B	A	B
CN	36	60	65	75
Peak Flow [m^3/s]	0	2.3	3.3	5.3
[ft^3/s]	0	81	115	188
Runoff Volume [m^3]	740	16000	20000	31000
[ft^3]	26000	560000	720000	1100000

One riser-barrel structure was used for each pond of varying sizes with drain holes so that the 90% runoff volume would still be captured and anything over that would pass through the riser-barrel. The 90% event was captured within the first 38 cm (15 in) of rise in the stage, where the top of the riser was set. Any additional runoff then flowed through the riser-barrel and out. The number and size of draw down holes were manipulated so that the first 38 cm (15 in) of storage drew down within 60 hrs to less than 0.03 m (0.1 ft) (3 cm (1.2 in)) of storage.

Post-development runoff results are listed in Appendix M. Forested A soils were the only pre-development scenarios when the 90% event pond did not mitigate for peak pre-development flows. Virtually no runoff occurred from the Forested A soil, therefore when the area is developed, runoff is produced, and a detention pond may be needed. Peak outflow rates are the peak runoff rates of post-development for these cases. Another note, is that the larger the pond is, the smaller the peak rise in stage will be for the ponds.

CONCLUSIONS

The 90% rainfall event depth was greater than the first flush depth (2.5 cm (1 in) or 3.8 cm (1.5 in)) for all cities listed. The current first flush depth typically lies between just below the 80% and just below the 90% rainfall events. Therefore, if the 90% rainfall event was used as a standard, stormwater structures would be larger than their current sizes. However, 90% storms were found, by modeling, to have peak outflows below pre-development peak runoff rates, except for pre-developed Forested A soils.

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APPENDIX I

WEIR BOX AND SUBMERSIBLE LIQUID LEVEL TRANSMITTER CALIBRATION

PROCEDURE

The WIKA Submersible Liquid Level Transmitter® (SLLT) was calibrated on May 10, 2004 in the Weaver laboratory space. A 19 liter (5 gallon) bucket was used to simulate controlled depths of water measured by a ruler. Current readings were recorded from the WIKA SLLT by an Omega Data Logger® (DL-AC). The data logger was programmed to take a reading every 60 seconds. The first reading was taken with no water in the bucket. Then the water level was raised every 60 seconds by 10 mm. The depth was plotted against the current so that a direct relationship could be used to convert current data collected at the site to water depth (Figure 1).

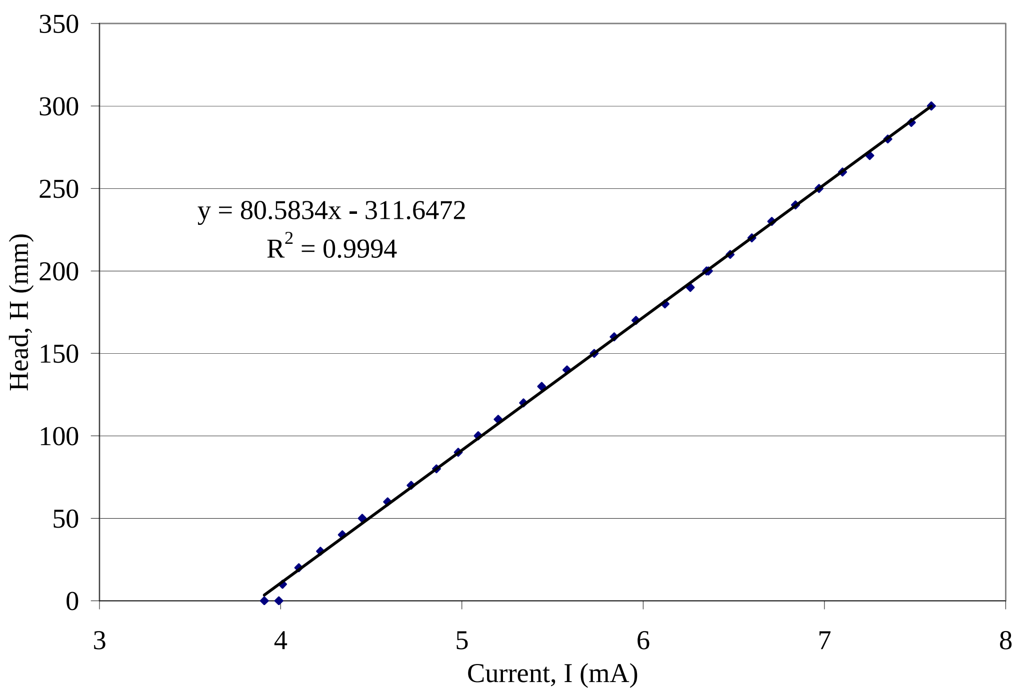

Figure 1. Depth versus current relationship data from calibration of a WIKA Submersible Liquid Level Transmitter® for use at the Cary site.

The resulting depth, or head (H, m), versus current (I, mA) relationship determined through calibration can be seen in Equation 1.

$$H = 0.080583 * I - 0.31165 \qquad \text{(Equation 1)}$$

WEIR BOX CALIBRATION

On June 4, 2004, Mark Blevins from the Agricultural Extension Department, and the author calibrated a weir box used at the Cary monitoring site. Figure 2 shows graphical results of the data. The dimensions of the weir box are below, Figure 3. The testing was conducted at the Air Quality Research Facility at Lake Wheeler Farms on North Carolina State University property. The site has a pressurized well pump that was used to supply water at different flow rates for the testing. Water was set at seven different unknown flow rates that were found to range from approximately 0.028 l/s (0.001 cfs) to 2.8 l/s (0.1 cfs). The water supply hose was hung in the weir box between the back wall and the baffle. A 19 l (5 gal) bucket was used to capture outflow from the weir box. To measure flow, time was recorded from when water started filling the bucket to when the bucket was filled and started over-flowing. This was repeated twice more for three time recordings. The depth of water was then measured from the bottom to the surface of the water on the backside of each side of the weir. Flows were determined for each time recording and then averaged for each flow setting. Depths were converted to head by subtracting the height of the weir, and then converted to meters. Flow (Q, l/s) was plotted as a function of head (H, m) and a function was derived. The calculated function is reported in Equation 2.

$$Q = 1014.7 * H^{2.3794} \qquad \text{(Equation 2)}$$

With this equation, depth data from the WIKA SLLT® and ISCO 6712® could be converted to flow data.

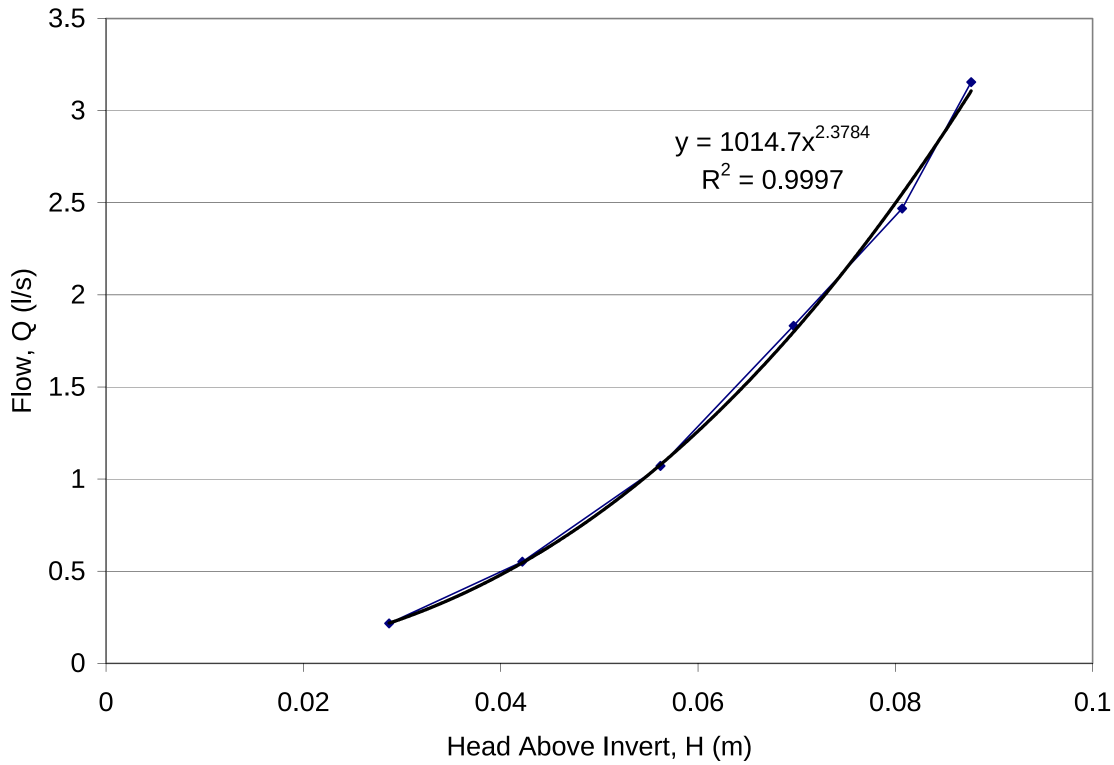

Figure 2. Head versus flow calibration for weir box used at the Cary site.

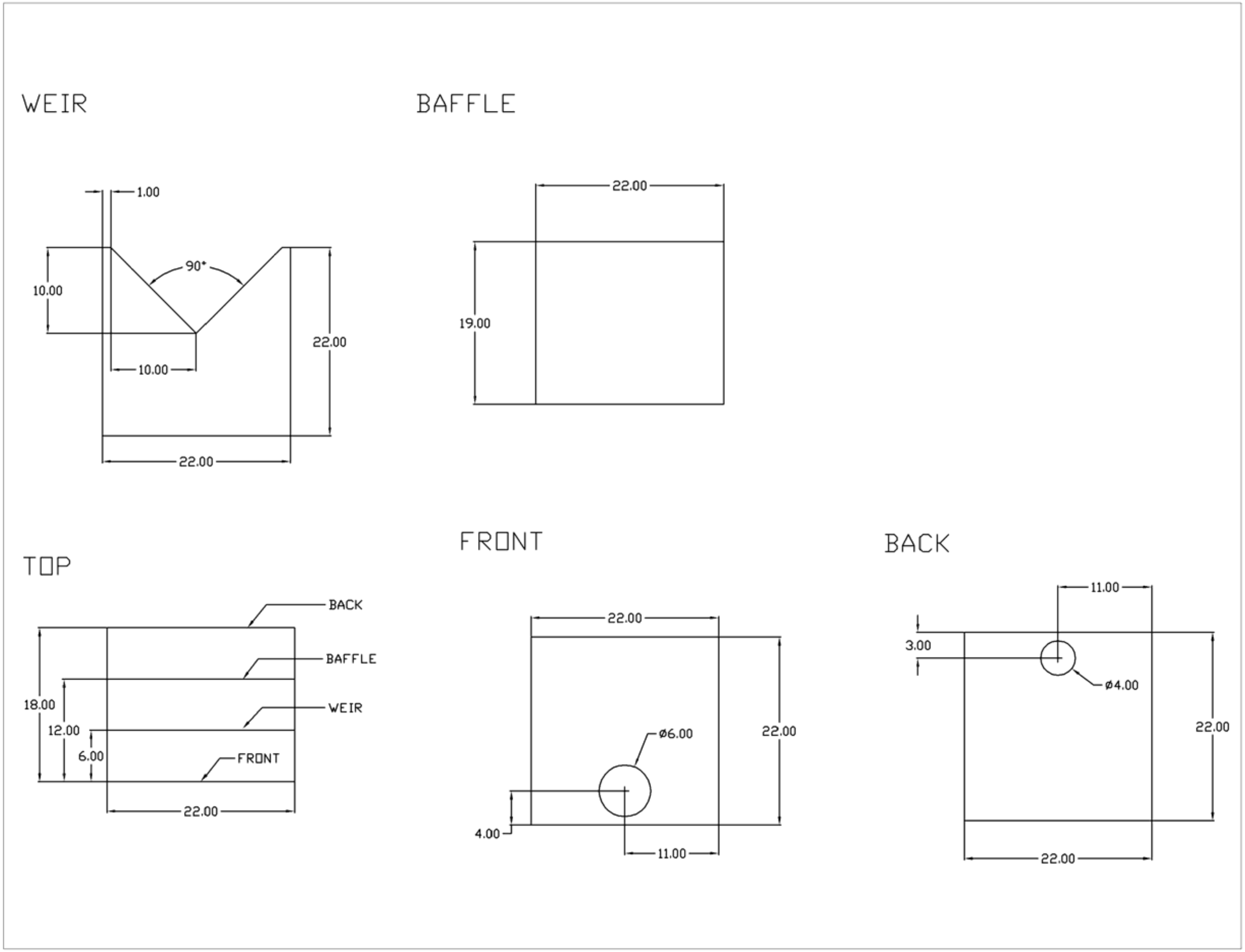

Figure 3. Weir box dimensions in inches from the Cary site.

APPENDIX J

Basic-X BX-24® Rain Gage Data Logger Program

```
Public Sub Main()

Dim PulseWidth As Integer
Dim DACcounter As Byte
Dim Ptovlt As Single
Dim CountRain As Integer
Dim Vlto As Integer
Dim Var As Integer
Dim Apex As Byte
Const Maxrain As Single = 12.00
Const Pini As Byte = 20
Const Pino As Byte = 13

Var = 0
Call PutPin(Pini, 3)

CountRain =0
Do

                Apex = GetPin(Pini)
                 Debug.Print CStr(Apex)&" Apex"
                If (Var =0) Then
                        If(0=Apex) Then
                                CountRain = CountRain + 1
                                Debug.Print CStr(CountRain)&" CountRain"
                                Var = 1
                        End If
                ElseIf(Apex=1) Then
                        Var = 0
                End If
                Ptovlt = 0.2 + CSng(CountRain)/1500.0
                Debug.Print CStr(Ptovlt)&" Ptovlt"
                Call PutDAC(Pino, Ptovlt, DACcounter)
Loop
End Sub
```

APPENDIX K

CALCULATED PERCENT EVENT DEPTHS

Table 1. Calculated percent event depths from 10% to 90% for municipalities across North Carolina.

	Percent Event Depth (cm)									
City	**90**	**85**	**80**	**70**	**60**	**50**	**40**	**30**	**20**	**10**
Asheville	3.26	2.57	2.10	1.50	1.08	0.78	0.55	0.36	0.21	0.09
Brevard	3.94	3.25	2.75	2.03	1.50	1.11	0.79	0.54	0.32	0.14
Charlotte	4.07	3.25	2.69	1.93	1.42	1.04	0.74	0.49	0.29	0.13
Elizabeth City	4.05	3.12	2.55	1.79	1.30	0.94	0.66	0.43	0.25	0.10
Fayetteville	3.93	3.15	2.61	1.89	1.39	1.01	0.71	0.47	0.27	0.13
Greensboro	3.96	3.13	2.58	1.85	1.37	1.00	0.71	0.47	0.28	0.12
Greenville	4.71	3.59	2.91	2.03	1.48	1.07	0.75	0.50	0.30	0.13
Raleigh	3.65	2.94	2.46	1.79	1.32	0.97	0.70	0.47	0.28	0.12
Wilmington	5.69	4.37	3.55	2.46	1.76	1.27	0.89	0.59	0.35	0.15

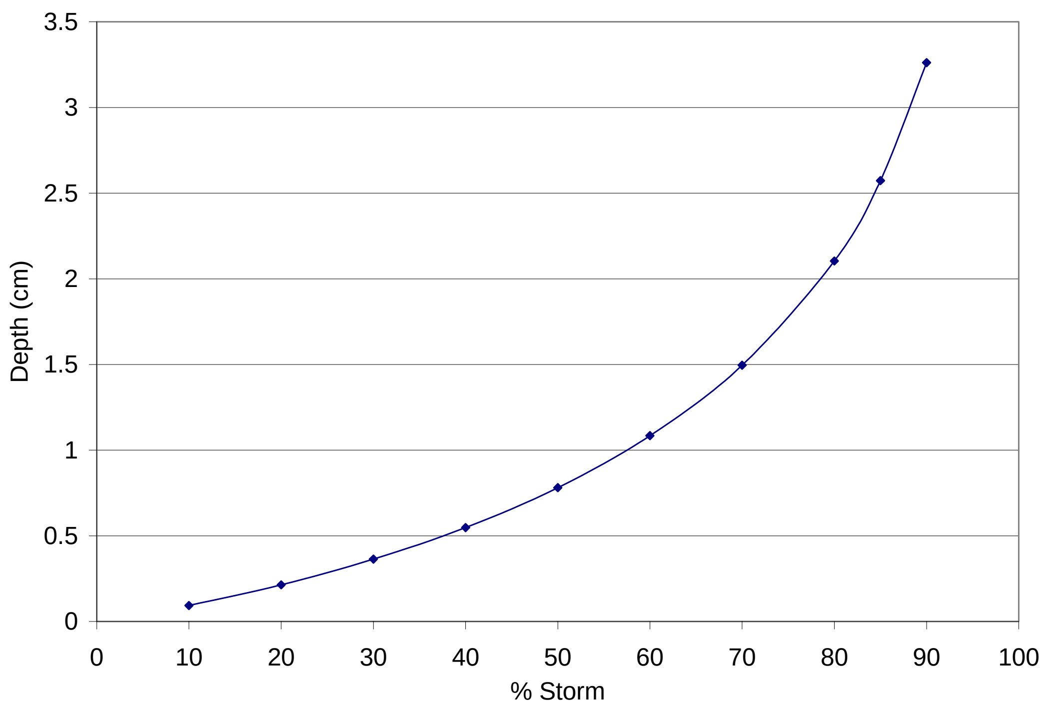

Figure 1. Calculated percent events for Asheville, NC.

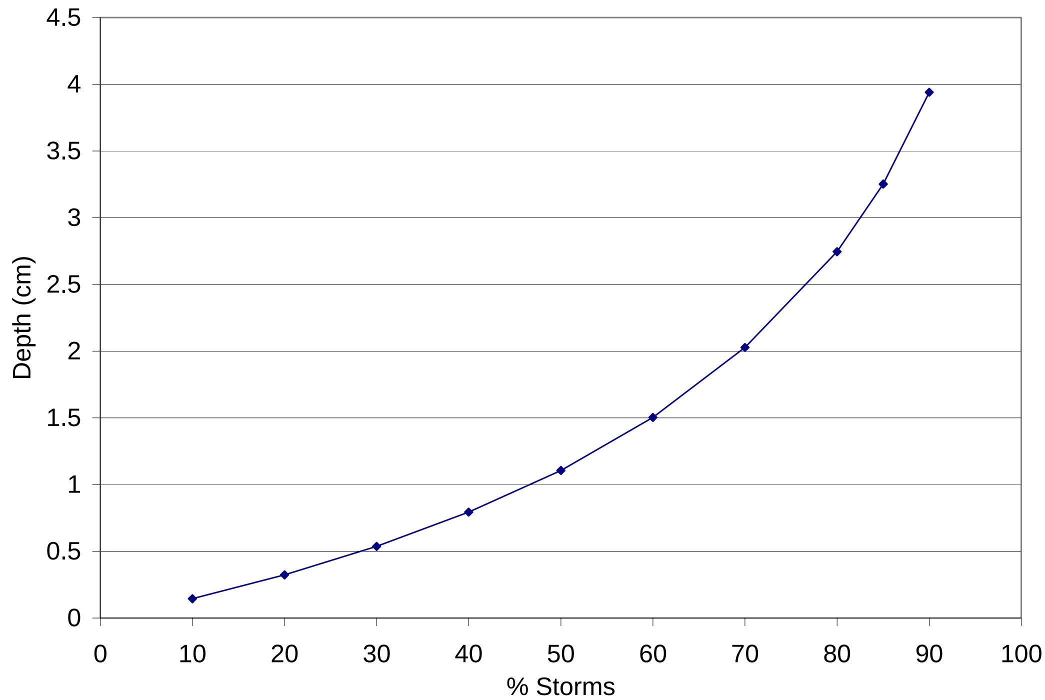

Figure 2. Calculated percent events for Brevard, NC.

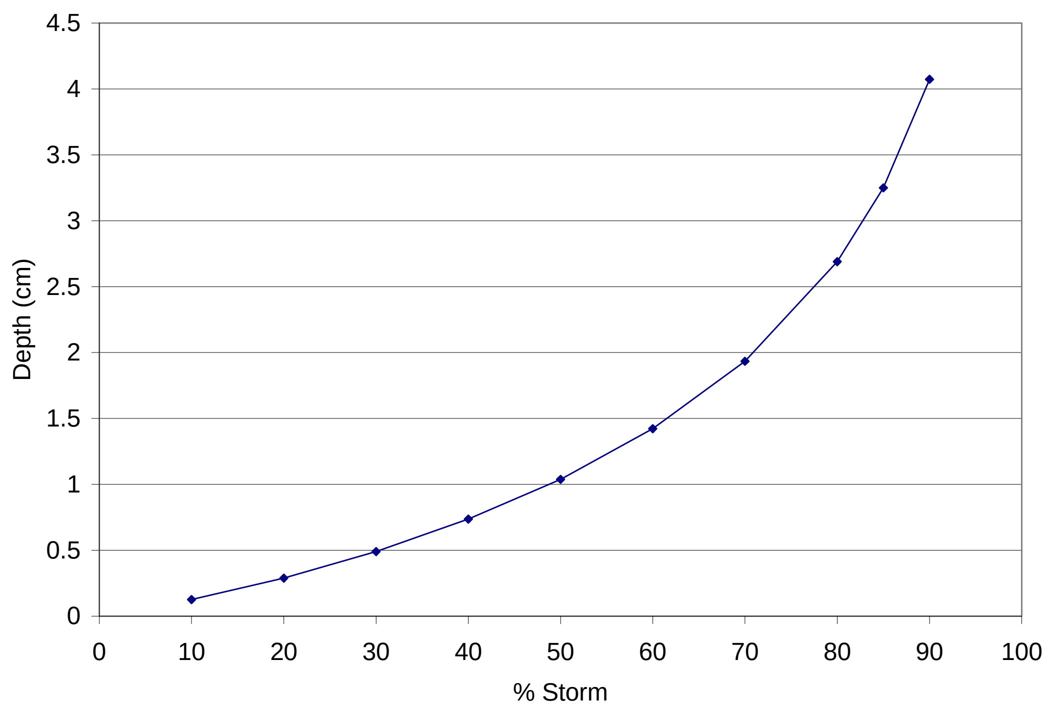

Figure 3. Calculated percent events for Charlotte, NC.

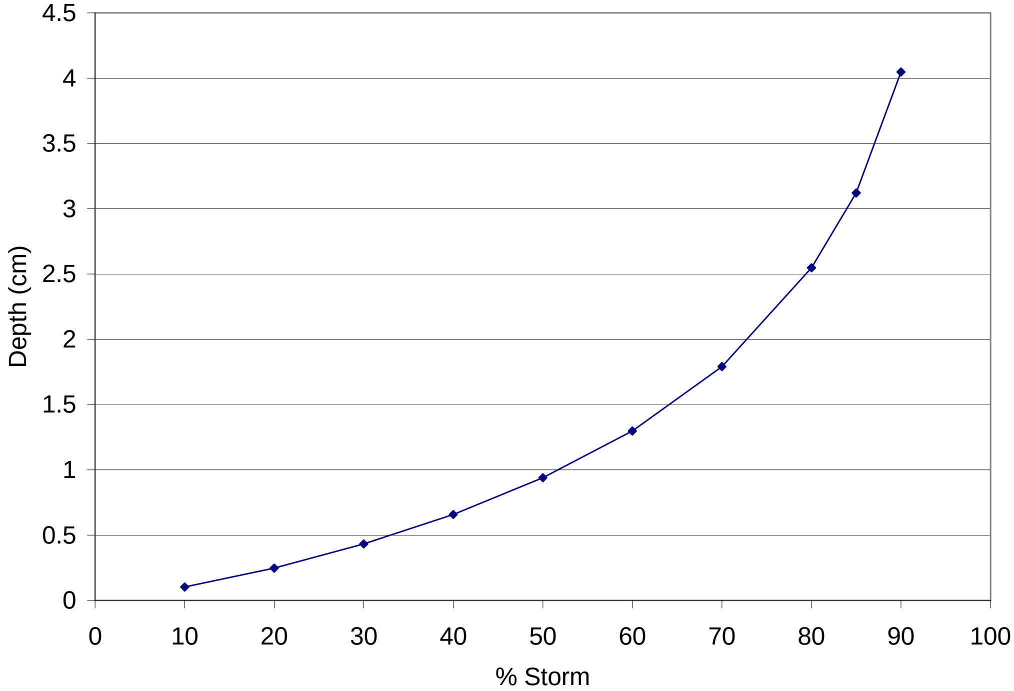

Figure 4. Calculated percent events for Elizabeth City, NC.

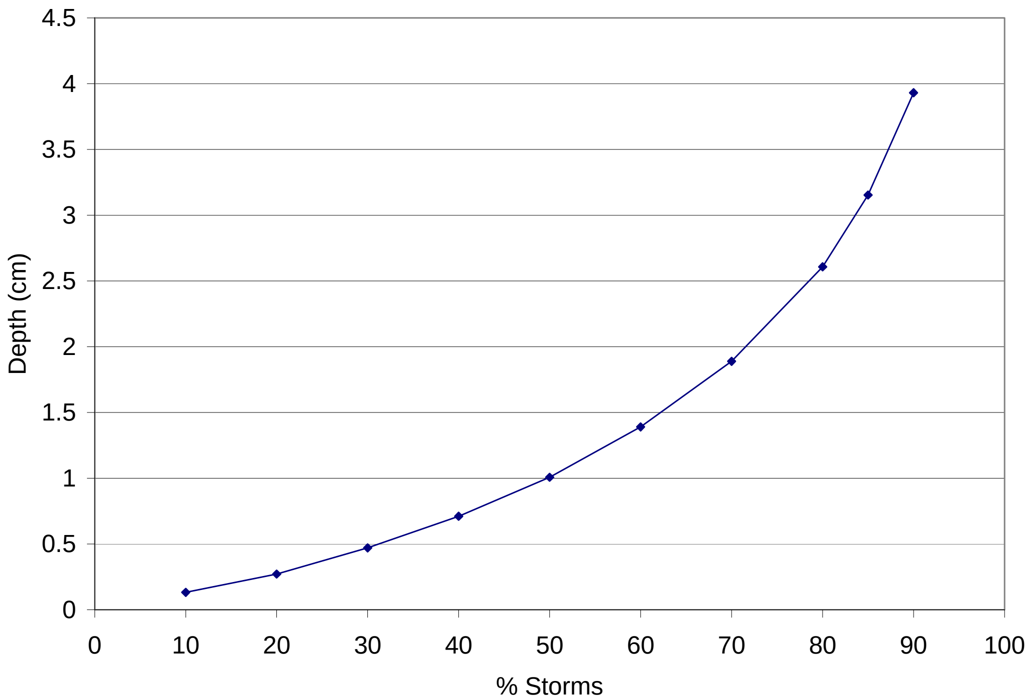

Figure 5. Calculated percent events for Fayetteville, NC.

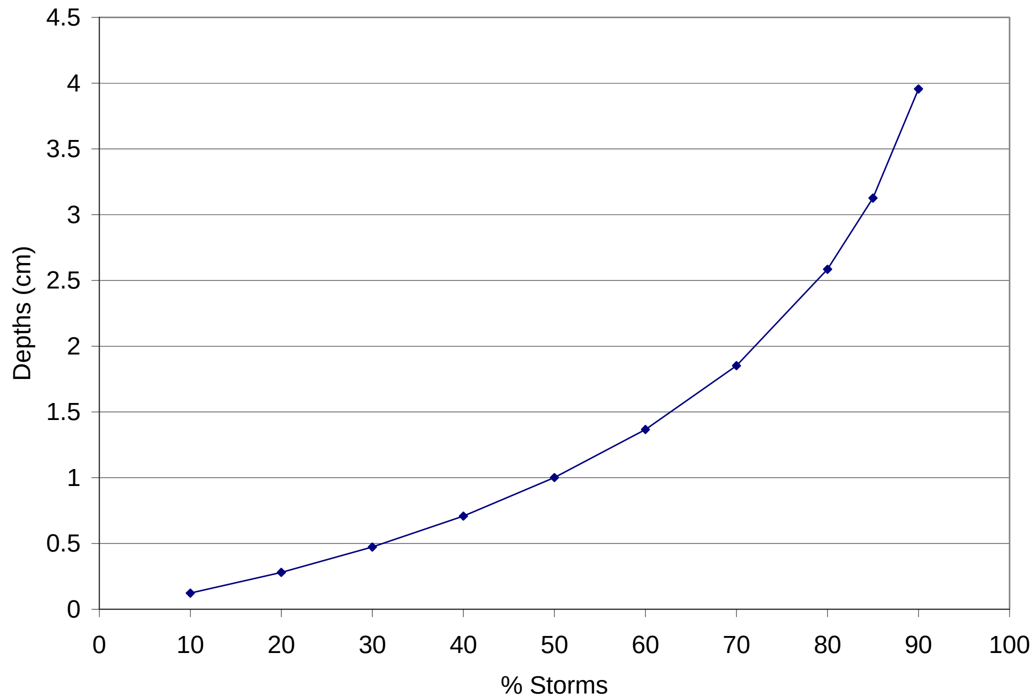

Figure 6. Calculated percent events for Greensboro, NC.

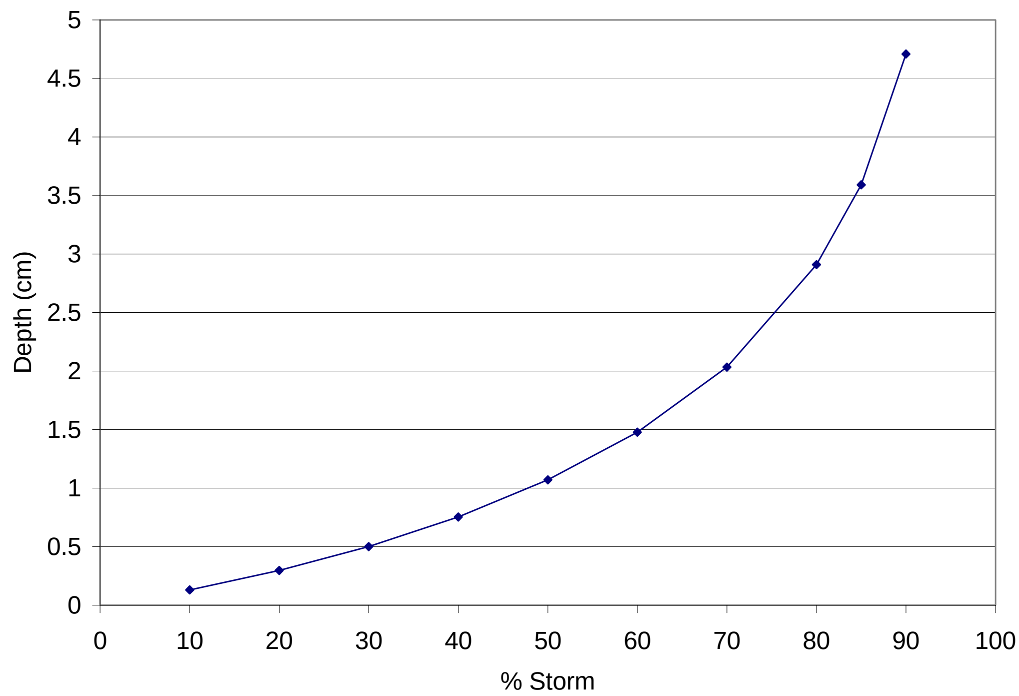

Figure 7. Calculated percent events for Greenville, NC.

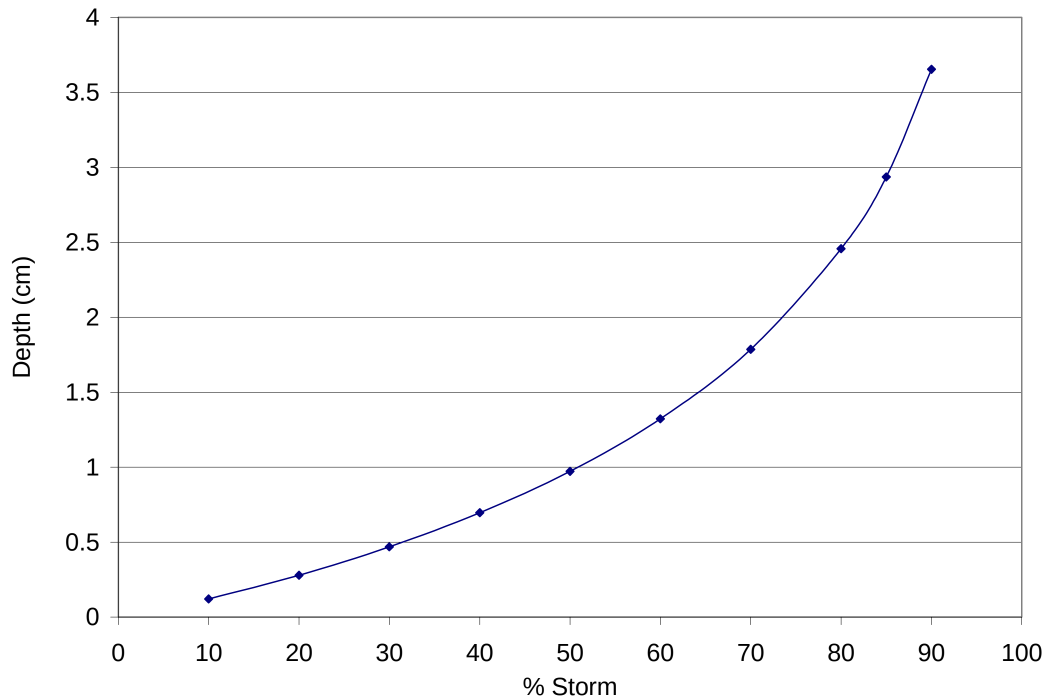

Figure 8. Calculated percent events for Raleigh, NC.

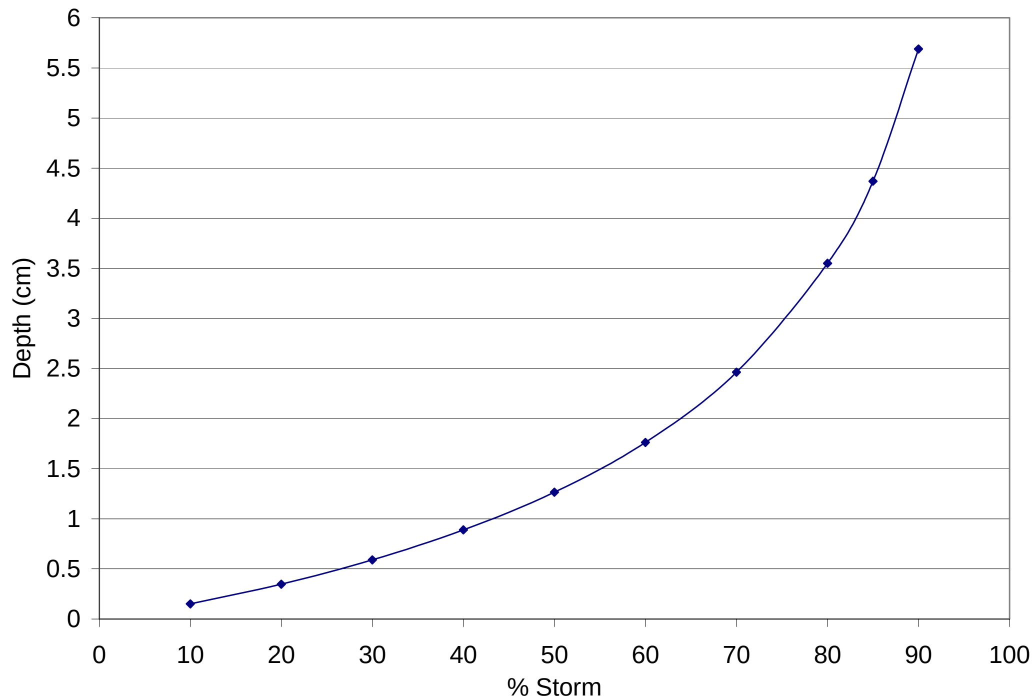

Figure 9. Calculated percent events for Wilmington, NC.

APPENDIX L

MODELED POND SIZES FOR CAPTURING THE 90% EVENT

Table 1. Pond sizes to capture the calculated 90% event for Asheville, NC, for various land uses.

Asheville						
Pond Areas (ha)		% Pervious				
Watershed Area (ha)	Soil Group	12	25	38	65	85
4	A	0.00	0.00	0.00	0.03	0.13
	B	0.00	0.01	0.03	0.09	0.17
	C	0.03	0.05	0.07	0.14	0.20
	D	0.06	0.09	0.11	0.17	0.22
20	A	0.00	0.00	0.00	0.17	0.64
	B	0.01	0.05	0.13	0.44	0.85
	C	0.17	0.25	0.35	0.70	1.01
	D	0.32	0.44	0.53	0.85	1.11
61	A	0.00	0.00	0.00	0.52	1.93
	B	0.03	0.16	0.39	1.31	2.54
	C	0.52	0.76	1.06	2.11	3.04
	D	0.95	0.97	1.59	2.54	3.33

Table 2. Pond sizes to capture the calculated 90% event for Brevard, NC, for various land uses.

Brevard						
Pond Areas (ha)		% Pervious				
Watershed Area (ha)	Soil Group	12	25	38	65	85
4	A	0.00	0.00	0.00	0.06	0.18
	B	0.01	0.03	0.05	0.13	0.23
	C	0.06	0.08	0.11	0.20	0.27
	D	0.10	0.13	0.16	0.23	0.29
20	A	0.00	0.00	0.02	0.31	0.91
	B	0.05	0.13	0.25	0.66	1.15
	C	0.31	0.42	0.56	0.99	1.34
	D	0.51	0.66	0.78	1.15	1.45
61	A	0.00	0.00	0.05	0.94	2.74
	B	0.16	0.40	0.76	1.98	3.46
	C	0.94	1.27	1.67	2.96	4.03
	D	1.53	1.98	2.33	3.46	4.34

Table 3. Pond sizes to capture the calculated 90% event for Charlotte, NC, for various land uses.

Charlotte						
Pond Areas (ha)		% Pervious				
Watershed Area (ha)	Soil Group	12	25	38	65	85
4	A	0.00	0.00	0.00	0.07	0.19
	B	0.01	0.03	0.06	0.14	0.24
	C	0.07	0.09	0.12	0.21	0.28
	D	0.11	0.14	0.17	0.24	0.30
20	A	0.00	0.00	0.02	0.34	0.97
	B	0.06	0.15	0.28	0.70	1.21
	C	0.34	0.46	0.60	1.04	1.40
	D	0.55	0.70	0.83	1.21	1.51
61	A	0.00	0.00	0.06	1.03	2.90
	B	0.19	0.45	0.84	2.11	3.63
	C	1.03	1.38	1.79	3.13	4.21
	D	1.64	2.11	2.48	3.63	4.54

Table 4. Pond sizes to capture the calculated 90% event for Elizabeth City, NC, for various land uses.

Elizabeth City (Original Data)						
Pond Areas (ha)		% Pervious				
Watershed Area (ha)	Soil Group	12	25	38	65	85
4	A	0.00	0.00	0.00	0.04	0.15
	B	0.00	0.02	0.03	0.10	0.19
	C	0.04	0.06	0.08	0.16	0.22
	D	0.08	0.10	0.12	0.19	0.24
20	A	0.00	0.00	0.00	0.22	0.73
	B	0.02	0.08	0.17	0.51	0.94
	C	0.22	0.31	0.42	0.80	1.12
	D	0.38	0.51	0.61	0.94	1.22
61	A	0.00	0.00	0.01	0.65	2.19
	B	0.06	0.23	0.51	1.52	2.83
	C	0.65	0.92	1.26	2.39	3.36
	D	1.14	1.52	1.83	2.83	3.66

Table 5. Pond sizes to capture the calculated 90% event for Fayetteville, NC, for various land uses.

Fayetteville						
Pond Areas (ha)		% Pervious				
Watershed Area (ha)	Soil Group	12	25	38	65	85
4	A	0.00	0.00	0.00	0.06	0.18
	B	0.01	0.03	0.05	0.13	0.23
	C	0.06	0.08	0.11	0.20	0.27
	D	0.10	0.13	0.16	0.23	0.29
20	A	0.00	0.00	0.02	0.32	0.91
	B	0.05	0.13	0.25	0.66	1.15
	C	0.32	0.42	0.56	0.99	1.34
	D	0.51	0.66	0.78	1.15	1.44
61	A	0.00	0.00	0.05	0.95	2.74
	B	0.16	0.40	0.76	1.98	3.45
	C	0.95	1.27	1.67	2.96	4.02
	D	1.53	1.98	2.33	3.45	4.33

Table 6. Pond sizes to capture the calculated 90% event for Greensboro, NC, for various land uses.

Greensboro						
Pond Areas (ha)		% Pervious				
Watershed Area (ha)	Soil Group	12	25	38	65	85
4	A	0.00	0.00	0.00	0.06	0.18
	B	0.01	0.03	0.05	0.13	0.23
	C	0.06	0.09	0.11	0.20	0.27
	D	0.10	0.13	0.16	0.23	0.29
20	A	0.00	0.00	0.02	0.32	0.92
	B	0.05	0.14	0.26	0.67	1.16
	C	0.32	0.43	0.56	1.00	1.35
	D	0.52	0.67	0.79	1.16	1.46
61	A	0.00	0.00	0.05	0.96	2.77
	B	0.16	0.41	0.77	2.01	3.49
	C	0.96	1.29	1.69	2.99	4.06
	D	1.55	2.01	2.36	3.49	4.38

Table 7. Pond sizes to capture the calculated 90% event for Greenville, NC, for various land uses.

Greenville (Original Data)						
Pond Areas (ha)		% Pervious				
Watershed Area (ha)	Soil Group	12	25	38	65	85
4	A	0.00	0.00	0.00	0.06	0.17
	B	0.01	0.02	0.05	0.13	0.22
	C	0.06	0.08	0.11	0.19	0.26
	D	0.10	0.13	0.15	0.22	0.28
20	A	0.00	0.00	0.01	0.29	0.87
	B	0.04	0.12	0.23	0.63	1.10
	C	0.29	0.40	0.53	0.94	1.29
	D	0.48	0.63	0.74	1.10	1.39
61	A	0.00	0.00	0.03	0.88	2.61
	B	0.13	0.36	0.70	1.88	3.31
	C	0.88	1.19	1.58	2.83	3.87
	D	1.44	1.88	2.22	3.31	4.18

Table 8. Pond sizes to capture the calculated 90% event for Raleigh, NC, for various land uses.

Raleigh						
Pond Areas (ha)		% Pervious				
Watershed Area (ha)	Soil Group	12	25	38	65	85
4	A	0.00	0.00	0.00	0.05	0.16
	B	0.01	0.02	0.04	0.11	0.21
	C	0.05	0.07	0.09	0.17	0.24
	D	0.09	0.11	0.13	0.21	0.26
20	A	0.00	0.00	0.01	0.25	0.80
	B	0.03	0.10	0.20	0.57	1.03
	C	0.25	0.35	0.47	0.87	1.21
	D	0.43	0.57	0.67	1.03	1.31
61	A	0.00	0.00	0.02	0.76	2.40
	B	0.09	0.29	0.60	1.70	3.08
	C	0.76	1.05	1.41	2.61	3.62
	D	1.28	1.70	2.02	3.08	3.93

Table 9. Pond sizes to capture the calculated 90% event for Wilmington, NC, for various land uses.

Wilmington						
Pond Areas (ha)		% Pervious				
Watershed Area (ha)	Soil Group	12	25	38	65	85
4	A	0.00	0.01	0.03	0.16	0.34
	B	0.06	0.09	0.14	0.27	0.40
	C	0.16	0.20	0.24	0.36	0.44
	D	0.22	0.27	0.30	0.40	0.47
20	A	0.00	0.04	0.17	0.80	1.68
	B	0.28	0.46	0.69	1.33	1.98
	C	0.80	0.98	1.18	1.78	2.21
	D	1.11	1.33	1.50	1.98	2.34
61	A	0.00	0.13	0.52	2.39	5.03
	B	0.85	1.39	2.07	3.99	5.95
	C	2.39	2.93	3.54	5.33	6.64
	D	3.32	3.99	4.49	5.95	7.01

Table 10. Pond sizes to capture calculated runoff from the 2.5 cm (1 in.) event for various land uses.

Runoff from 2.5 cm (1 in.)						
Pond Areas (ha)		% Pervious				
Watershed Area (ha)	Soil Group	12	25	38	65	85
4	A	0.00	0.00	0.00	0.01	0.08
	B	0.00	0.00	0.01	0.05	0.11
	C	0.01	0.02	0.04	0.09	0.14
	D	0.03	0.05	0.06	0.11	0.15
20	A	0.00	0.00	0.00	0.06	0.39
	B	0.00	0.01	0.04	0.24	0.55
	C	0.06	0.11	0.18	0.44	0.69
	D	0.16	0.24	0.30	0.55	0.77
61	A	0.00	0.00	0.00	0.20	1.17
	B	0.00	0.02	0.13	0.71	1.65
	C	0.20	0.34	0.54	1.31	2.06
	D	0.47	0.71	0.91	1.65	2.31

Table 11. Pond sizes to capture calculated runoff from the 3.8 cm (1.5 in) event for various land uses.

Runoff from 3.8 cm (1.5 in)						
Pond Areas (ha)		%Pervious				
Watershed Area (ha)	Soil Group	12	25	38	65	85
4	A	0.00	0.00	0.00	0.06	0.17
	B	0.01	0.16	0.04	0.13	0.22
	C	0.06	0.08	0.11	0.19	0.25
	D	0.09	0.13	0.15	0.22	0.28
20	A	0.00	0.00	0.01	0.29	0.86
	B	0.04	0.82	0.23	0.62	1.09
	C	0.29	0.39	0.52	0.93	1.28
	D	0.47	0.62	0.73	1.09	1.38
61	A	0.00	0.00	0.03	0.86	2.58
	B	0.13	2.45	0.69	1.85	3.28
	C	0.86	1.17	1.55	2.80	3.83
	D	1.42	1.85	2.19	3.28	4.14

Table 12. Pond sizes for capturing 2.5 cm (1 in.) of runoff.

2.5 cm (1 in) of Runoff	
Watershed Area (ha)	Pond Areas (ha)
4	0.28
20	1.38
61	4.14

Table 13. Pond sizes for capturing 3.8 cm (1.5 in.) of runoff.

3.8 cm (1.5 in) of Runoff	
Watershed Area (ha)	Pond Areas (ha)
4	0.41
20	2.07
61	6.21

APPENDIX M

PRE- AND POST-DEVELOPMENT MODEL OUTPUTS

Table 1. Pre-development peak runoff and total runoff volumes form 2-year 24-hour rainfall event for simulated watersheds in Charlotte, NC.

Charlotte	**2-year, 24-hour Event**		**8.9 cm**	**(3.5 in.)**
61 ha (150 ac)	SCS Type II Storm			
	Forested		Ag Soil	
Soil	A	B	A	B
CN	36	60	65	75
Peak Flow [m^3/s]	0	4.0	5.7	9.1
[ft^3/s]	0	140	200	320
Runoff Volume [m^3]	0	8000	12000	20000
[ft^3]	0	290000	410000	710000

Table 2. Pre-development peak runoff and total runoff volumes form 2-year 24-hour rainfall event for simulated watersheds in Wilmington, NC.

Wilmington	**2-year, 24-hour Event**		**11 cm**	**(4.5 in.)**
61 ha (150 ac)	SCS Type III Storm			
	Forested		Ag Soil	
Soil	A	B	A	B
CN	36	60	65	75
Peak Flow [m^3/s]	0	2.3	3.3	5.3
[ft^3/s]	0	81	115	188
Runoff Volume [m^3]	740	16000	20000	31000
[ft^3]	26000	560000	720000	1100000

Table 3. Post-development runoff model outputs for 90% and 2-yr event for Charlotte, NC (A).

Charlotte	SCS Type II Storm					
61 ha						
Land Use	2 ac		1/2 ac		1/4 ac	
Soil	A	B	A	B	A	B
CN	46	65	54	70	61	75
90 % Event Designed Pond:						
[m^2]	0	1114.1	0	2639.835	369.8561	4911.29
[ha]	0.00	0.11	0.00	0.26	0.04	0.49
90% Runoff Volume [m^3]	0	2343	0	5551	778	10328
2-yr, 24-hr Model	8.9 cm (3.5 in.)					
Size of Riser Barrels [cm]	0	38	0	38	38	38
Stage Rise [m]	0.00	2.14	0.00	1.61	3.24	1.34
Post Peak Outflow [m^3/s]	0.07	0.40	2.06	0.34	0.51	0.30
Post Total Runoff Volume [m^3]	1588	11595	4823	15543	8831	20072
Peak Flow Exceed Pre-Development Conditions?						
Forested:	YES	NO	YES	NO	YES	NO
Agriculture:	NO	NO	NO	NO	NO	NO

Table 4. Post-development runoff model outputs for 90% and 2-yr event for Charlotte, NC (B).

Charlotte	SCS Type II Storm			
61 ha				
Land Use	1/8 ac		Commercial	
Soil	A	B	A	B
CN	77	85	89	92
90 % Event Designed Pond:				
[m^2]	6061	12421	17006	21311
[ha]	0.61	1.24	1.70	2.13
90% Runoff Volume [m^3]	12745	26120	35761	44814
2-yr, 24-hr Model	8.9 cm (3.5 in.)			
Size of Riser Barrels [cm]	38	61	76	76
Stage Rise [m]	1.26	0.88	0.76	0.71
Post Peak Outflow [m^3/s]	0.28	0.55	0.74	0.70
Post Total Runoff Volume [m^3]	22056	31090	36344	40655
Peak Flow Exceed Pre-Development Conditions?				
Forested:	YES	NO	YES	NO
Agriculture:	NO	NO	NO	NO

Table 5. Post-development runoff model outputs for 90% and 2-yr event for Wilmington,, NC (A).

Wilmington	SCS Type III Storm					
150 ac						
Land Use	2 ac		1/2 ac		1/4 ac	
Soil	A	B	A	B	A	B
CN	46	65	54	70	61	75
90% Event Designed Pond:						
[m^2]	0	4970	764	8115	3022	12136
[ha]	0.00	0.50	0.08	0.81	0.30	1.21
90% Runoff Volume [m^3]	0	10451	1607	17065	6356	25520
2-yr, 24-hr Model	11 cm (4.5in.)					
Size of Riser Barrels [cm]	0	38	38	61	38	76
Stage Rise [m]	0.00	1.35	2.17	0.94	1.51	0.87
Post Peak Outflow [m^3/s]	1.20	0.30	0.40	0.58	0.32	0.54
Post Total Runoff Volume [m^3]	5141	20512	10655	25807	16641	31614
Peak Flow Exceed Pre-Development Conditions?						
Forested:	YES	NO	YES	NO	YES	NO
Agriculture:	NO	NO	NO	NO	NO	NO

Table 6. Post-development runoff model outputs for 90% and 2-yr event for Wilmington, NC (B).

Wilmington	SCS Type III Storm			
61 ha				
Land Use	1/8 ac		Commercial	
Soil	A	B	A	B
CN	77	85	89	92
90% Event Designed Pond:				
[m^2]	14019	23457	29587	34979
[ha]	1.40	2.35	2.96	3.50
90% Runoff Volume [m^3]	29481	49328	62219	73556
2-yr, 24-hr Model				
Size of Riser Barrels [cm]	61	91	107	0
Stage Rise [m]	0.85	0.66	0.62	0.59
Post Peak Outflow [m^3/s]	0.53	0.93	1.00	1.01
Post Total Runoff Volume [m^3]	34084	44854	50806	55538
Peak Flow Exceed Pre-Development Conditions?				
Forested:	YES	NO	YES	NO
Agriculture:	NO	NO	NO	NO