ASSESSING THE HYDROLOGY AND POLLUTANT REMOVAL EFFICIENCIES OF WET DETENTION PONDS IN SOUTH CAROLINA

A thesis submitted in partial fulfillment of the requirements for the degree

MASTER OF SCIENCE

in

ENVIRONMENTAL STUDIES

by

MARK JOSEPH MESSERSMITH
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at

THE GRADUATE SCHOOL OF THE COLLEGE OF CHARLESTON

Approved By:

____
Dr. Denise Sanger, Thesis Advisor

____
Dr. Guy DiDonato

____
Dr. David White

____
Dr. Susan Wilde

____
Dr. Amy Thompson McCandless, Dean of the Graduate School

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ABSTRACT

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Stormwater ponds are a commonly used best management practice (BMP) to mitigate flooding risks and to minimize the impact of development on surrounding aquatic ecosystems. Of the different types of stormwater ponds, wet detention ponds are the most commonly utilized systems in coastal SC. These ponds are designed to reduce stormwater flow to pre-development conditions, and retain the first flush of runoff. This study investigated the performance of two wet detention ponds constructed on the relatively flat topography that characterizes this region. Two different ponds systems were: a single pond (SRP) and the terminal pond in a series of 5 interconnected ponds (MRP). The objectives were to determine (1) the rate of sedimentation within the ponds, (2) the hydraulic functioning of the ponds, and (3) the overall effectiveness with regard to water quality. To accomplish these objectives, a bathymetric survey was performed and Teledyne ISCO® autosamplers were deployed to collect hydrographic information and flow-weighted composite samples. In the 5-7 years since their construction, MRP has 15 % less storage capacity, while SRP has 36 % less storage capacity. MRP had a slower rate of discharge than SRP, possibly due to the effect of the multiple ponds in series and a higher length to width ratio (3.14 to 1.5, respectively). Event removal efficiencies were variable and ranged from negative (up to –456 %), to near 100% removal. The total pollutant removal efficiency for fecal coliform bacteria was 55 % and 14 %, TSS was 88 % and 19 %, TP was 71 % and –6 % and TN was 39 % and –3 %, for MRP and SRP, respectively. This study found that stormwater ponds result in outflow being more typical of urban/suburban hydrographs than forested ones, and that they can result in high pollutant loadings to the receiving water bodies.

Introduction

Human population increase inevitably leads to landscape changes resulting from urbanization, including increasing impervious cover and changing soil permeability, vegetation types, and changing drainage morphology (Kelly 1993, Jeng et al. 2005). These landscape changes reduce the amount of stormwater that can penetrate into groundwater, thereby increasing the amount of surface runoff entering receiving water bodies (Arnold and Gibbons 1996). Runoff from impervious areas can be up to 16 times higher than that from natural, more pervious areas (Schueler 1995). Increasingly urbanized landscapes not only effect water quantity but can also lead to increased non-point source pollution (Booth and Jackson 1997, Harrell and Ranjithan 2003, Holland et al. 2004). Recent research has shown that stormwater runoff can result in negative impacts on receiving water bodies, including erosion of stream banks, loss of aquatic habitat, eutrophication in lakes, and declining water quality (Bricker et al. 1999, Marsalek et al. 2002, Holland et al. 2004, Houlahan and Findlay 2004, Mallin et al. 2004, Taebi and Droste 2004, Jeng et al. 2005).

To mitigate the effects of urbanization and increasing stormwater runoff, the Pollution Control Act (PCA) of South Carolina, Title 48 of the 1976 Code of Laws was enacted to maintain "reasonable standards of purity of air and water while allowing for development of the state". Regulation 72-300 of the South Carolina Stormwater Management and Sediment Reduction Act (1991) provided the requirements for preparing a stormwater management and sediment control plan (SCDHEC 2002b). This regulation requires that Best Management Practices (BMPs) be used for all land-disturbing activities in order to treat stormwater runoff so that the post-development stormwater discharge does not exceed the pre-development discharge and the first half-inch (12.7mm) of rainfall must be detained and discharged over a 24 hour period (SCDHEC 2002a). Many pollutants have a strong affinity to adhere to sediment (Jeng et al. 2005), which settle out during the detention of the water resulting in the retention of many pollutants of concern before being discharged into the receiving water bodies. Ponds are constructed based on modeling programs that use designed rainfall events, however, little information is gathered after construction. Understanding the performance of these ponds is essential for improving our coastal watershed management practices and protecting SC's economically and environmentally critical natural resources.

BMPs are a wide-ranging group of management practices that involve both structural and non-structural components to compensate for land disturbing activities by controlling the quantity and quality of stormwater runoff (SCDHEC 2003, Villarreal et al. 2004). Examples of BMPs include detention ponds (wet and dry), retention ponds, wetlands, percolation trenches, grassed swales, grassed buffer zones, pervious pavement, infiltration basins, vegetated waterways, inlet controls, and aeration (Lawrence et al. 1996, SCDHEC 2005). Stormwater ponds as a BMP have been widely utilized throughout the United States and some European countries (Anderson et al. 2002, USEPA 1999), and they are the most commonly used BMP in coastal South Carolina (SCDHEC 2005, J. Fersner personal communication).

Stormwater pond is the broad term used to define any structure that holds water during and after storm events. These ponds are generally designed to function as either detention or retention ponds. Detention ponds are defined by SCDHEC (2002a) as a "permanent stormwater management structure whose primary purpose is to temporarily store stormwater runoff and release the stored runoff at controlled rates". Detention ponds can be classified as either wet or dry ponds. A wet detention pond has a permanent pool of water, while a dry detention pond remains dry except during rain events. Retention ponds are a permanent structure designed to store stormwater, and "release of the given water is by infiltration and/or evaporation (SCDHEC 2002a). These ponds are designed to handle the volume of water calculated to run off a site based on the 24-hour designed storm event (Harrell and Ranjithan 2003). They are modeled primarily for water quantity control and there is little emphasis placed on the resultant pollutant loadings from them, the water quality within them, or the affects on groundwater quality.

Stormwater pond water quality is an increasing concern in coastal South Carolina. One reason for concern is that residents commonly view these ponds as an amenity, and they are being used for recreation, fishing, crabbing, boating, swimming and irrigation (Anderson et al. 2002). Parcels around ponds are also commonly sold as waterfront property, which increases property values. Therefore, residents are concerned when algal blooms, fish kills, sedimentation, and other water quality problems are observed within the ponds.

Stormwater ponds can become natural "incubators" for microorganisms, including harmful algae species (HABs) (Lewitus et al. 2003). HABs are a concern for a number of reasons, including aesthetic (e.g., fish kills), environmental (e.g., low dissolved oxygen) and human health (e.g., aerosolizing toxins) (Lewitus et al. 2003). Algal blooms can occur as a result of high nutrient inputs that change the microbial and phytoplankton composition within the ponds (Lewitus et al. 2003, Serrano 2005, Mason-Brock 2006). Lewitus et al. (2003) found 82% of the stormwater ponds on Kiawah Island, SC, had HABs associated with them. Other water quality indicators have also been found to be elevated in stormwater ponds, including fecal coliform bacteria, and various nutrient parameters (Lewitus et al. 2003, Serrano 2005, Drescher unpublished data). In a compilation of a number of studies, Schueler (1994) found the mean nutrient concentrations in wet detention pond effluent were over the U.S. Environmental Protection Agencies (USEPA) standards for freshwater bodies. White et al. (2004) found that while stormwater ponds may be effective for detention, they are a landscape alteration that can reduce bio-filtration and nutrient uptake, unlike natural wetlands.

The impact of stormwater on the ecological health of receiving water bodies is not well understood (Hall and Anderson 1988, Wei and Morrison 1992, Jeng et al. 2005). In the coastal zone of SC, many stormwater ponds release water directly into tidal creeks that ultimately drain into larger estuaries. Researchers have found that stormwater is contributing to adverse affects within these receiving waters, including both short and long term problems to biological life (Taebi and Droste 2004, Jeng et al. 2005). Given the negative effects of development on the water quality and overall health of receiving waters and more specifically, tidal creek ecosystems (Jordan et al. 1991, Bricker et al. 1999, Mallin et al. 2000, Gaffield et al. 2003, Holland et al. 2004, Goonetilleke et al. 2005, Jeng et al. 2005), it is important to study the functionality of BMPs, particularly the commonly utilized wet detention ponds.

There have been a number of studies that have focused on obtaining pollutant removal efficiencies for water quality parameters in stormwater ponds; however, these studies have found varying results (Lindsey et al. 1992, Wu et al. 1996, Borden et al. 1998, Harper 1999, Pettersson et al. 1999, Comings et al. 2000). Variable pollutant removal efficiencies are generally attributed to the vast differences in pond design, including size, volume, shape, residence time, and ratio of pond area to catchment area (Pettersson et al. 1998). For example, Mallin et al. (2002) studied the pollutant removal efficacy of three wet detention ponds in New Hanover County, NC. They observed fecal coliform removal rates varied from 56% to 86% in two of the ponds, to a removal rate of -15% in the third pond, and that higher removals were found to be primarily dependent upon the length to width ratio between the inflow and outflow pipes within the ponds.

The goal of this study was to evaluate the performance of wet detention ponds constructed on the relatively flat topography along coastal South Carolina. The specific objectives of this study were to: (1) determine the change in storage capacity, (2) determine the pollutant removal efficiencies, (3) determine if the wet detention ponds are meeting the SC regulations for pre- and post-development runoff rates, and (4) develop water budgets to analyze the hydraulic functioning of the ponds.

Materials and Methods

Study Sites

Daniel Island, located in the City of Charleston, Berkeley County, SC is a recently developed community that has experienced a high rate of growth over the last 10 years, and is continuing to see increased development pressures (Figure 1). Since the community is relatively new, the stormwater ponds were designed and permitted in accordance with the 1991 South Carolina Stormwater Management and Sediment Reduction Act.

Two stormwater pond systems were selected for investigation. The first pond, termed multiple residential pond (MRP), was the terminal pond in a series of five interconnected ponds permitted by OCRM on April 7, 1999. MRP has three inflow pipes: number one receives input from the first four ponds in the series and contributes water from 70.1% of the surrounding watershed, number two receives input from 18.2% of the surrounding watershed, and number three receives input from 5.1% of the surrounding watershed (Figure 2). The outflow pipe releases the water to the headwaters of an unnamed creek that flows into the Wando River. The second pond, termed single residential pond (SRP), was an individual pond permitted on March 4, 1999. SRP has two inflow pipes draining its surrounding watershed of 30% and 57%, respectively, and one outflow pipe releasing its water to Beresford Creek (Figure 3).

Both ponds were modeled to meet both the state of South Carolina and the City of Charleston requirements. The state of South Carolina has the following requirements:

1. "Peak post-development release rates from the basin shall be at or below pre-development rates for the 2 and 10 year 25 hour storm events.
2. Permanent water quality ponds having a permanent pool shall store and release the first ½ inch of runoff from the site over a 24-hour period.

3. During construction, at least 80% of the sediment must be trapped on site."

In addition to the state requirements, the ponds must be designed to meet the standards of the city where they were constructed. In this case, the City of Charleston has the following requirements:

1. "Detention shall be adequate to keep the peak post-development release rate from the basin at or below pre-development rates for the 10-year storm.
2. Maximum lake levels for the 25-year storm will allow roads to remain passable.
3. Maximum lake levels for the 100-year storm will be below minimum building floor elevations.
4. Drainage basin study will identify impact of the fully developed basin on downstream and upstream properties."

Drainage Basins

Environmental Systems Research Institute, Inc.'s (ESRI) ArcGIS 9 ® was used to map the drainage basins and land use of each pond system. Engineering site plans, developed by Thomas and Hutton Engineering Company (T&H), were used to define the drainage basins for both of the ponds, and were hand digitized.

Land cover was delineated within each pond watershed using six-inch resolution aerial imagery provided by Berkeley County. A modified version of the Anderson (1976) Land Use Classification System was used, which included structures, landscaped, forested, roads, sidewalks/driveways, and water. Area was calculated for each of these categories. Percent impervious cover was calculated as the sum of structure, road, and sidewalk/driveway areas (impervious categories) divided by the total land area and multiplied by 100.

Bathymetry

Bathymetric surveys of the two study ponds were conducted using a Garmin ® GPSMAP 135 Sounder with Garmin's ® GBR 23 beacon receiver to collect x, y coordinates with each depth value (z coordinate). The unit was fixed to a canoe and data were collected throughout each of the ponds. The depths obtained by the sounder were corrected by adding the depth of the water above the sounder plus the difference between current water level and the normal water elevation (1.37 m above mean high tide).

These x, y coordinates and depth values were entered into ArcGIS ®. The pond edge (i.e., water level) was hand digitized using aerial photography, which was assigned a depth of 0 meters. These depth points were used to create a Triangulated Irregular Network (TIN) and pond contours using the Arc 3-D Analyst ® extension. A new TIN was created, based on the contours and the outline of the pond, and used to calculate the volume for each pond.

The designed volume of each pond was determined from the stormwater plans submitted for approval to OCRM. Contour lines were buffered from the outline at the specified intervals and depths shown in the stormwater plans (T&H 1998, T&H 1999). These contour lines were converted to points and the aforementioned procedure was applied to ascertain the designed volumes for each pond. The ponds were assumed to be constructed as designed; however, the accuracy of that assumption is unknown. Therefore, a comparison of the designed volume and the existing volumes will be made to evaluate how much sedimentation has occurred within the ponds since their construction.

Event Sampling

Monitoring occurred throughout the summer of 2006, and efforts were focused on sampling one pond at a time. When rain was approaching, sampling equipment was deployed and automatic samplers were activated. Events had to be intense enough to produce stormwater flow into and out of the ponds. Sampling efforts were geared towards obtaining a multitude of data, including: (1) semi-continuous physical water quality; (2) in pond water quality prior to and after the events; and (3) hydrographic and pollutant concentration data to be used in determining water budgets, as well as loads, efficiencies and yields.

Semi-Continuous Water Quality

Semi-continuous water quality was measured within the ponds during the course of sampling at each site. Data were collected in MRP from 5/24/2006 to 7/18/2006 and in SRP from 8/30/2006 to 10/3/2006. Stevens Water Monitoring Systems, Inc. dataloggers collected conductivity, temperature, depth, pH (model CTDP300) and dissolved oxygen (DO) (model DO300) at 30 minute intervals. Salinity was calculated using the un-normalized conductivity according to the algorithm outlined in Standard Methods for the Examination of Water and Wastewater (APHA 1998). Datasondes were deployed three meters from the edge of the pond. The depth sensors were positioned approximately 0.3 m off the pond bottom. Dataloggers were calibrated prior to deployment and Quality Assurance / Quality Control (QA/QC) was performed post-deployment. Data not within the manufacturer suggested ranges failed QA/QC and were not included in the dataset. DO data was removed from 7/3/2006 to 7/18/2006 in MRP and 9/7/2006 to 9/16/2006 in SRP. The water quality data were summarized to provide insight into changing pond conditions during the length of deployment and surrounding the rain events. Although standards for water quality do not exist for stormwater ponds, results were compared to existing water quality standards as a guideline as to what is ‘normal’ in freshwater lakes (Table 1).

In Pond Sampling

Water samples in each pond were collected prior to each rain event (pre) and after each rain event at approximately 24 and 48 hours (post-24 and post-48). The water sample consisted of two grab samples from three locations around the edge of the pond using a pre-cleaned 1 L polypropylene Nalgene bottle. Each sample was collected at a depth of 0.3 m. The samples were composited in a pre-cleaned (using LiquiNox ®) 9.5 L polypropylene Nalgene bottle and homogenized by inverting ten times before aliquots of the water sample were divided into appropriate containers (Table 2). Samples were transported to Trident Laboratory Services in Ladson, SC for processing. All samples were analyzed within the specified holding times (Table 2). Constituents measured were fecal coliform bacteria (FC), total suspended solids (TSS), total kjedhal nitrogen (TKN), nitrate + nitrite (NO_{2+3}), ammonium (NH_4^+), dissolved TKN, dissolved NO_{2+3}, dissolved NH_4^+, total phosphorus (TP), total dissolved phosphorus (TDP), and orthophosphate (PO_4^{3-}) (Table 2).

For data analysis, concentrations below the lower limit of detection were reported as the lower limit. Fecal coliform concentrations that were above the limit of detection were reported as the upper limit of detection. This ensured a minimal bias in the sample analyses.

Data were obtained for all events except for events 3, 4 and 5 in MRP, which occurred consecutively and resulted in limited data for pre- and post- pond samples. In order to analyze these data, the pre- sample was used from event 3 and the post-24 and post-48 samples were used from event 4. These two events occurred less than 24 hours apart. Data from MRP and SRP were analyzed together due to the low sample size (i.e., 3 events per pond). All statistical analyses were conducted using Minitab® version 15 software. Normality was tested on the percent differences between time periods using the Ryan-Joiner test and were considered normal at $\alpha = 0.10$. Using the percent differences between paired samples enabled a paired t-test to test for differences between pre and post-24, pre and post-48, and post-24 and post-48. Results were considered significant at $\alpha = 0.10$.

Hydrographic Data

Along with pond water quality surrounding rain events, Teledyne ISCO 6712 Portable Auto Samplers were used to measure rainfall and hydrographic data, and collect composite flow weighted samples during these events. Each ISCO was equipped with a tipping bucket rain gauge (ISCO model 674) and an Area Velocity Flow Module (ISCO model 750). Using Doppler technology, the flow module detects the average velocity of the water as it moves through the pipe. The flow module was equipped with a pressure transducer that measures the level of water. The channel dimension was programmed into the ISCO and used to calculate the flow rate along with level and velocity.

Flow meters and sample tubing were positioned at the bottom of the storm pipes. Deployment designs varied due to size, position, depth, and distance into the pond of each inflow/outflow structure. All inflow pipes contained water that resulted in deploying the instrumentation in submerged pipes except for the input connecting the pond in series (MRP, inflow 1). To minimize pond water interference at the end of the pipes, the flow meter and sample tubing were deployed approximately 1.25 m from the end of the pipe. MRP, Inflow 1 was the connection between the fourth pond in the series and the terminal or fifth pond (MRP). Pond four in the series had a riser box that resulted in water spilling over a weir and into the box before traveling to the terminal pond. Therefore, the flow meter and sample tubing were positioned on the spillway and the flow was determined using a weir calculation using Flow-Link © software. This calculation uses the channel dimensions of the weir and the height of the water to determine flow volume. MRP, inflow 3 was not measured due to an insufficient amount of instrumentation and because the drainage basin was the smallest of the three inflow pipes.

Hydrographic data (i.e., flow rates and rainfall) were downloaded to an ISCO Rapid Transfer Device (RTD) © following each sampling event. The data were uploaded using Teledyne ISCO software and imported into Microsoft Excel © for analysis and graphing review. QA/QC was performed on all data. If readings were negative or erratic flow rates were observed then the data were deleted and/or interpolated using the best available information to provide estimates of the flow as follows: (1) all negative inflows were assigned a value of 0 Lps (liters per second), (2) interpolations were made between periods of unreasonable readings, and (3) trendlines were used to extrapolate the ends of some hydrographs that were stopped before flow rates reached 0 Lps.

If hydrographic data was not available (i.e., pipe not instrumented or unreasonable values obtained), then the runoff was determined by developing water budgets and using runoff calculations.

Development of Water Budgets

A water budget was developed for each pond and event to account for the water moving through the pond (Equation 1, Figure 4)

$$\Delta V/\Delta t = Q_i + P + R - O - ET - Q_o \pm S \qquad [1]$$

Where: $\Delta V/\Delta t$ = change in volume over the change in time

Q_i = inflow from pipes

P = direct precipitation

R = sheetflow runoff

O = pond overflow

ET = evapotranspiration

Q_o = ouflow from pipes

S = groundwater recharge or discharge

Qi was measured if the pipe was monitored and functioned correctly, or was estimated using the Simple Method (see below) if the pipe was not monitored or the equipment malfunctioned. Precipitation (P) was the water gained from rainfall falling directly into the pond and was calculated as the amount of rainfall multiplied by the area of the pond. Sheetflow runoff (R) was the amount of water that entered the pond from the perimeter of the pond and was determined based on the calculated volume obtained by the Simple Method. Engineered outflow (Q_o) is the discharge measured by the ISCO flow meter in the outflow pipe. Evapotranspiration (ET) was assumed to be negligible during a rain event and was assigned a value of 0. Groundwater (S) could be a potential source or sink to the pond over the long term; however, this is thought to be minimal over the course of a rain event (T. Callahan personal communication). Overflow volume (O) was determined by mass balance when water was suspected to have left over the weir (MRP only).

Three methods were evaluated to determine which was most appropriate for estimating the unmeasured inflows and the sheetflow: (1) the Soil Conservation Service (SCS) Technical Release 55 (TR 55) method using actual precipitation, (2) the SCS TR 55 method using 24-hr fixed precipitation, and (3) the Simple Method (Schueler 1987).

SCS TR 55 is the most widely used and accepted method to calculate runoff (J. Fersner personal communication, J. Dupre personal communication). The two versions of the SCS TR55 method both use the following equations (2, 3, and 4):

$$Q_{vol} = Q_d A_d / 12 \qquad [2]$$

Where: Q_{vol} = volume of runoff, acre-feet

Q_d = depth of runoff, inches

A_d = drainage area, acres

12 = conversion factor, inches/feet

$$Q_d = (P - I_a)^2 / (P - I_a + S) \qquad [3]$$

Where: P = depth of 24-hr rainfall (in.)

I_a = initial abstraction (rainfall lost to infiltration and surface depressions before runoff occurs (in.). This is commonly given as 0.2S (Fergusen 1998)

S = potential maximum retention after runoff begins (in.) and is calculated by:

$$S = (1000/CN) - 10 \qquad [4]$$

Where: CN = curve number (function of drainage ability of an area's soil and land use)

A CN is a number between 0 and 100, in which larger numbers designate less permeable areas. Weighted CN's were determined by summing the different fractions of land use multiplied by the CN for each particular land use. The two variations of the method involved the precipitation variable (P). The first calculation involved using the actual measure of precipitation for each event (J. Dupre personal communication, R. Geer personal communication), while the second method standardized the precipitation from each event based on a 24-hr event to ensure more accurate results based on rainfall intensity (e.g., 25.4 mm / 6 hr. period = 101.6 mm / 24 hr. period).

The method used by T&H to design the ponds was based on the SCS TR 55 method. They used the "Advanced Interconnected Channel and Pond Routing" (adICPR) (Streamline Technologies, Inc) method to design the ponds and model the rainfall, runoff and hydraulics within the drainage basins. This application uses a detailed analysis of the watershed characteristics and can analyze an extensive array of different stormwater control devices that engineers' use, including culverts, drop structures, and weirs. The user can generate both graphical and tabular forms of peak rates of flow and pond elevations to demonstrate system design to regulatory agencies. This method generates weighted curve numbers (CNs) from the SCS TR 55.

The third method was the Simple Method, developed by Schueler (1987). This method calculates the event runoff from each of the watersheds (Equation 5).

$$Q = \{[(P * P_j * R_v)/12] * A\} * 1233481.84 \qquad [5]$$

Where: Q = Runoff (Liters)

P = Precipitation (inches)

P_j = fraction of event that produces runoff

R_v = runoff coefficient (fraction of event that is converted to runoff)

12 = unit conversion (inches to feet)

A = watershed area (acres)

1233481.84 = unit conversion (acre-feet to liters)

The Simple Method was developed to calculate annual loadings, but was modified in this case by setting P_j to 1. This assumes any rainfall produced a measurable volume of runoff (Smith 2005). The runoff coefficient (R_v) was calculated using Equation 5 (Schueler 1987), which is based on the linear regression between the percent impervious cover of a variety of watersheds and their associated runoff coefficients.

$$R_v = 0.05 + 0.009I \qquad [6]$$

Where: I = percent impervious cover in the watershed

The Simple Method provides a reasonable estimate of the runoff volume from a watershed, but some limitations should be noted. The primary factor influencing runoff in this method is impervious cover and watershed area. This method does not factor in land use type, soil type, or rainfall intensity, as well as the conveyance of stormwater from the site. Additionally, the pre-rain event elevation of water within the stormwater pond is not considered. These factors could influence the actual volume of runoff. The three method results were compared to the measured volumes at the inflow and outflow pipes to determine which method to use.

Flow-Weighted Composite Sampling

Along with measuring and calculating volume from rain events, flow-weighted composite samples were collected to determine the mean concentrations of pollutants entering and leaving the ponds during the monitored rain events. Composite samples were collected using ISCO auto samplers programmed to begin sampling at a pre-determined intensity of rainfall (i.e., 2.54 mm / 30 minutes), and collect samples based on the flow rate. Samples were collected into 9.5 L (2.5 gallon) glass containers through PVC tubing. Sample containers were iced during rain events and remained on ice until delivery to Trident Laboratory Services. The rainfall had to be intense enough to convey runoff from the drainage area and into the ponds, while at the same time providing enough volume to raise the pond elevation enough to have flow through the outflow pipe. Every effort was made to sample consecutive rain events in each pond to ensure that sampling was not biased to events that follow long dry periods, where pollutant concentrations are likely to be higher (Pettersson et al. 1999).

Each ISCO was also programmed to then collect stormwater samples at a given flow rate (e.g., draw 250 ml of sample for every 10,000 L of flow) based on results obtained during test runs. This results in more sample volume being collected at greater flow rates. A flow-weighted composite provides a more reliable estimate of the total loadings leaving a site compared to point sampling during the event (Comings et al. 2000). Within 24-hours of the event, composite samples were retrieved; however, if discharge from an outflow was still occurring then a grab sample was collected from the pipe along with initiating the collection of a second composite sample. Each set of samples were homogenized by inverting ten times before aliquots of the water sample were divided into appropriate containers and transported to Trident Laboratory Services in Ladson, SC for processing as described above (Table 2). All samples were analyzed within the holding times except for rain event auto-samples, which did not adhere to the six-hour fecal coliform bacteria holding times, due to the flashy nature of summertime rain events in the southeastern United States. For the purposes of this study it was adequate to ensure that samples were analyzed within 24 hours (APHA 1998).

Event Loadings and Yields

Using the volumes used from the water budgets and concentrations obtained from the flow-weighted composite samples, event loads were calculated for each specific inflow and outflow (Equation 7).

$$L = Q * C \qquad [7]$$

Where: L = Load (kilograms)

Q = Flow volume (Liters)

C = Concentration (kilograms/Liters)

In some cases it was difficult to completely composite the entire rain event and the following series of criteria were used to obtain event loads: (1); If only one composite sample was obtained for a given hydrograph, those concentrations were applied to the entire volume from that hydrograph; (2); If a grab sample was collected from the pipe after the flow-weighted composite was collected, then the grab sample represented the volume from the time that the flow-weighted composite sampling ended until the end of the hydrograph, or until the next composite sample started collecting; and (3); If there were two flow-weighted composites collected during an event, the first sample represented the volume of the first composite and the second sample represented the volume from the time the first composite sample ended until the end of the event hydrograph.

For some rain events, the sample volume was not sufficient to process for all parameters identified. Therefore, these unmeasured parameter concentrations were estimated based upon samples collected during the rain event by the following rules: (1) unmonitored inflow pipes (e.g., MRP, inflow 3) were assigned the concentrations from a pipe that drained a similar land use type (e.g., MRP, inflow 2), and (2) if two or more composite samples were taken from a pipe over the course of a rain event and not all parameters were measured for one of the composites, then the unknown concentrations were assigned those from the other composite.

From the loading calculation, a yield was determined for each pond and each rain event by dividing the load (kg) by the area of the watershed (ha). These yields provided a means to standardize the loads between the ponds.

Pollutant Removal Efficiencies

The US Environmental Protection Agency (2004) defines pollutant removal efficiency as a "measure of how well a BMP or BMP system removes pollutants". Specific removal efficiencies were calculated in each pond for each parameter during each rain event. The pollutant removal efficiency was calculated by dividing the amount of each constituent removed by the amount of each constituent that entered the pond (Equation 8):

$$\%\ Removal\ efficiency = [(\Sigma L_{in} - \Sigma L_{out}) / \Sigma L_{in}] * 100 \qquad [8]$$

Where: $\Sigma L = L_1 + L_2 + \ldots + L_n$

And L_n = parameter load from a pipe

Removal efficiencies are presented for each individual event, along with the average event removal. Additionally, in order to determine the percent removal over a period of time rather than from specific events, and to enable more representative comparisons between ponds, the percent removal from the sum of the total loads from all events captured was also obtained (Equation 9) (Wu et al. 1996 and Comings et al. 2000).

$$\%\ Removal\ efficiency = [(\Sigma L_{in\ (all\ events)} - \Sigma L_{out\ (all\ events)}) / \Sigma L_{in\ (all\ events)}] * 100 \quad [9]$$

Results

Drainage Basins and Land Cover

The entire drainage basin area of MRP was 28.35 ha (Table 3). The drainage basin area for inflow 1 represents 19.87 ha, while the drainage area for inflow 2 represents 5.16 ha. Inflow 3 and NPS were similar with drainage basin areas representing approximately 1.5 ha. MRP pond 5 had a length to width ratio of 3.29, and a pond area to drainage area ratio of 0.0009. When all ponds in the drainage area of MRP were used for this calculation, the pond to drainage area ratio was 0.0272. Impervious cover represented 39.4 % of the drainage area for MRP and within the sub-basins, ranged from 28.3% to 50.1%. The predominant land cover type was the landscaped areas (15.4 ha) followed by houses (4.3 ha) (Table 3, Figure 5). The drainage basin for MRP also included 1.8 ha of undisturbed/forested land.

The entire drainage basin area of SRP was 3.14 ha (Table 4). The drainage basin for inflow 1 represented 0.95 ha, while the drainage area for inflow 2 represented 1.79 ha. The sheetflow, or non-point sources represented the remaining 0.26 ha of the drainage area for SRP. SRP had a length to width ratio of 0.377, and a pond area to drainage area ratio of 0.0446. Impervious cover represented 50.3% of the drainage area for SRP and within the sub-basins, ranged from 9.4% to 52.6%. The predominant land cover type was landscaped areas (1.56 ha). Roads, concrete and houses represented approximately 0.5 ha each of the drainage area for SRP (Table 4, Figure 6). The drainage basin for SRP included no area of undisturbed/forested land.

Bathymetry

The stormwater plan designed volume for MRP was 3,897,180 L of water, and the side slope was designed to be 2:1 around the entire pond (Figure 7A). The current volume of water in MRP was 3,308,760 L, a decrease of 15.1% from the designed volume (Figure 7B). SRP had a designed volume of 1,674,590 L of water, and the stormwater plans also identified a 2:1 slope around the entire pond (Figure 8A). The current volume of water in SRP was 1,069,730 L, a decrease of 36.2% from the designed volume (Figure 8B). In both ponds, areas of sedimentation and bank erosion were evident (Figures 7 and 8). The inflow pipes were the primary areas of increased sedimentation. The center depths of both ponds were similar to the designed depth.

Rainfall

A total of five rain events were sampled in MRP (Table 5). These events were short in duration, ranging from only 0.17 hours to 1.83 hours. They were also generally high, yet variable in intensity and ranged from 5.59 mm/hr to 52.29 mm/hr. The rainfall totals for MRP ranged from 5.59 mm to 26.92 mm. The largest rain event in both rainfall amount and duration occurred when sampling SRP on 8/31/2006, during the passing of Hurricane Ernesto. SRP had longer lasting rain events, ranging from 4.08 hours to 16.92 hours. These events were less intense than those studied for MRP and ranged from 2.39 mm/hr to 6.54 mm/hr. The rainfall totals in SRP ranged from 16.51 mm to 85.34 mm. Thunderstorms in the Southeastern United States are common in the summer and can often produce varying rainfall amounts that occur during high intensity rainfall. Small rain events were intense enough to generate high volumes of stormwater runoff and spikes in inflow and outflow were recorded. The subsequent sections provide the results from data collection during the event sampling periods.

Event Sampling

Semi-Continuous Water Quality

MRP had a mean temperature of 29.34 °C, and ranged from 26.14 ºC to 32.13 ºC, while SRP had a mean temperature of 27.88 ºC, and ranged from 24.35 ºC to 32.69 ºC (Table 6). Both ponds showed an expected diurnal temperature pattern, with highest temperatures occurring during the late afternoon and lowest temperatures during the early morning hours (Figure 9). Temperature values were decreased during and after rain events, usually displaying a 12-hour lag time before returning to the typical diurnal pattern (Figure 9).

The pH data also displayed a diurnal signal in both ponds (Figure 9). The pH declined following rain events, and this pattern was stronger in SRP than in MRP. MRP had a mean pH of 6.94, with a range from 6.44 to 8.14, while SRP had a mean pH of 7.16, with a range from 6.41 to 8.69 (Table 6).

The dissolved oxygen (DO) concentrations were variable and displayed a diurnal signal (Figure 10). Maximum DO concentrations displayed a slight decrease following rain events in both ponds before a return to normal (Figure 10). The mean DO in MRP was 8.13 mg L^{-1}, and had a maximum concentration of 13.12 mg L^{-1}, and a minimum concentration of 3.66 mg L^{-1} (Table 6). The mean DO in SRP was 4.57 mg L^{-1}, and ranged from 0.92 mg L^{-1} to 8.68 mg L^{-1} (Table 6).

The mean salinity between the ponds varied slightly. MRP had a mean salinity of 0.01 ppt and SRP had a mean salinity of 3.58 ppt (Table 6). The maximum salinity observed in MRP was 0.09 ppt, indicating that estuarine water had not entered the outflow pipe. SRP had salinities near 0 ppt until a series of tides caused estuarine water to flow into the pond through the outflow pipe and subsequently raise the salinity in the pond to 5.65 ppt (Figure 11). These high tides were confirmed by negative flows recorded within the outflow pipe. The rain event on 9/13/2006 corresponded to a decrease in the salinity level from 5.65 ppt to approximately 5 ppt (Figure 11).

In Pond Sampling

In order to determine changing pond water quality conditions surrounding rain events, composite samples were collected from both ponds prior to, and approximately 24 and 48 hours after each event. The average percent change from the pre rain event samples are shown for FC, TSS, TN, and TP (Figure 12).

The general pattern of fecal coliform concentrations reveals an increase in concentration after rain events followed by a decline around 48 hours after the events (Figure 13). Results of the paired t-tests indicated that there were significant differences in fecal coliform bacteria concentrations between Pre and Post-24 ($p=0.018$) and Post-24 and Post-48 ($p=0.071$).

All phosphorus parameter concentrations increased 24 hours after the rain events but returned to background concentrations at 48 hours. The paired t-tests for TP found significant differences between Pre and Post-24 ($p=0.068$) (Figure 14). Similar to TP, TDP increased in concentration 24 hours after rain events followed by a decrease 48 hours after the rain events (Figure 15). Results of the paired t-tests found a significant difference between Pre and Post-24 TDP concentrations ($p=0.071$). The concentrations of PO_4^{3-} were statistically similar between the rain event sampling periods (Figure 16).

Nitrate + nitrite and ammonia samples were consistently below the limit of detection for both the whole and the dissolved fractions. Therefore, these parameters were not analyzed. No patterns were observed for TSS (Figure 17), TN (Figure 18) or TDN (Figure 19).

Hydrographic Data and Water Budgets

The sub-basin inflows into MRP and SRP exhibited a pattern of increased flow rate in association with rainfall intensity (Figures 20 and 21). The outflows of both ponds and the inflow for MRP also showed an increase in flow according to intensity; however, the flow rate was delayed after the rainfall, and displayed a steep increase followed by a slow decline afterwards (Figures 20 and 21).

Five rain events were captured in MRP, resulting in 13 total hydrographs passing QA/QC. Three rain events were captured in SRP, resulting in 9 hydrographs passing QA/QC. When data failed QA/QC, runoff calculations were used for the development of the water budgets (described later). Figure 22 is an example which failed QA/QC due to erratic flow readings and the discrepancy between volume measured and total rainfall within the watershed.

The peak rate of flow in SRP ranged from 6 Lps to 91 Lps compared to 3 Lps to 39 Lps in MRP (Table 7), with the highest flow rates occurring with the highest amount of rainfall. Regression analysis confirmed the relationship was significant (Figure 23).

On average, SRP drained faster than MRP (Figure 24). In SRP, approximately 99.5 % of the water left the pond within 24 hours; however, 75 % of the water discharged after only 6 hours during event 1, 6.75 hours during event 2 and 8 hours during event 3 (Table 7). A high spring tide resulted in tidal water input to the pond during event 2; however, the majority of the water had already discharged (Figure 25). After 24 hours, the discharge from MRP ranged from 69 % to 85 %. There is no data for the 24-hour time period for event 3 due to the overlap of event 4, which occurred before all the water had exited the pond. The time it took for 75 % of the water to discharge from the pond ranged from 13 hours to 28.5 hours (Table 7). Although a direct determination was not possible, both ponds appeared to meet the criteria of retaining the first 12.7 mm (1/2 inch) of rainfall and releasing it over 24 hours.

When stormwater pipes were not measured for flow and/or when flow measurements failed QA/QC, runoff calculations were used in developing the water budgets, and ultimately to obtain loads and efficiencies. Three methods, SCS Actual Precipitation, SCS 24-hr Precipitation, and the Simple Method, were used to calculate runoff from all drainage basins to determine which provided the best estimate. In order to do this, measured hydrographic data and rainfall within the basins were compared to each method results by analyzing volumes and consistency of the percentage of runoff. In MRP, the SCS Actual Precipitation resulted in percentages of runoff from the different drainage areas ranging from 0 % to 44 % for all rain events (Table 8). There was no consistency in occurrence of the smallest percentages or the largest percentages amongst events, and results appeared smaller than the measured volumes. The SCS 24-hour Precipitation also resulted in varying percentages of runoff, ranging from 46 % to 94 % (Table 8). These results appeared to be elevated when compared to the measured volumes of water from the different pipes. SRP showed similar variation with large differences between the calculated SCS methods and the actual measurements. The SCS Actual Precipitation showed values for the percentage of runoff from the different sub-basins ranging from 1 % to 55 % (Table 8). The SCS 24-hour Precipitation also showed varying results for the percentage of runoff from the different watersheds, and ranged from 18 % to 70 % (Table 8).

The Simple Method uses only watershed size and impervious cover as the driving factors in the calculation. This resulted in percentages that were consistent across rain events (Table 8). In MRP, 38 % of the water ran off from the inflow 1 drainage basin, 50 % from inflow 2, 42 % from inflow 3, and 30 % from the non-point sources. In SRP, 50 %of the water ran off from the inflow 1 drainage basin, 53 % from inflow2, and 14 % from the non-point sources. The non-point sources were the smallest watersheds in both ponds, as well as having the least amount of impervious cover, which resulted in the least amount of non-point source input to both ponds. Results using the Simple Method were the most comparable to the results from the measured stormwater pipes, and were used for completing the water budgets.

Water budgets were developed for each rain event in MRP (Table 9). The average percent difference or percent error in the budgets was –5.5 %, and ranged from individual event differences of –38.94 % to 16.18 %. Some notes about the water budgets for MRP should be mentioned, including: (1) there was no storage component during any of the events; (2) mass balance was used to determine the volume of water input by inflow 1 during event 1 because the influence of the first four ponds negated the use of the Simple Method; and (3) 0 L of water was used for inflow 1 during events 3 and 4 because the water level in pond four did not raise enough to spill over the riser box and flow into MRP. The inflows of MRP made up about 65 to 90 % of the total volume of water input to the pond during each event.

In SRP, water budgets were also developed for each rain event (Table 10). The percent differences for all three of the events were –0.28 %, 8.82 %, and 77.54 %, respectively. The inflows to the pond for all events were consistently around 90 % of the total water input to the pond. Due to malfunctions of the flow meter, inflow 1 of event 2 and inflows 1 and 2 of event 3 were calculated by the Simple Method. One reason for the large percent difference in event 3 could be due to low rainfall intensity resulting in less water entering the pond than calculated using the Simple Method.

Pollutant Loadings and Removal Efficiencies

The loading of each parameter (i.e., volume * concentration) for each event was calculated to evaluate the total amount of each constituent entering the pond and the receiving waterbody, and for use in determining the pollutant removal efficiencies. Loadings were calculated for each inflow and outflow pipe. The outflow loadings are particularly important considering they provide information about the total amount of each constituent entering the receiving waterbody (i.e., the area the stormwater pond is designed to protect). The pollutant removal efficiency provides information about how well the pond is retaining pollutants. A positive number indicates that more pollutants entered into the pond than were exported and a negative number indicates more pollutants were exported than entered. Removal efficiencies were calculated for each parameter and event, and were also calculated based on the sum of the total load removal of all events. Additionally, event yields were determined to standardize loadings based on drainage area.

The fecal coliform bacteria loadings to the tidal creek from MRP varied by three orders of magnitude (10^8-10^{11}). In comparison, the fecal coliform loadings to the tidal creek from SRP were less variable in the mid-point range of 10^{10} compared to MRP (Tables 11 and 12). For both ponds, the individual event loading into the pond was on average higher than the loading out except event 2 in MRP and event 3 in SRP which resulted in a higher export of fecal coliforms to the estuary than was received into the pond (Figure 26A). The total of all the loads in and out revealed that both ponds remove bacteria (Figure 26B). In MRP, the event pollutant removal efficiencies for fecal coliform bacteria ranged from –477.3 % to 98.8 % (Table 13). The average of the event specific removal efficiencies for fecal coliform bacteria in MRP was –67.3 %. This differs drastically from the percent removal from the sum of the total loads in and out from each event, which was 55.3 %. In SRP, fecal coliform bacteria removal efficiencies were only calculated for events 2 and 3. SRP removed 60.5 % of fecal coliforms during event 2 and had a –232.2 % removal during event 3. The average of these two events was –85.86 %; however, the percent removal from the sum of the total loads was 13.7 % (Table 14). Results from the yield calculations reveal that billions of these bacteria are entering the receiving water bodies from every hectare of land within the drainage basins of both ponds (Figure 27). The rainfall amounts varied which makes a direct comparison difficult, but these results demonstrate that the SRP may have higher yields than the MRP system.

In MRP, outflow loading of TSS ranged from 8.8 kg to 164.2 kg (Table 11). In SRP, TSS loading ranged from 17.3 kg to 208.9 kg (Table 12). The highest overall loading occurred from event 1 in SRP (Table 12). For both ponds, the individual event loading in was on average higher than the loading out (Figure 28A), and the total of all the loads in and out revealed TSS removal in both ponds (Figure 28B). In MRP, the event removal efficiency of TSS was relatively consistent, and ranged from 69.4 % to 91.0 %. The average event removal was 83.9 %, and the removal from the sum of the total loads in and out was 87.7 % (Table 13). The highest inflow loadings normally occurred from the inflow pipe with the largest input of water. SRP showed varying results, with event removal efficiencies ranging from –79.1 % to 77.2 %. The average TSS removal was 5.3 %, and the removal from the sum of the total loads was 18.5 % (Table 14). In SRP, the least intense rainfall resulted in the greatest removal efficiencies. Results from the yields calculations reveal that SRP may have greater yields than MRP (Figure 29). A general increase in yields corresponding to rainfall amount was observed, however more data would be needed to verify this.

In MRP, the outflow loadings of TP ranged from 0.1 kg to 1.8 kg, and generally corresponded with amount of rainfall (Table 11). In SRP, the outflow loadings of TP ranged from 0.1 kg to 5.2 kg, and also corresponded with rainfall amount (Table 12). For both ponds, the individual event loading in was higher on average than the loading out (Figure 30A). However, the total load pollutant removal efficiency was greater in MRP (positive removal) than SRP (negative removal) (Figure 30B). In MRP, TP displayed fairly consistent event removal efficiencies, ranging from 62.3 % to 76.5 % (Table 13). The average of the event removals and the removal from the sum of the total loads for all events were similar at approximately 70 % removal (Table 13). In SRP, event removals ranged from –122.5 % to 38.7 % (Table 14). Both the average event removals and the removal from the sum of the total loads were negative, at -24.4 % and -5.5 %, respectively (Table 14). Results from the event yield calculations reveal that SRP appeared to have greater yields than MRP, and that the yield was positively correlated with rainfall amount (Figure 31).

In MRP, outflow loadings of TDP ranged from 0.04 kg to 0.62 kg (Table 11). Except for event 5, orthophosphate comprised 50 % or less of the total load of TDP. In SRP, the outflow TDP loadings ranged from 0.09 kg to 4.32 kg (Table 12). Orthophosphate comprised 97 % or more of the TDP load in the outflows for all events in SRP.

In MRP, the outflow loading of TN ranged from 1.4 kg to 33.5 kg (Table 11). In SRP, the TN outflow loading ranged from 1.0 kg to 10.4 kg (Table 12). For both ponds, the individual rain event TN inflow loading to the pond was on average higher than the outflow loading to the receiving estuary (Figure 32A); however, MRP displayed a removal of TN before discharge to the receiving water, while SRP displayed a greater loading to the receiving water (Figure 32B). In MRP, event removals efficiencies ranged from 18.1 % to 63.2 %. The average event removal was 44.2 % and the removal from the sum of the total loads was 39.2 % (Table 13). In SRP, the event removal efficiencies ranged from –41.3 % to 16.3 %. The average event removal was –5.0 %, and the removal from the sum of the total loads was –2.5 % (Table 14). Results from the event yield calculations revealed TN yields from the pond drainage basins were variable surrounding rain events, but that SRP generally seemed to have higher yields than MRP (Figure 33).

Discussion

Coastal South Carolina is known for its abundance of tidal creeks and salt marshes, and stormwater ponds are the predominant BMP utilized in this region to protect these waterbodies. In general, stormwater ponds are designed to reduce runoff rates, retain water, and reduce pollutant loadings to the receiving water, primarily through sedimentation and retention of the first flush of runoff. However, little is known about the efficacy and the long-term effectiveness of stormwater ponds, particularly in coastal South Carolina (Lewitus et al. 2003, Libes and Bennett 2004, Serrano 2005, Brock 2006, Drescher unpublished data). In SC, permitting requires that stormwater ponds: (1) reduce post-development runoff rates to the pre-development rates for the designed 2 and 10 year, 24 hour storm events; (2) retain the first 12.7 mm (½ inch) of runoff and slowly release it over 24 hours at a non-erosive flow rate; and (3) maintain an 80 % reduction of TSS during the construction phase of development (SCDHEC 2002). The USEPA has recently mandated changes in how stormwater is permitted in each state (USEPA 1999a). For example, SC is mandating that large municipalities review stormwater permits for development activities at the local level instead of at the state level in order to comply with the National Pollutant Discharge Elimination System (NPDES) Permit requirements (SCDHEC 2006). Therefore, the importance of studying the effectiveness of these stormwater ponds is particularly relevant to provide to both state and local entities.

Pond Storage Capacity

Stormwater ponds are designed to retain 80 % of the sediment load during construction and presumably maintain that function afterwards. Therefore, they are expected to fill in over time, and periodic dredging (i.e., sediment removal) is necessary to maintain their designed storage capacity. Sedimentation can be a problem because it can clog inlets and outlets, pollute downstream water bodies, and reduce the storage volume (Lindsey et al. 1992). A suggested maintenance schedule of dredging every 10 years is recommended by the state (SCDHEC 2000); however, this recommendation was not based on local information. In general, ponds are only dredged either when a local flooding problem occurs, or for aesthetic reasons (H. Repik, personal communication).

This study observed sedimentation within ponds, with storage capacities decreasing by 15.1 % and 36.2 % for MRP (~5-7 years old) and SRP (~5-7 years old), respectively. The observed decrease of storage capacity in these ponds appeared to be primarily at the inflows, and from a calculated decrease in side slopes over time. An unexpected result was the maintenance of the maximum depth in the center of both ponds. Therefore, future surveys of stormwater ponds should measure depth along several cross sections to ensure that the change in storage capacity is accounted for. These cross sections should include the areas around the inflow pipes to ensure that an overestimation of storage capacity is not concluded from a single center depth. In addition, a measure of the sediment within the pipes themselves should also be conducted considering that a number of pipes were observed in this study with approximately half of the pipe full of sediment, which may lead to flooding issues and changes in hydrology.

The current storage capacity in MRP (the terminal pond) decreased less than SRP, and was probably the result of the measured pond being the last pond in a series of ponds. Therefore, a large proportion of the sediment load was probably deposited in the previous ponds, although a bathymetry survey of those ponds was not performed. This indicates that ponds in series may retain more sediments, as well as some sediment associated pollutants (see below).

Pipe location and angle into the pond are important design features of stormwater ponds that influence the hydraulics and flow patterns within them (Pettersson et al. 1998, Walker 1998). The location and angles of the inflow pipes in MRP and SRP may have contributed to greater sediment buildup near them. The findings of this study support previous research that conclude that pipe location should maximize the length to width ratio of the pond, thereby increasing settling time and displacing sediment gradually (Bloesh 1995, Lawrence et al. 1996, Mallin et al. 2002).

Another factor that may influence sedimentation rate within a pond is the amount of vegetation around the edges. The contours around the edge of MRP were tighter than those of SRP, indicating a steeper slope and less change from the designed bathymetry. The 1-2 meters of vegetation along the banks of MRP may explain why the 2:1 slope was maintained better when compared SRP, which had no vegetation. Vegetation may enhance edge stability by minimizing the slumping of the pond edges, thereby slowing the surface and subsurface flow into the pond. Although the use of vegetative buffers has not been studied to this effect, they are commonly suggested as a BMP to minimize the effect of non-point source pollution from development on receiving water bodies (SCDHEC 2000, Mallin et al. 2002, SCDHEC 2003, Vandiver-Hayes 2005). As such, their use around the edges of ponds may reduce sedimentation within the ponds, while also having water quality benefits.

It is unknown if the majority of the sedimentation entered the ponds during the construction phase of development, or if it was from the gradual deposition over time. Until recently in SC, contractors were not required to submit as-built surveys after construction to prove that the pond was constructed and reestablished post-construction at the designed dimensions (SCDHEC 2006). This knowledge will enable regulators to confirm project depth is adequate before the ponds are deeded to the homeowner associations, or other responsible parties.

Runoff Rates

This study also examined the hydrology of wet detention ponds. Two of the requirements for SC stormwater management are that post-development runoff be equal to or less than the pre-development runoff rates, and that the first 12.5 mm (1/2 inch) of runoff is slowly released over a 24-hour period. Although this objective was not completely met due to the inability to directly relate measured events with the state requirements, this study demonstrated some interesting findings. The outflow hydrographs revealed that both ponds were still discharging after 24 hours and that the stormwater in SRP discharged faster than the stormwater in MRP. In SRP, 75 % of the water was found to have exited within 6 to 8 hours. In MRP, 75 % of the water discharged between 13 and 25.5 hours. The multiple pond system (MRP) was effective at slowing the rate of inflow and subsequently slowing the outflow rate. Still, both ponds have hydrographs that were similar to typical urban hydrographs rather than forested/undisturbed hydrographs in that the flow rates peaked and declined relatively quickly instead of slowly rising, peaking and slowly falling (Smith 2005). This suggests that the ponds may not be completely meeting the intentions of the state requirements.

There were a few interesting points about the engineering plans for these ponds pre-development. The pre-development conditions were calculated based on the total drainage area as depicted by the post-development plans. This drainage area was then divided into smaller sub-basins where peak rates of flow were determined for the individual sub-basins and then summed together. For example, the drainage basin for MRP had 9 sub-basins pre-development, and each had a calculated peak rate of flow associated with it. These peak rates of flow for a 28.39 ha drainage basin summed to 1,496 Lps for the 2-year, 24-hour storm event (116.8 mm of rain over 24 hours). This intensity equates to 4.87 mm/hr, which is around the second lowest intensity rainfall that occurred during the time of sampling for this project. For comparison, Smith (2005) measured a peak rate of flow from Old House Creek (forested watershed, 7% impervious cover) in the Charleston, SC area to be 1,332.19 Lps from a 279.1 ha watershed during a 109.7 mm rain event, which occurred over 48 hours. Therefore, the smaller watershed investigated in this studywith a similar rainfall amount was calculated by the engineers to result in a slightly larger peak flow in comparison to results measured from a watershed that was 10 times larger. This discrepancy indicates the peak flow calculations of pre-development may overestimate actual peak discharges.

Furthermore, the post-development runoff for the MRP watershed was calculated by the engineers to be 1,116 Lps for the same 2 year, 24 hour storm. This number was the culmination of all discharges to the boundary, the main two being the outflow from MRP and the emergency weir in MRP, with the weir calculated to have the greatest amount of discharge. Although there were no rain events that approached 116.8 mm, MRP had rain events around 25.4 mm that were of greater intensity and produced peak rates of flow that were 2 orders of magnitude lower than both those calculated for pre- and post-development. For example, event 2 of MRP produced a peak rate of flow of only 39 Lps, and was 3 times more intense than the 2-year designed event. No water ever left the emergency weir during any of the sampled events at MRP. Although a direct comparison was not possible, this information should be explored further to determine if the model calculations being used are applicable in the low topography of South Carolina.

Hydraulic Functioning

Along with the determination of runoff rates, automatic samplers were deployed to obtain the volumes of water entering and exiting each pond. These data were used to develop a water budget and ultimately determine the hydraulic functioning of the ponds and verify the volumes used for the loads into and out of the ponds. However, there were some difficulties that presented themselves during this project, including the inability to measure all inflows to the pond and equipment failures. One cause of unreasonable flow measurements from the automatic samplers was due to the submersion of the inflow pipes upon entry to the ponds. In most instances, the flow rates measured in inflow pipes mirrored the rainfall intensity of the events. However, low flow rates combined with submerged pipes resulted in erratic readings. Comings et al. (2000) noted similar difficulties in an efficiency study of wet ponds in Bellevue, Washington. To counter this problem, they used a mass balance determined from other measurements to and from the ponds in order to calculate the flow from that source. Outflows from the two ponds in this study were substantially easier to sample. Invert elevations of the outflow pipes were above the normal water elevation, which equated to laminar flow when the pond level raised enough from a rain event.

Due to these reasons, estimates were calculated for certain inflows and for sheetflow. The three methods that were utilized to estimate inflows and sheetflow were the SCS TR 55 method using actual precipitation data, the SCS TR 55 method using 24-hour fixed precipitation, and the Simple Method. The most appropriate method for use in this project was determined to be the Simple Method because of the consistency of results.

The application of the SCS TR 55 method was more simplified in this project than the method used by stormwater engineers in modeling stormwater ponds; however, some common problems that were encountered by this study should be mentioned. The SCS TR 55 method uses regional rainfall time distributions based on 24-hour rainfall amounts because most weather data is given based on 24-hour increments (USDA 1986). USDA (1986) advises that results for actual storms should be used with caution, because the runoff curve number (CN) equation does not have an expression for time and does not take into account rainfall duration and intensity. Additionally, the CN procedure is less accurate when rainfall is less than 0.5 inches (USDA 1986), which occurred commonly during the sampling of MRP.

The development of the water budgets resulted in a percent difference between the outflow volume and the inputs to the pond. Evapotranspiration was assumed to be negligible for each event. However, over a long-term budget, evapotranspiration would be an important component to consider (Mitsch and Gosselink 2000). The percent differences observed in this study were most likely attributable to either sampling error, inaccuracies in runoff calculations, or from groundwater interactions. Groundwater interactions have not been considered in past research on stormwater pond efficiency (Wu et al. 1996, Borden et al. 1998, Pettersson et al. 1999, Comings et al. 2000, Mallin et al. 2002). In the case of consecutive events, such as MRP events 3, 4, and 5, which spanned 4 days, groundwater interactions are likely to be higher. These three events could have raised the water table enough to result in groundwater discharge into the pond. Over the course of a single rain event, groundwater may not significantly influence a pond; however, it is possible that consecutive rain events over a short period of time could contribute to a pond water budget (T. Callahan personal communication).

Water Quality and Pollutant Removal

The volume data used in the development of the water budgets was also used for determining the water quality and pollutant removal ability of the ponds. As localized drainage basins become increasingly urbanized, the potential for water quality degradation in receiving waters increases due to increased volume and pollutant loadings in stormwater runoff (Harrell and Ranjithan 2003, Holland et al. 2004, Mallin et al. 2004). Utilization of wet detention ponds is a common suggestion for mitigating the impacts from non-point source runoff from these increasingly urbanized landscapes (SCDHEC 2000, SCDHEC 2001). These wet detention ponds serve a dual purpose of controlling runoff and removing pollutants (Wang et al. 2004). Previous research has shown that these ponds are susceptible to water quality issues arising from nutrient and bacteria loading (Bannerman et al. 1993, Puckett 1994, Lewitus et al. 2003, Mallin et al. 2004). In coastal SC, Lewitus et al. (2003) and Brock (2006) found that stormwater ponds have inadvertently created the potential for harmful algal bloom development and thereby create a potential human health hazard.

The physical parameters and various pollutants sampled during this study will be discussed as they pertain to water quality, pollutant loadings and/or efficiencies. Loadings were determined from composite sampling and the impacts on receiving waterbodies are particularly important. Previous research has demonstrated that the application of an event mean concentration to the total of all the event volumes provides the most reasonable long-term estimate of removal efficiency (Wu et al. 1996, Comings et al. 2000). This study determined the removal efficiencies for the sampling time period and thus applied the flow-weighted event concentrations to the volumes from each event. Based on the large event variability observed in this study, removal efficiencies obtained from the sum of the total loads across all events provided the most appropriate comparison.

Physical Parameters

The semi-continuous measure of physical parameters was performed to understand changes in basic water quality surrounding rain events. The average temperatures observed in this study were similar to results obtained in other studies of stormwater ponds in coastal South Carolina (Serrano 2005, Brock 2006). Thermal pollution from stormwater has been found to be a concern in freshwater streams (Roa-Espinosa et al. 2003); however, this study did not observe higher temperatures in stormwater ponds following rain events. In fact, temperatures decreased following rain events, resulting in cooler waters entering the receiving waters. A study of 23 tidal creeks draining a variety of land use types across South Carolina found no impacts from thermal pollution (Holland et al. 2004). Receiving waters in other regions of the State that are fed from mountain streams, have large trees which provide shade, or have deeper water, might be more impacted by thermal pollution from stormwater ponds.

Dissolved oxygen is important for organisms and maintaining reasonable levels is important to receiving water bodies (Bricker et al. 1999). MRP had a higher mean DO concentration than SRP, which was probably due to the extensive vegetation around the pond. In addition, DO concentrations decreased slightly after rain events, which may have been the result of oxygen consuming organic matter entering the ponds during the rain events (Taebi and Droste 2004). In extreme circumstances, this lower DO discharging from the pond may have an adverse impact on estuarine organisms (Bricker et al. 1999, Taebi and Droste 2004).

Total Suspended Solids

TSS is often used as a regulatory metric for stormwater ponds due to the strong affinity of pollutants to adhere to sediment (SCDHEC 2003, Jeng et al. 2005). In South Carolina, TSS is only regulated during the construction phase of development, when sediment loading is presumed to be highest (SCDHEC 2003). TSS was observed to settle out within 24 hours after rain events, which indicates that they are providing removal over that time frame. Additionally, pond concentrations were relatively low, indicating that ponds were allowing sediment deposition. That said, the mean loading to the receiving waters observed in this study was still high (75 – 101 kg). A study in Bellevue, WA concluded that the annual loading of TSS was 200 kg for a pond that drained a 5 hectare area, and was designed specifically for water quality improvement (Comings et al. 2000). Another pond, draining 40 hectares and designed for flow attenuation, had an annual load of 2,300 kg of TSS (Comings et al. 2000). Outflow loading of TSS has been found to be dependent upon pond area to drainage area, land use type, and pond design (i.e., retention time, length:width, etc.) (Wu et al. 1996, Pettersson et al. 1999, Comings et al. 2000, Somes et al. 2000).

A compilation of the results of numerous studies throughout the United States has found that wet detention ponds have an 80 % median pollutant removal for TSS (National Pollutant

Removal Performance Database 2000). Comings et al. (2000) found TSS removal of 81 % in one pond and 61 % in another. Similarly, Pettersson et al. (1999) found TSS removal of 70 % and 84 % in two different ponds. The findings of this study, particularly with MRP (88 %) are in agreement with these studies; however, SRP had a lower removal efficiency of only 19 %. There are many reasons that could explain why SRP was less efficient than MRP. SRP discharged stormwater faster than MRP, which may adversely impact pond hydrodynamics and result in the discharge of water before sediment has time to settle out. Additionally, sediment resuspension may have occurred after stormwater entered the pond. This study also observed that SRP had larger sediment plumes around its inflow pipes than MRP.

Event yields of TSS were higher from SRP than MRP, which means that more sediment per hectare was leaving the SRP watershed, which results in higher loading per hectare entering the receiving water from SRP compared to MRP. TSS yields were similar to yields obtained by Smith (2005) in suburban and urban watersheds in the Charleston, SC region (average yield = 5.5 kg/ha). A forested watershed, however, resulted in lower yields than the ponds in this study (average yield = 0.92 kg/ha). This indicates that urbanization and increasing impervious cover will increase the amount of sediment discharging to the receiving water regardless of implementation of BMPs.

Even though the concentration of TSS within the ponds might be minimal, the defined source of flow resulting from the outflow pipes might be enough to cause the re-suspension of sediments in the tidal creeks to which these ponds drain. Marsalek et al. (2002) determined that stormwater runoff could contain high concentrations of TSS, and even higher concentrations in the creek water due to sediment and channel erosion. The resuspension of sediments could also cause the release of bound pollutants (i.e., fecal coliforms, and nutrients). USEPA has recommended that regular maintenance and removal of accumulated sediment is important to prevent the resuspension of trapped sediments (USEPA 1999b).

Fecal Coliform Bacteria

Fecal coliform bacteria originate from warm-blooded animals and are used as an indicator organism for bacterial pollution (Burkhardt et al. 2000). The concentrations of these bacteria in tidal creeks have been found to be strongly correlated with the occurrence of rainfall (Mallin et al. 2002, Jeng et al. 2005). Fecal coliform bacteria concentrations in this study were observed to significantly increase 24 hours after rain events followed by a decline 48 hours afterwards. Jeng et al. (2005) also found that an effective reduction of fecal bacteria was observed within 24 to 48 hours following a rain event.

In the study ponds, fecal coliform bacteria concentrations prior to rain events were below the SCDHEC recreational contact limit for similar water bodies. However, the mean outflow loading to the receiving waters for MRP and SRP was 2.41×10^{10} colonies and 2.02×10^{10} colonies, respectively. These results indicate that while background concentrations might be relatively low, the loading of bacteria to the receiving waters following rain events is a concern. The reduction of bacteria has been shown to result from settling of the bacteria into sediment (Ketchum et al. 1952, Jeng et al. 2005). This raises concern for the potential of resuspension of bacteria from a high inflow of stormwater, or from disturbance from wildlife or human recreation. Grimes (1975) demonstrated that sediments could serve as a reservoir for bacteria, and that disturbance such as increased flow can re-suspend these sediments. The potential re-suspension of contaminated sediment is a water quality concern that may impact the loading of bacteria to the receiving water bodies.

Although outflow loadings were high, both ponds were found to be effective at reducing fecal coliform bacteria from the loading into the pond. MRP (55 %) displayed higher total load removal efficiency than SRP (14 %), but both ponds displayed event variability. Mallin et al. (2002) found the variability of mean fecal coliform concentrations between inputs and outputs of three ponds ranged from –15 % to 86 %, and varied between events as well. Causes of event variability could be from differing storm dynamics and pond water conditions, and/or resuspension of sediments. Many types of birds and waterfowl utilize these ponds as a source of food and/or refuge, which could also contribute to variability in efficiencies. These factors could have contributed to two events (1 in MRP and 1 in SRP) that had large negative removal efficiencies indicating a source of bacteria in the ponds. Similarly, Borden et al. (1998) found that while one wet detention pond in NC typically reduced fecal coliform bacteria by 70 % to 90 %, the annual average removal was significantly lower (48 %) because of one high fecal coliform measurement in the outflow during one event.

While the ponds were generally effective at reducing some fecal coliform bacteria, they still resulted in relatively large yields. Smith (2005) determined that fecal coliform yields were significantly lower in an undeveloped watershed than in suburban or urbanized watersheds. Results from this study found that yields from MRP and SRP were similar (average yield = 10^9 colonies/ha) to yields from the suburban and urban creeks in the watershed scale study by Smith (2005). This is relevant because the creeks studied by Smith (2005) contain older developments with some areas containing few BMPs, and the yields from the SCDHEC-OCRM permitted ponds (MRP and SRP) were similar to or larger than the yields from those older watersheds.

Nutrients

Overall inorganic nitrogen concentrations were low, and no major pattern was noticed surrounding rain events. Serrano (2005), Brock (2006), and Drescher (unpublished data) found similar low inorganic nutrient concentrations in coastal South Carolina stormwater ponds. Low concentrations of inorganic nitrogen constituents are not uncommon as inorganic nitrogen constituents are typically the first nitrogen constituents taken up by primary producers (Lewitus et al. 2004). The higher concentrations of TN indicate that the majority of nitrogen in the stormwater ponds was in the organic form. In stormwater ponds on Kiawah Island, Lewitus et al.

(2004) found that concentrations of dissolved organic nitrogen were routinely in excess of 1.6 mg/L and were the major fraction of nitrogen. Results of this study indicate that the loadings of nitrogen into the receiving waters consisted primarily of organic nitrogen (Tables 11 and 12). In MRP, the removal efficiency of TN was 39 %, while TDN removal was 43 %. SRP was less efficient in nitrogen removal, and had –2 % removal of TN and 12 % removal of TDN. Previous research has found similarly low nitrogen removal in stormwater ponds (Wu et al. 1996, Borden et al. 1998 and Pettersson et al. 1999). Borden et al. (1998) found that high nitrogen removal during the growing season was associated with biological uptake; however, annual nitrogen removal was low because of nitrogen release during fall turnover.

Concentrations of phosphorus parameters were relatively low when compared to other local studies (Lewitus 2004, Serrano 2005, Brock 2005, Drescher unpublished data). Yet, for comparison, the average concentration of TP was higher than SCDHEC guidelines for similar water bodies. Phosphorus parameters also displayed variability between ponds. MRP displayed high removal efficiency for TP, TDP and PO_4^{3-}, at 71 %, 70 % and 88 %, respectively. SRP discharged more phosphorus than was input to it, and had removals of –6 %, -5 % and –3 % for TP, TDP and PO_4^{3-}, respectively. The reason for the negative removals in SRP could be because of the relatively low inflow loading to the pond. Schueler (1996) noted the concept of an "irreducible concentration", which means the concentration of a pollutant cannot be lower than in a natural system and can ultimately limit the reduction of a pollutant from a pond.

There are multiple reasons that could explain why nutrient concentrations are relatively low in these ponds. Sampling occurred in mid to late summer when the wide spread application of fertilizers is not recommended. Despite this, Brock (2006) found that the ponds on Kiawah Island displayed no seasonality in their nutrient concentrations. Additionally, it should be noted that the drainage areas for these ponds were primarily residential, and ponds draining other land uses (i.e., golf courses, commercial, and industrial) might behave differently. Although, the nutrient concentrations were low in both ponds, the yields obtained from MRP and SRP were substantially higher than yields from suburban and urban creeks in Charleston, SC (Smith 2005). This indicates that wet detention ponds could be a contributor of non-point source nutrient pollution to these receiving water bodies.

The variability of removal efficiencies for the nutrient parameters indicates that pond interactions may significantly alter the stormwater concentrations for these various parameters. Libes (1999) noted the effect of these pond interactions in a study of a retrofit of a dry detention pond into a pond/wetland system in Conway, SC. She noted removal efficiency ranges of 42 % to 96 % for nitrate + nitrite, 23 % to 97 % for orthophosphate, and 62 % to 95 % for ammonium for a pond/wetland system. These numbers indicate high variability in a system designed for water quality. This high variability is likely due to biological uptake and varying hydrological conditions. MRP probably displayed greater nutrient removal when compared to SRP because of the extent of aquatic vegetation around it and the greater amount of vegetation around its edges.

Recommendations

Future Studies

The results of this study provide insight into designing future studies relating to stormwater ponds. Studies relating to depth and volume of stormwater ponds should focus on measuring depth along transects from the pond banks to the point where the depth levels off, as well as obtaining depth in the center of the pond. Specific attention should be given to locations around the inflow pipes. Future research could also focus on stormwater pond sediment quality and actively search for uses of the sediment, such as fill material for construction sites or for new infrastructure projects.

Further research into stormwater pond efficiencies should be continued in the low topography of South Carolina. This work should evaluate ponds of varying ages, as well as the effectiveness of multiple BMPs. Also, the methods and equipment used in this project presented some difficulties in capturing rain events. Research on stormwater pond efficiencies should consider the location of the inflow pipes and determine a course of action if pipe location will prohibit laminar flow (i.e., submerged). The equipment used in this study was not designed to measure flow in completely submerged conditions. Additionally, when determining removal efficiency, the use of the total load removal provides the most insight into a pond's long-term effectiveness.

The site-specific nature of stormwater management assumes that pre and post-development runoff from sub-watersheds results in an additive affect. This research found that current methods might grossly overestimate both pre- and post-development runoff. Future studies should address actual runoff from an undeveloped site, prior to commencement of construction activities in order to determine if the designed methods are accurately configuring the runoff from the site.

Stormwater Management

A number of important implications for stormwater managers are apparent. Results of this study indicate that the use of additional BMPs in addition to, and in conjunction with, stormwater ponds is warranted given the relatively low pollutant removals. Additionally, water quality is a concern within stormwater ponds because it can reduce the aesthetic appeal of the ponds and because of the potential for adverse effects on receiving waters. Issues related to water quality stemming from stormwater ponds are not commonly addressed in stormwater management plans. This study demonstrates that pollutant loadings from stormwater ponds can be relatively high. Drescher (unpublished data) sampled a wide variety of stormwater ponds throughout the coastal zone of South Carolina. These data should be used to determine if a monitoring program should be developed to obtain water quality standards for these stormwater ponds, or for the stormwater loadings to receiving waters. As a greater percentage of our watersheds become developed, stormwater pond water quality will undoubtedly impact the estuarine environments into which they discharge.

Stormwater pond inspections should continue on a regular basis in the future, and should be evaluated at least once every 5 years. The USEPA recommends that bottom sediments be removed about every 2 to 5 years in order to maintain a pond's storage capacity (USEPA 1999). Inspections of these systems ensure that they are functioning properly and also place responsibility on due parties. Inspectors should quantitatively evaluate pond sedimentation rates (see above), while also measuring the amount of sedimentation within the pipes. Inflow 2 of MRP had 0.6 m of sediment within it, which comprised about half of the pipe. The clogging of stormwater pipes probably impacts the rate of flow into the pond. It is easy for stormwater inspectors to overlook this aspect of pond maintenance, as many inflow pipes are submerged upon entry to the ponds. Managers should also seek out ways to determine a percentage of volume loss, or some other quantitative measure, to know when to require dredging of stormwater ponds. Contractors should be required to dredge the ponds back to their original design capacity (to some degree of error) and inspectors should enforce this detail.

Presently, stormwater ponds are modeled around a designed storm event. The two-year frequency storm is one that is expected to be equaled or exceeded once every two years, while the ten-year event is expected to be equaled or exceeded only once every ten years (SCDHEC 2002a). These designed events focus the construction of ponds on quantity of runoff volume to control flooding issues that could arise from extreme events. But, it is apparent that designing stormwater BMPs around these unlikely and extreme rain events inevitably ignores the water quality issues that can occur from more frequent rain events.

Additionally, this study found that while these ponds were designed to attain pre-development flow rates, they resulted in hydrographs that were more similar to a typical urban hydrograph rather than a forested hydrograph. With greater numbers of these BMPs being utilized as more land is being developed, the focus of stormwater management should shift toward water quality and the protection of our economically and ecologically valuable estuaries. One way to do this could be by focusing on more frequently occurring rain events. Another could be by updating models that are built around nutrient, sediment, and bacteria removal to locally obtained data.

Stormwater planners should focus on developing watershed-scale management plans that individual construction sites must adhere to. Total Maximum Daily Loads (TMDLs) are required for impaired water bodies, yet more preventative measures should be utilized to maintain the integrity of all watersheds.

Additionally, education and outreach programs should be included in all stormwater management strategies. Specifically, more emphasis should be placed on the ability of individual home owners, commercial businesses and industry to reduce runoff at its source, by utilizing rain gardens (mini wetlands), green roofs, or rain barrels, all of which provide added benefits beyond just stormwater control.

Conclusions

This study found high pollutant loadings occurred from storm events draining through stormwater ponds. While these ponds are designed around large events, results from this study show that even small events can result in high pollutant loading to the receiving waters. Of the two ponds studied, the multiple pond system appears to remove pollutants more effectively than the single pond. However, the demonstrated event variability means that the evaluation of the performance of these detention ponds should be based on the cumulative loads over a number of events. The reason for the better efficiency of MRP probably relates to the decreased rate of inflow, the larger length to width ratio, and the greater extent of aquatic vegetation. This study also found that outflows from the two ponds generally resemble an urban hydrograph, even though their design is such that the outflow hydrograph is intended to resemble pre-development conditions.

As more and more stormwater outflow pipes are appearing in receiving waters, they are turning what was a diffuse, non-point source of pollution into more of a point source problem. While pollutants may still ultimately come from indiscriminate locations, they are collected, stored, transformed and ultimately discharged at explicit points. The inevitable increase of development along the South Carolina coastal zone ensures that stormwater management will be an integral tool in addressing the longevity and health of our coastal ecosystems. This drastic transformation of the coastal landscape needs to be managed on a watershed scale, rather than site-specific projects. Stormwater management should also focus on water quality issues associated with BMPs and provide engineers with the ability to creatively utilize landscapes to implement comprehensive stormwater plans, rather than focusing on numbers derived from models of theoretical rain events.

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TABLES

Table 1. Summary of SCDHEC water quality standards for freshwater bodies of water. SCDHEC does not have standards for stormwater pond water quality and the standards for freshwater will be used as guidelines for stormwater ponds.

Parameter	Criteria	Water Quality	Source
pH	6.0 to 8.5	normal	SCDHEC, regulations 61-68
Dissolved Oxygen	daily average >= 5 mg L^{-1}, low of 4.0 mg L^{-1}	standard	SCDHEC, regulations 61-68
	2.0 to 5.0 mg L^{-1}	biologically stressed	Bricker et al. 1999
	<2.0 mg L^{-1}	hypoxic	
	0 mg L^{-1}	anoxic	
Total Nitrogen	1.50 mg L^{-1}	Not to exceed	SCDHEC, regulations 61-68
Total Phosphorus	0.09	Not to exceed	SCDHEC, regulations 61-68
Fecal Coliform Bacteria	200 cfu	Monthly geometric mean not to exceed	SCDHEC, regulations 61-68
	400 cfu	Daily max not to exceed	SCDHEC, regulations 61-68

Table 2. Constituent analysis methods, holding times and preservatives are summarized for each parameter. EPA = Environmental Protection Agency, SM = Standard Methods for the Evaluation of Water and Wastewater 18th Edition.

Parameter	Bottle Type	Preservative	Method	Holding Time
TSS	900 mL plastic	none	EPA 160.1	7 days
FC	100mL plastic	NS	SM 9221E	6 hrs
NO_{2+3}^{-}	900 mL plastic	H_2SO_4	EPA 353.3	48 hrs
NH_4^{+}	900 mL plastic	H_2SO_4	EPA 350.3	7 days
TKN	900 mL plastic	H_2SO_4	EPA 351.3	7 days
TP	900 mL plastic	H_2SO_4	EPA 365.2	7 days
PO_4^{3-}	900 mL plastic	none	EPA 300.0	48 hrs

Table 3. Land cover (hectares and impervious cover (%)) data is provided for each drainage basin and sheet flow (NPS) into MRP. Whole is the sum of the entire drainage basin. Refer to Figure 2 for a diagram of drainage basins.

	MRP: Drainage Basin				
	Inflow 1	**Inflow 2**	**Inflow 3**	**NPS**	**Whole**
Concrete	1.94	0.63	0.37	0.33	3.27
Landscaped	11.41	2.58	0.79	0.65	15.43
Ponds	0.49	0.00	0.00	0.00	0.77
Structure	2.65	1.30	0.24	0.12	4.30
Road	2.17	0.66	0.00	0.00	2.83
Undisturbed	1.22	0.00	0.07	0.47	1.76
Total	19.87	5.16	1.46	1.57	28.35
% Impervious Cover	36.47	50.12	41.37	28.29	39.38

Table 4. Land cover (hectares and impervious cover (%)) data is provided for each drainage basin and sheet flow (NPS) into SRP. Whole is the sum of the entire drainage basin. Refer to Figure 3 for a diagram of drainage basins.

	SRP: Drainage Basin			
	Inflow 1	**Inflow 2**	**NPS**	**Whole**
Concrete	0.15	0.27	0.00	0.42
Landscaped	0.48	0.85	0.23	1.56
Pond	0.00	0.00	0.00	0.14
Structure	0.15	0.29	0.02	0.46
Road	0.17	0.37	0.00	0.54
Total	0.95	1.79	0.26	3.14
% Impervious Cover	49.58	52.55	9.39	50.28

Table 5. **R**ainfall events sampled in each pond for this study. 1 in. = 25.4 mm.

Pond	Event #	Date	Antecedent Rainfall (days)	Rainfall (mm)	Duration (hr)	Intensity (mm/hr)
MRP	1	6/3/2006	6	26.92	1.75	15.39
	2	7/6/2006	9	23.37	1.83	12.77
	3	7/26/2006	1	8.89	0.17	52.29
	4	7/27/2006	0	5.59	1.00	5.59
	5	7/29/2006	1	16.51	1.33	12.41
SRP	1	8/31/2006	4	85.34	16.92	5.04
	2	9/6/2006	5	26.67	4.08	6.54
	3	9/13/2006	5	16.51	6.92	2.39

Table 6. Semi-continuous water quality data summary for each pond.

Pond		Temp (^oC)	pH	DO (mg/L)	Salinity (ppt)
MRP	Average	29.34	6.94	8.13	0.01
	Std. Dev.	1.32	0.29	2.08	0.01
	Max	32.13	8.14	13.12	0.09
	Min	26.14	6.44	3.66	0.00
SRP	Average	27.88	7.16	4.57	3.58
	Std. Dev.	1.34	0.42	1.64	2.07
	Max	32.69	8.69	8.68	5.65
	Min	24.35	6.41	0.92	0.00

Table 7. Hydrographic data obtained for each event. Values denoted as "n/a" are present when data were not valid, did not pass QA/QC, or if a composite wasn't applicable due to only one sample being triggered or if the composite was not flow-weighted.

Pond	Event #	Watershed	Volume (L)	Peak Flow (Lps)	75 % Discharge (hours)	24-hr Discharge Volume (L)
MRP	1	Inflow 1	n/a			
		Inflow 2	465671			
		Outflow	2052006	39	25.5	1486873
	2	Inflow 1	531748			
		Inflow 2	436262			
		Outflow	1474781	24	24.5	1091791
	3	Inflow 1	no flow			
		Inflow 2	130715			
		Outflow	217799	6	13	n/a
	4	Inflow 1	no flow			
		Inflow 2	173077			
		Outflow	220819	3	28.5	151614
	5	Inflow 1	44371			
		Inflow 2	208596			
		Outflow	786407	22	18	666539
SRP	1	Inflow 1	266166			
		Inflow 2	1103579			
		Outflow	1394155	91	6	1388161
	2	Inflow 1	n/a			
		Inflow 2	230534			
		Outflow	370817	21	6.75	n/a
	3	Inflow 1	n/a			
		Inflow 2	n/a			
		Outflow	147115	6	8	146384

Table 8. Calculated runoff values expressed in liters for each drainage basins of each pond. Italicized values are the fraction of runoff from the amount of rain that fell within each watershed, which can be converted into a percent. Methods shown are the Soil Conservation Service TR 55 Method (SCS TR 55) with actual precipitation and the 24-hour fixed precipitation and the Simple Method.

Pond	Event #	Watershed	SCS (actual P)		SCS (24-hr. fixed)		Simple Method	
			Runoff Volume	Rainfall fraction	Runoff Volume	Rainfall fraction	Runoff Volume	Rainfall fraction
MRP	1	Inflow 1	425,745	*0.08*	4,324,617	*0.81*	2,023,276	*0.38*
		Inflow 2	220,468	*0.16*	1,188,358	*0.85*	696,572	*0.50*
		Inflow 3	6,508	*0.02*	290,228	*0.74*	166,567	*0.42*
		NPS	10,084	*0.02*	316,002	*0.75*	128,515	*0.30*
		Whole	694,351	*0.09*	6,233,296	*0.82*	3,086,662	*0.40*
	2	Inflow 1	238,002	*0.05*	3,583,315	*0.77*	1,756,051	*0.38*
		Inflow 2	145,208	*0.12*	995,208	*0.82*	604,572	*0.50*
		Inflow 3	1,579	*0.00*	236,296	*0.69*	144,568	*0.42*
		NPS	3,294	*0.01*	258,129	*0.70*	111,541	*0.30*
		Whole	402,832	*0.06*	5,174,829	*0.78*	2,678,990	*0.40*
	3	Inflow 1	72,016	*0.04*	1,636,397	*0.93*	668,063	*0.38*
		Inflow 2	1,330	*0.00*	432,590	*0.94*	230,000	*0.50*
		Inflow 3	20,419	*0.16*	117,148	*0.90*	54,999	*0.42*
		NPS	18,341	*0.13*	126,006	*0.90*	42,434	*0.30*
		Whole	79,145	*0.03*	2,342,016	*0.93*	1,019,181	*0.40*
	4	Inflow 1	216,273	*0.19*	640,683	*0.58*	419,925	*0.38*
		Inflow 2	22,150	*0.08*	191,754	*0.66*	144,571	*0.50*
		Inflow 3	35,643	*0.44*	37,381	*0.46*	34,570	*0.42*
		NPS	33,906	*0.39*	41,784	*0.48*	26,673	*0.30*
		Whole	271,258	*0.17*	937,850	*0.59*	640,628	*0.40*
	5	Inflow 1	22,113	*0.01*	2,503,283	*0.76*	1,240,688	*0.38*
		Inflow 2	38,534	*0.05*	696,482	*0.82*	427,143	*0.50*
		Inflow 3	1,877	*0.01*	164,594	*0.68*	102,140	*0.42*
		NPS	907	*0.00*	179,901	*0.70*	78,806	*0.30*
		Whole	50,498	*0.01*	3,616,271	*0.77*	1,892,764	*0.40*
SRP	1	Inflow 1	425,694	*0.53*	512,069	*0.64*	401,030	*0.50*
		Inflow 2	832,122	*0.55*	991,239	*0.65*	796,546	*0.53*
		NPS	66,781	*0.31*	93,298	*0.43*	29,731	*0.14*
		Whole	1,427,760	*0.54*	1,711,688	*0.64*	1,345,164	*0.51*
	2	Inflow 1	38,575	*0.15*	175,647	*0.70*	125,322	*0.50*
		Inflow 2	81,304	*0.17*	338,119	*0.71*	248,921	*0.53*
		NPS	1,070	*0.02*	34,796	*0.51*	9,291	*0.14*
		Whole	132,817	*0.16*	586,006	*0.70*	420,364	*0.51*
	3	Inflow 1	6,766	*0.04*	62,399	*0.40*	77,580	*0.50*
		Inflow 2	16,067	*0.05*	123,891	*0.42*	154,094	*0.53*
		NPS	332	*0.01*	7,579	*0.18*	5,751	*0.14*
		Whole	24,315	*0.05*	210,413	*0.41*	260,225	*0.51*

Table 9. Water budget volumes measured in MRP. Percent differences are the unaccounted for water divided by the water measured leaving the pond. Negative differences mean more water left the pond and positive differences mean more water entered the pond.

MRP: Water Budgets							
Source	**Event 1**	**Event 2**	**Event 3**	**Event 4**	**Event 5**	**3,4,5 combined**	**All combined**
Storage	0	0	0	0	0	0	0
Inflow 1	1,215,858	531,748	0	0	44,371	44,371	1,791,977
Inflow 2	465,671	436,262	130,715	173,077	208,596	512,388	1,414,321
Inflow 3	166,567	144,568	54,999	34,570	102,140	191,709	502,844
Sheetflow	128,515	111,541	42,434	26,673	78,806	147,913	387,969
Pond Precip.	75,396	65,438	24,895	15,648	46,233	86,776	227,609
Outflow	2,052,006	1,474,781	217,799	220,819	786,407	1,225,025	4,751,812
Overflow Weir	0	0	0	0	0	0	0
Difference	0	-185,225	35,243	29,150	-306,261	-241,868	-427,092
% difference	0.00	-12.56	16.18	13.20	-38.94	-19.74	-8.99

Table 10. Water budget volumes measured in SRP. Percent differences are the unaccounted for water divided by the water measured leaving the pond. Negative differences mean more water left the pond and positive differences mean more water entered the pond.

SRP: Water Budgets (Liters)				
Source	**Event 1**	**Event 2**	**Event 3**	**All combined**
Storage	133,896	0	0	133,896
Inflow 1	266,166	125,322	77,580	469,068
Inflow 2	1,103,579	230,534	154,094	1,488,207
Sheetflow	29,731	9,291	5,751	44,773
Pond Precip.	122,859	38,393	23,767	185,020
Outflow	1,392,359	370,817	147,115	1,910,291
Overflow Weir	0	0	0	0
Difference	-3,920	32,723	114,078	142,880
% Difference	-0.28	8.82	77.54	7.48

Table 11. Event loadings calculated in MRP for each pipe. Cells containing "n/a" were not reported due to inadequate sample volume to measure all the constituents.

MRP: LOADINGS (kg)														
Event	site	fc (cfu)	TSS	TP	TDP	OP	NO3+2	NH4	TKN	TN	NH4 diss	NO3+2 diss	TKN diss	TDN
1	Inflow 1	1.69E+11	391.26	3.38	1.37	0.95	0.42	n/a	23.26	23.69	1.06	0.21	11.63	11.84
	Inflow 2	2.33E+10	325.97	2.19	0.84	0.79	0.09	n/a	6.52	6.61	1.86	0.09	4.19	4.28
	Inflow 3	1.49E+10	208.14	1.40	0.54	0.51	0.06	n/a	4.16	4.22	1.19	0.06	2.68	2.74
	Outflow	3.49E+09	164.16	1.64	0.62	0.21	0.41	n/a	12.31	12.72	2.05	0.41	10.26	10.67
2	Inflow 1	6.91E+09	127.62	0.96	0.37	0.16	0.11	n/a	9.57	9.68	n/a	0.11	n/a	n/a
	Inflow 2	7.42E+09	885.61	2.22	0.44	0.31	1.79	2.62	16.58	18.37	1.75	1.27	2.62	3.88
	Inflow 3	5.18E+09	618.00	1.55	0.30	0.21	1.25	1.83	11.57	12.82	1.22	0.88	1.83	2.71
	Outflow	1.13E+11	146.73	1.78	0.53	0.15	0.36	5.35	33.09	33.46	0.70	0.29	8.15	8.44
3	Inflow 1	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	Inflow 2	1.44E+09	79.74	0.34	n/a	n/a	0.60	0.13	1.57	2.17	n/a	n/a	n/a	n/a
	Inflow 3	4.06E+08	22.52	0.10	n/a	n/a	0.17	0.04	0.44	0.61	n/a	n/a	n/a	n/a
	Outflow	1.52E+09	10.89	0.13	0.04	0.02	0.04	0.22	1.31	1.35	0.22	0.04	1.31	1.35
4	Inflow 1	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	Inflow 2	2.77E+10	22.50	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	Inflow 3	7.82E+09	6.35	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	Outflow	4.44E+08	8.83	0.08	0.04	0.02	0.04	0.22	1.32	1.37	0.22	0.04	1.10	1.15
5	Inflow 1	n/a	6.66	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	Inflow 2	1.04E+09	244.06	0.73	0.23	0.21	0.31	0.21	3.34	3.65	0.21	0.04	0.63	0.67
	Inflow 3	5.11E+08	119.50	0.36	0.11	0.10	0.15	0.10	1.63	1.79	0.10	0.02	0.31	0.33
	Outflow	2.28E+09	45.51	0.38	0.08	0.08	0.15	0.76	10.62	10.77	0.76	0.15	3.03	3.19

Table 12. Event loadings calculated in SRP for each pipe. Cells containing "n/a" were not reported due to inadequate sample volume to measure all the constituents or, in the case of FC for SRP event 1, because all of the samples collected were >1600 cfu/100mL.

SRP: LOADINGS (kg)															
Event	site	fc (cfu)	TSS	TP	TDP	OP	NO3+2	NH4	TKN	TN	NO3+2 diss	NH4 diss	TKN diss	TDN
	Inflow 1	n/a	45.25	1.12	0.85	0.85	0.21	0.53	2.13	2.34	0.11	0.27	2.13	2.24
1	Inflow 2	n/a	208.50	4.64	3.53	3.53	0.44	1.10	8.83	9.27	0.44	1.10	8.83	9.27
	Outflow	2.23E+10	208.85	5.15	4.32	4.18	0.70	2.78	9.75	10.44	0.56	2.78	9.75	10.30
	Inflow 1	3.45E+08	4.14	0.20	0.18	0.18	0.10	0.14	0.90	0.99	0.10	0.07	0.90	0.99
2	Inflow 2	3.69E+10	39.19	0.67	0.60	0.60	0.32	0.46	3.00	3.32	0.32	0.23	3.00	3.32
	Outflow	1.47E+10	77.60	1.93	1.10	1.10	0.08	1.20	6.01	6.09	0.07	0.92	3.88	3.95
	Inflow 1	6.83E+09	58.07	0.07	0.03	0.03	0.01	0.09	0.38	0.39	0.01	0.09	0.34	0.35
3	Inflow 2	2.53E+08	17.73	0.14	0.05	0.05	0.02	0.17	0.76	0.78	0.02	0.17	0.68	0.69
	Outflow	2.35E+10	17.30	0.13	0.09	0.09	0.06	0.15	0.92	0.98	0.06	0.15	0.48	0.54

Table 13. Pollutant removal efficiencies calculated for MRP. Event specific pollutant removal efficiencies, the average event removal efficiencies and the removal efficiencies from the sum of the total loads from all events are presented. Cells with "n/a" are displayed if reasonable estimates for loadings were not achieved.

Pond MRP: Pollutant Removal Efficiencies (%)													
Event	**fc**	**TSS**	**TP**	**TDP**	**OP**	**NO3+2**	**NH4**	**TKN**	**TN**	**NH4 diss**	**NO3+2**	**TKN diss**	**TDN**
1	98.32	82.26	76.45	77.60	90.88	28.70	n/a	63.73	63.15	50.07	-12.72	44.54	43.43
2	-477.31	91.01	62.31	52.61	78.25	88.39	n/a	12.26	18.11	n/a	86.92	n/a	n/a
3	11.10	89.35	70.02	n/a	n/a	94.35	-29.93	35.04	51.47	n/a	n/a	n/a	n/a
4	98.75	69.39	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
5	n/a	87.48	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Avg. of event % removals	-67.28	83.90	69.59	65.10	84.56	70.48	-29.93	37.01	44.24	50.07	37.10	44.54	43.43
% removal from total	55.32	87.70	70.70	70.40	87.95	81.76	-29.93	36.60	39.19	50.07	73.06	44.54	43.43

Table 14. Pollutant removal efficiencies calculated for SRP. Event specific pollutant removal efficiencies, the average event removal efficiencies and the removal efficiencies from the sum of the total loads from all events are presented. Cells with "n/a" are displayed if reasonable estimates for loadings were not achieved.

Pond SRP: Pollutant Removal Efficiencies (%)													
Event	**fc**	**TSS**	**TP**	**TDP**	**OP**	**NO3+2**	**NH4**	**TKN**	**TN**	**NO3+2 diss**	**NH4 diss**	**TKN diss**	**TDN**
1	n/a	17.69	10.45	1.53	4.70	-6.39	-70.22	11.06	10.07	-1.65	-103.30	11.06	10.45
2	60.52	-79.10	-122.47	-41.64	-41.64	80.05	-100.06	-54.35	-41.29	82.31	-208.01	0.35	8.32
3	-232.23	77.17	38.73	-20.35	-20.35	-138.40	42.13	19.76	16.33	-138.40	42.13	53.14	48.46
Avg. of event % removals	-85.86	5.26	-24.43	-20.15	-19.10	-21.58	-42.72	-7.84	-4.96	-19.25	-89.73	21.51	22.41
% removal from total loads	13.71	18.54	-5.54	-5.21	-2.55	23.53	-65.93	-4.24	-2.46	30.32	-100.39	11.12	12.25

FIGURES

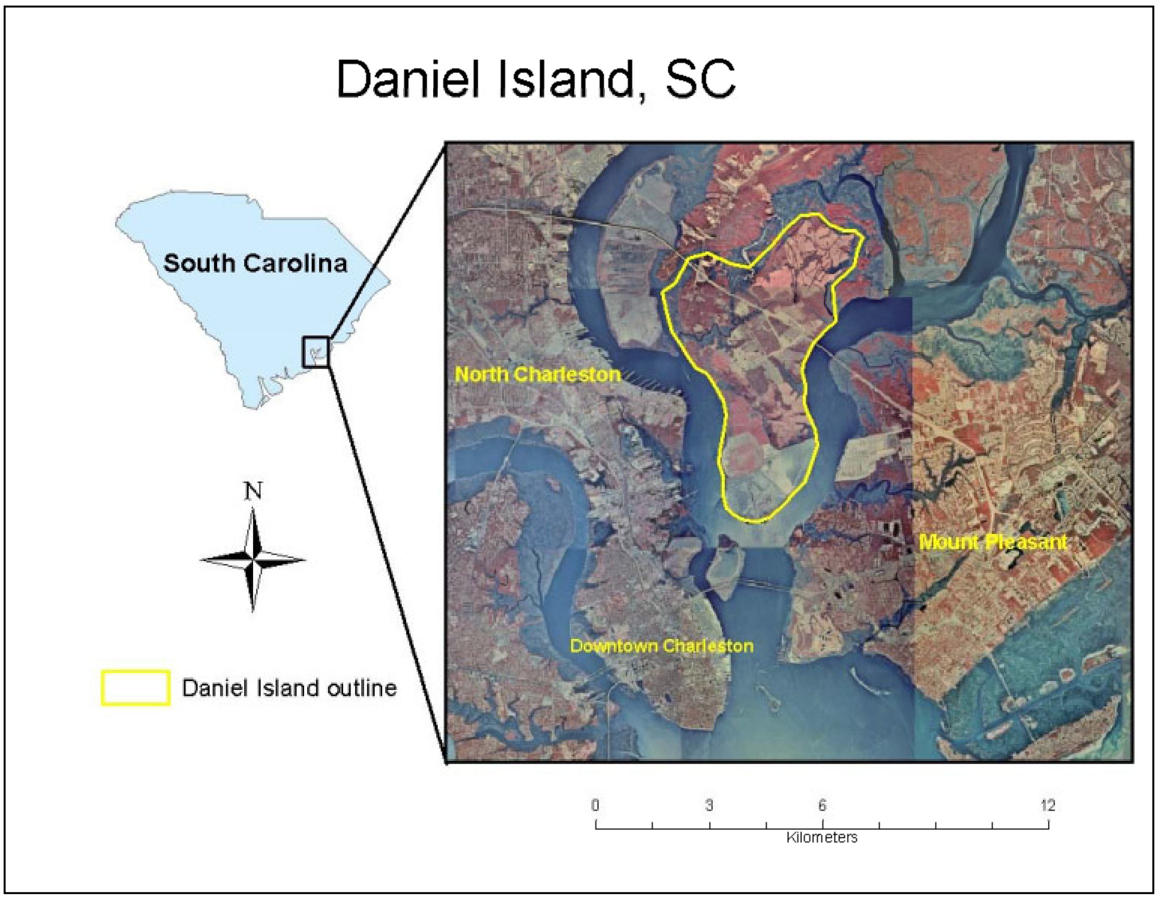


Figure 1. Map showing the Daniel Island, SC study area.

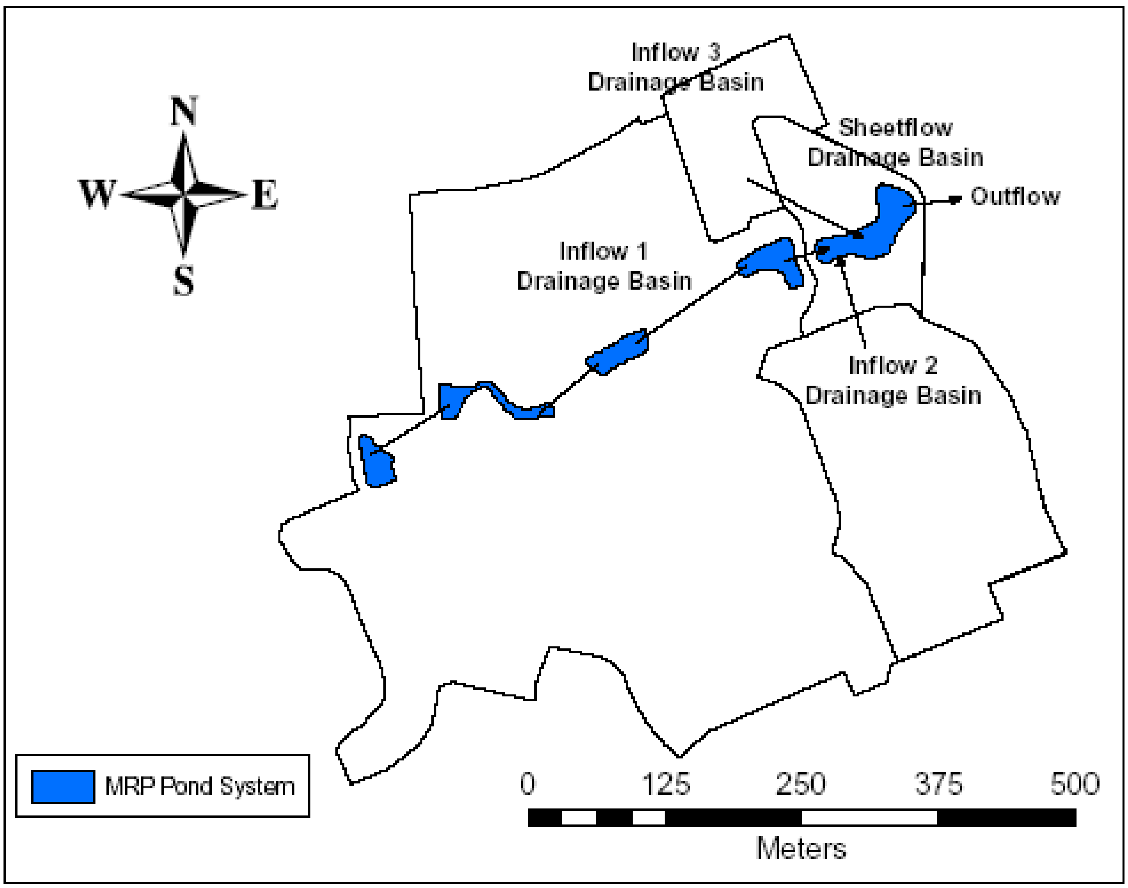


Figure 2. Diagram of the five ponds and drainage basins comprising the multiple residential pond (MRP) system.

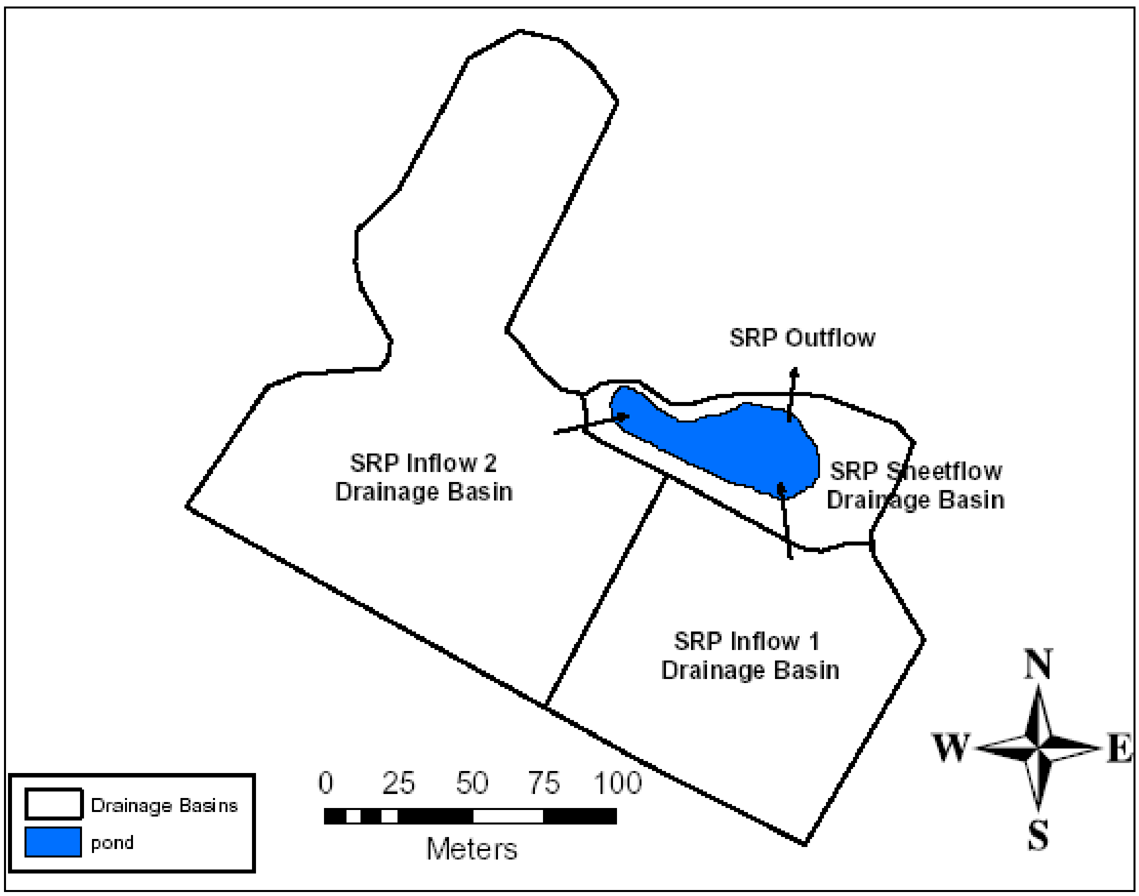


Figure 3. Diagram of the pond and drainage basins comprising the single residential pond (SRP) system.

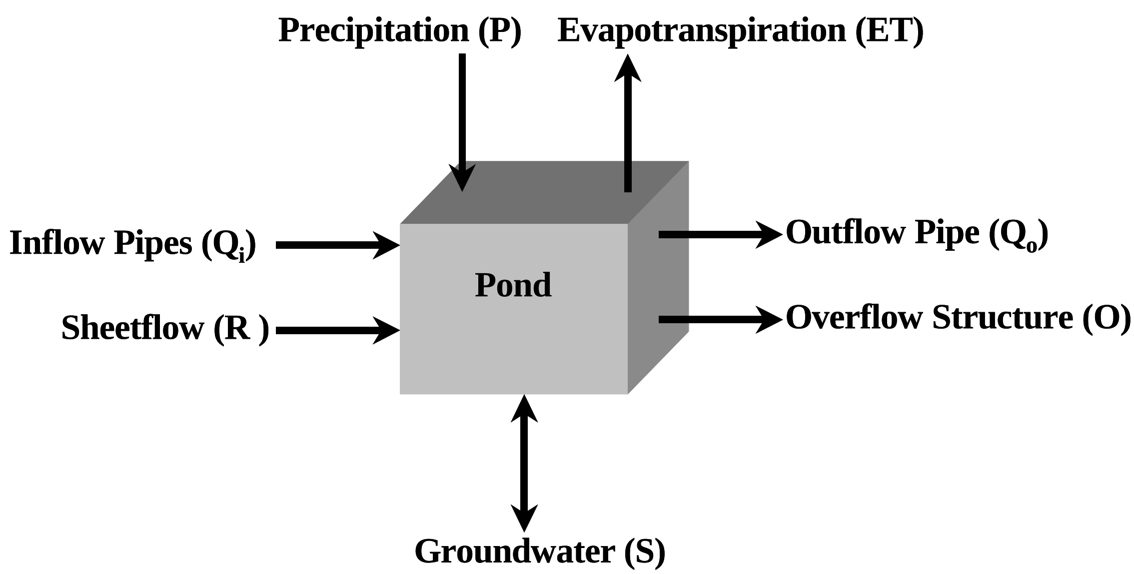


Figure 4. Model used for developing water budgets for two wet detention ponds (refer to Equation 1).

Figure 5. Map depicting the land cover in the MRP drainage basins.

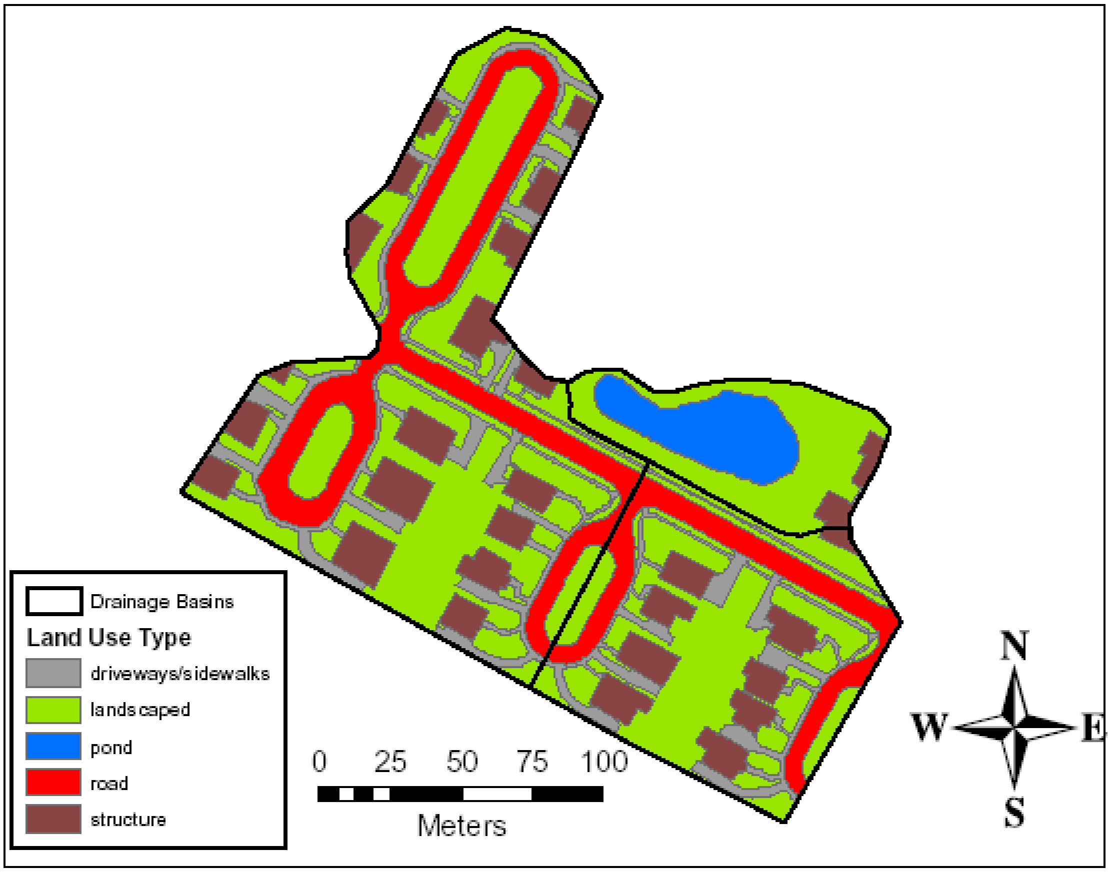
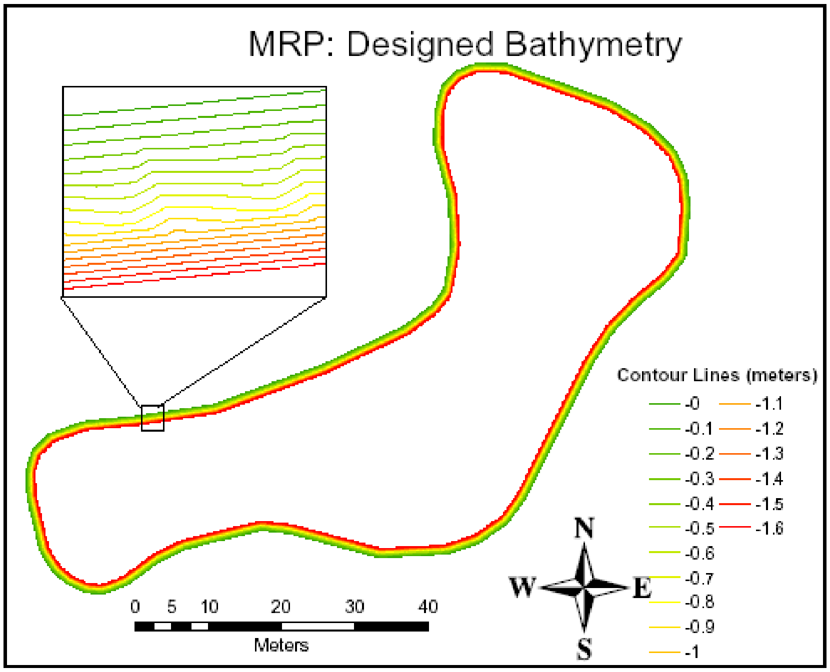


Figure 6. Map depicting the land cover in the SRP drainage basins.

(A)



(B)

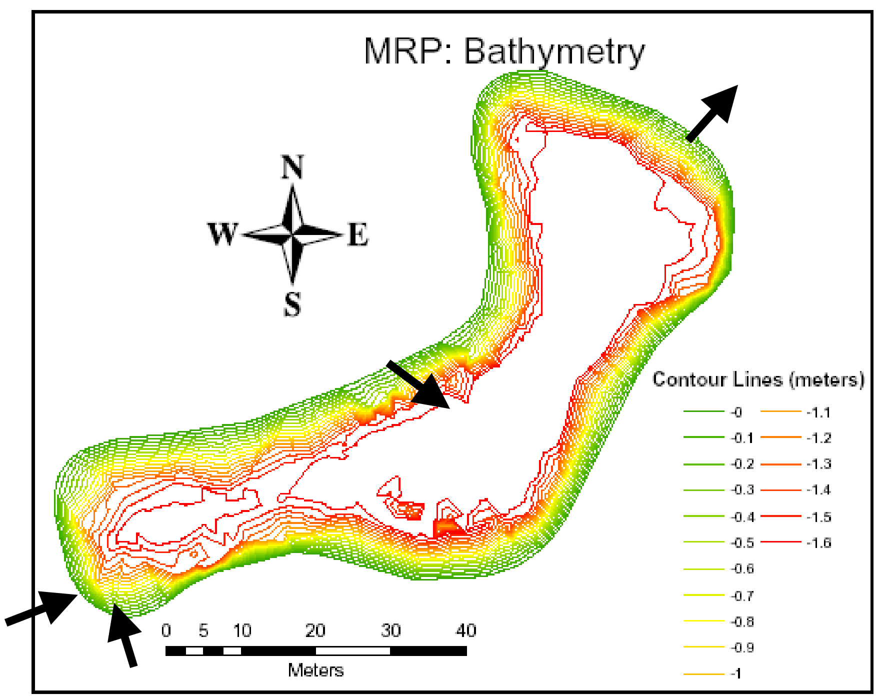


Figure 7. Designed bathymetry of MRP as defined by Thomas and Hutton Engineering Co. (A) and current bathymetry of MRP (B). Arrows represent inflow and outflow pipes.

(A)

(B)

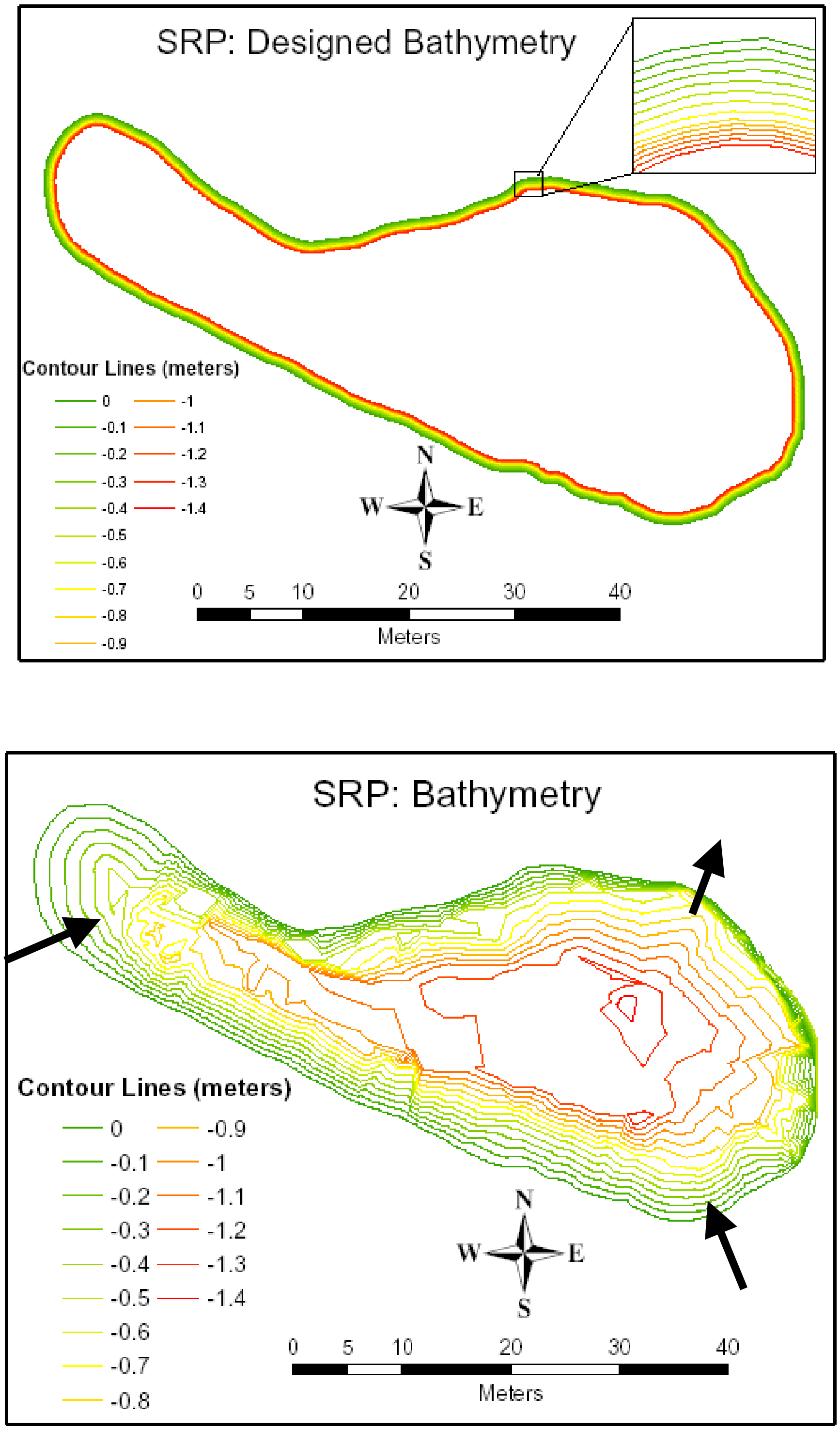
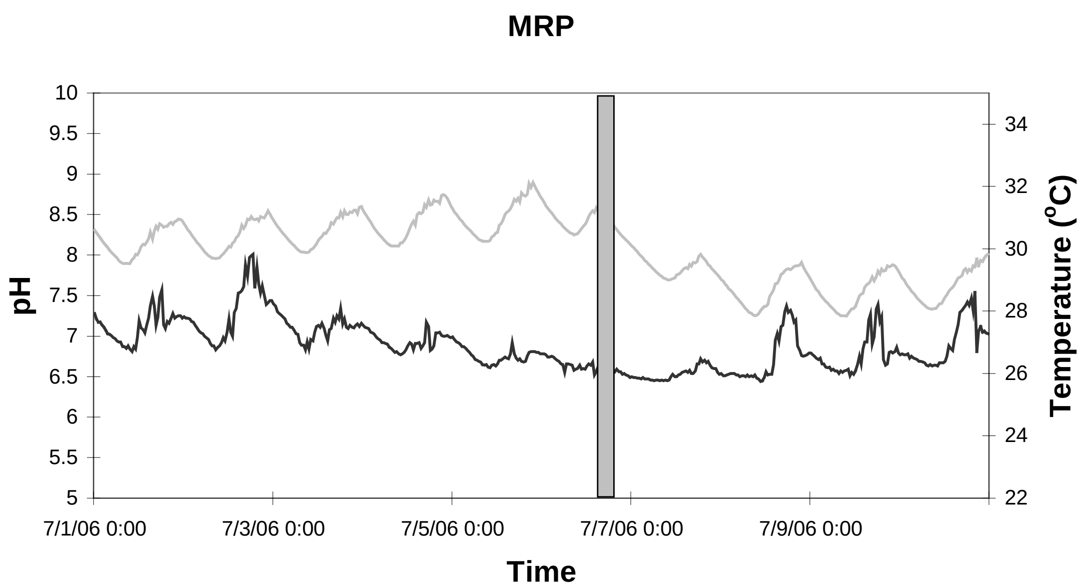


Figure 8. Designed bathymetry of SRP as defined by Thomas and Hutton Engineering Co. (A) and current bathymetry of SRP (B). Arrows represent inflow and outflow pipes.

(A)



(B)

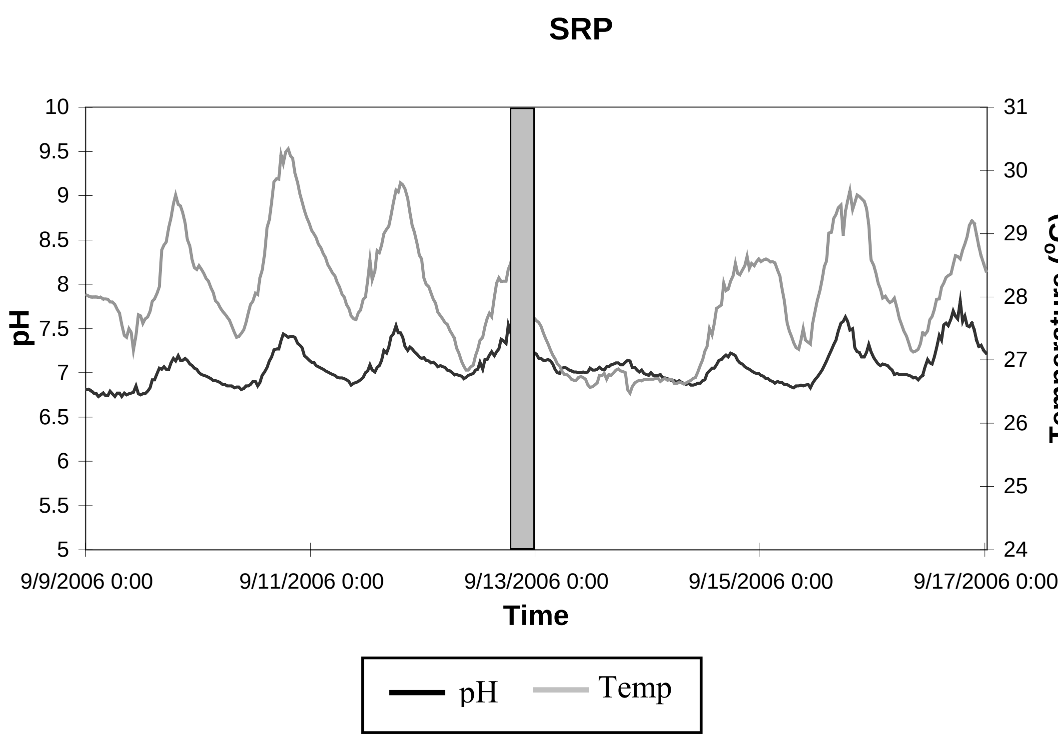
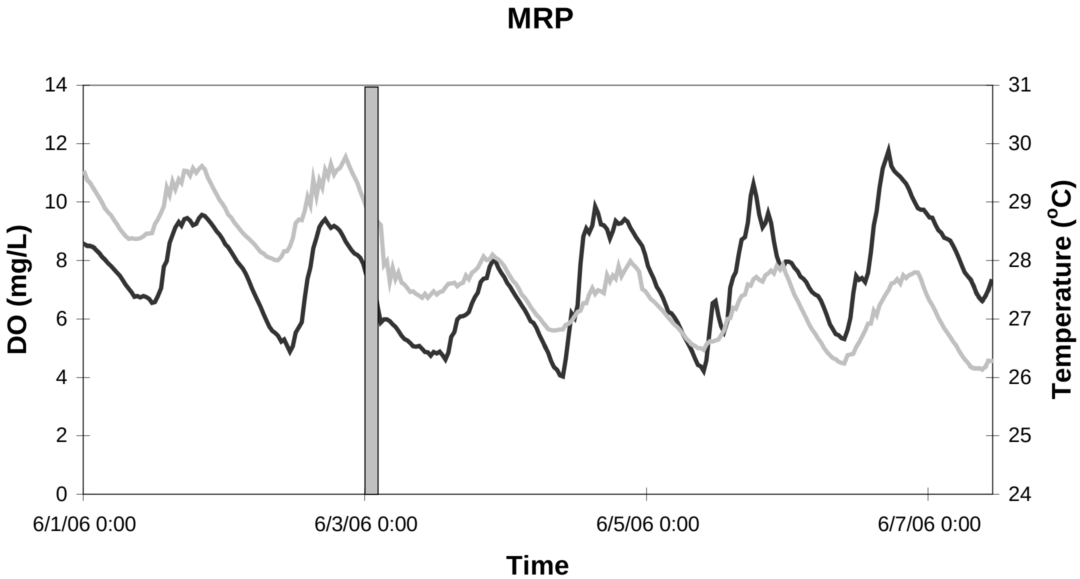


Figure 9. Temperature and pH levels observed in MRP (A) and SRP (B). Rainfall events are depicted by shaded areas.

(A)



(B)

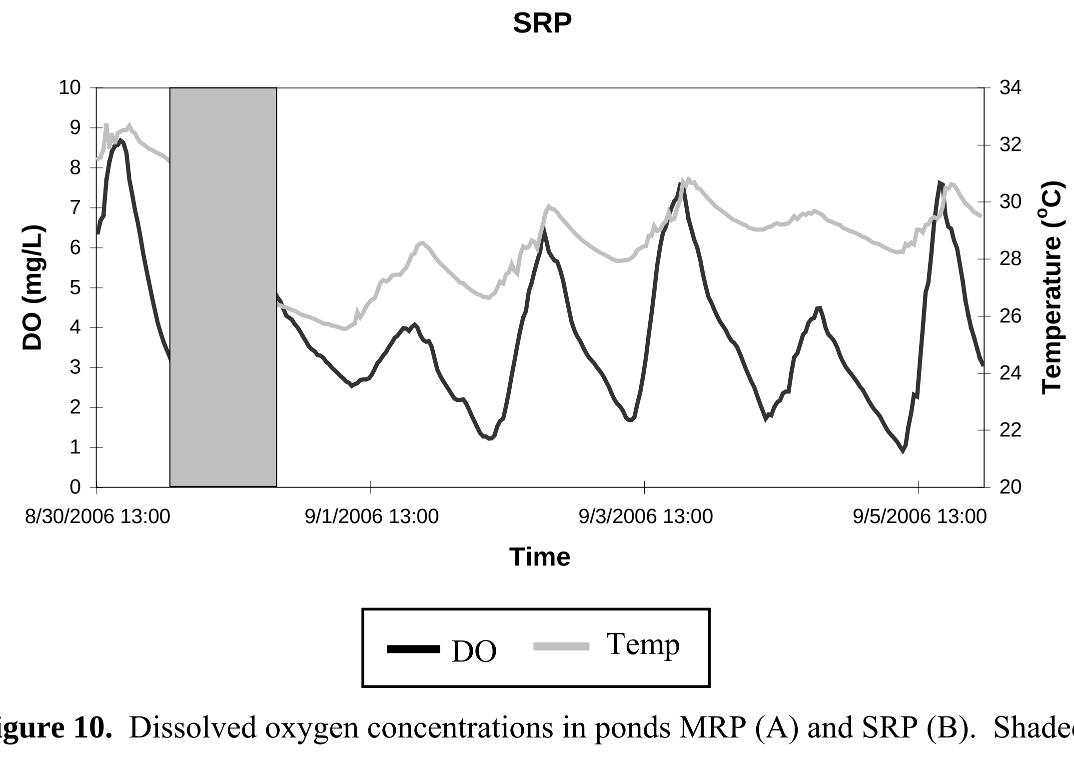


Figure 10. Dissolved oxygen concentrations in ponds MRP (A) and SRP (B). Shaded area represents period of rainfall.

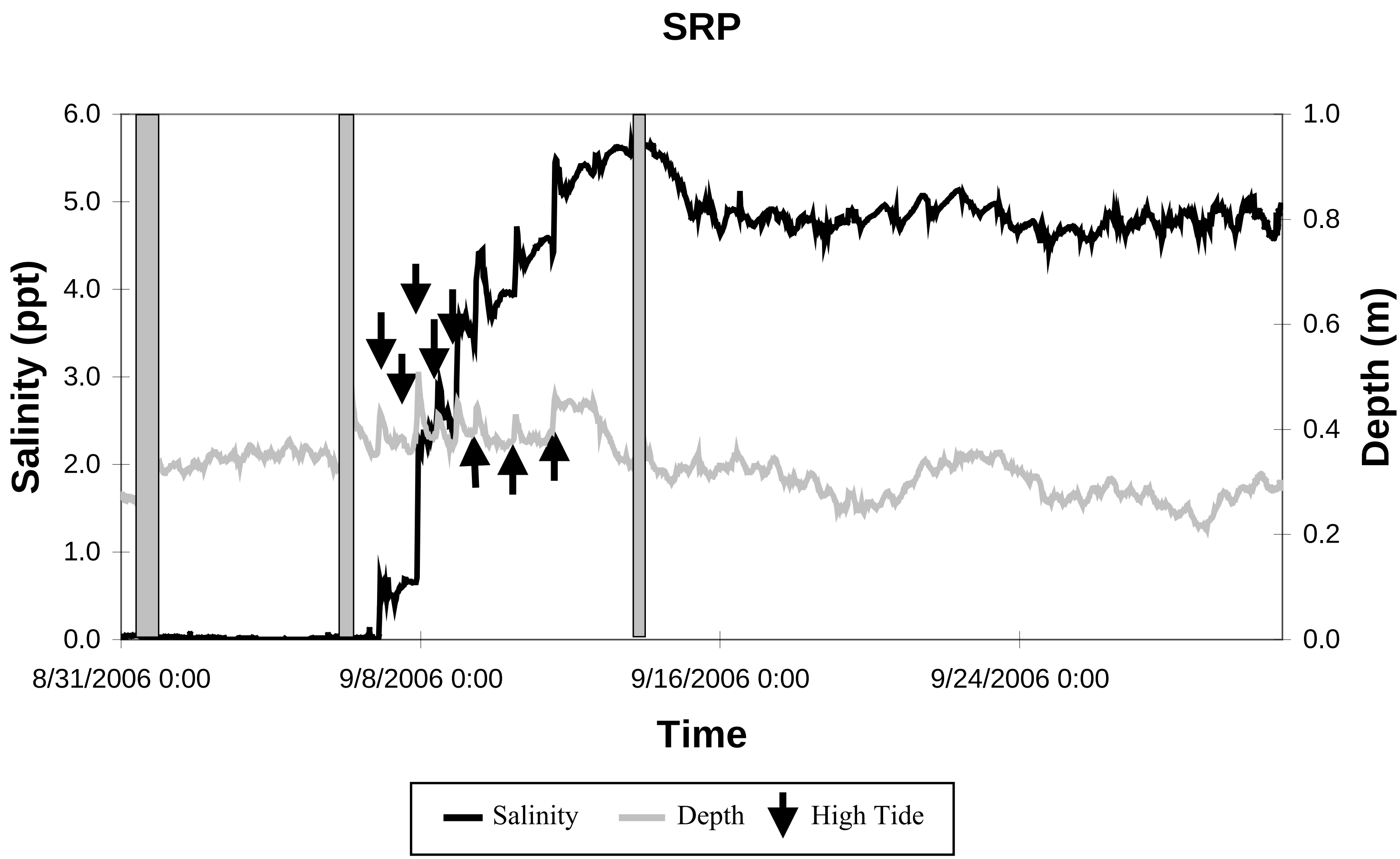


Figure 11. Salinity levels and depth in SRP following a series of spring tides. Shaded areas represent periods of rainfall, and arrows indicate periods of spring high tides.

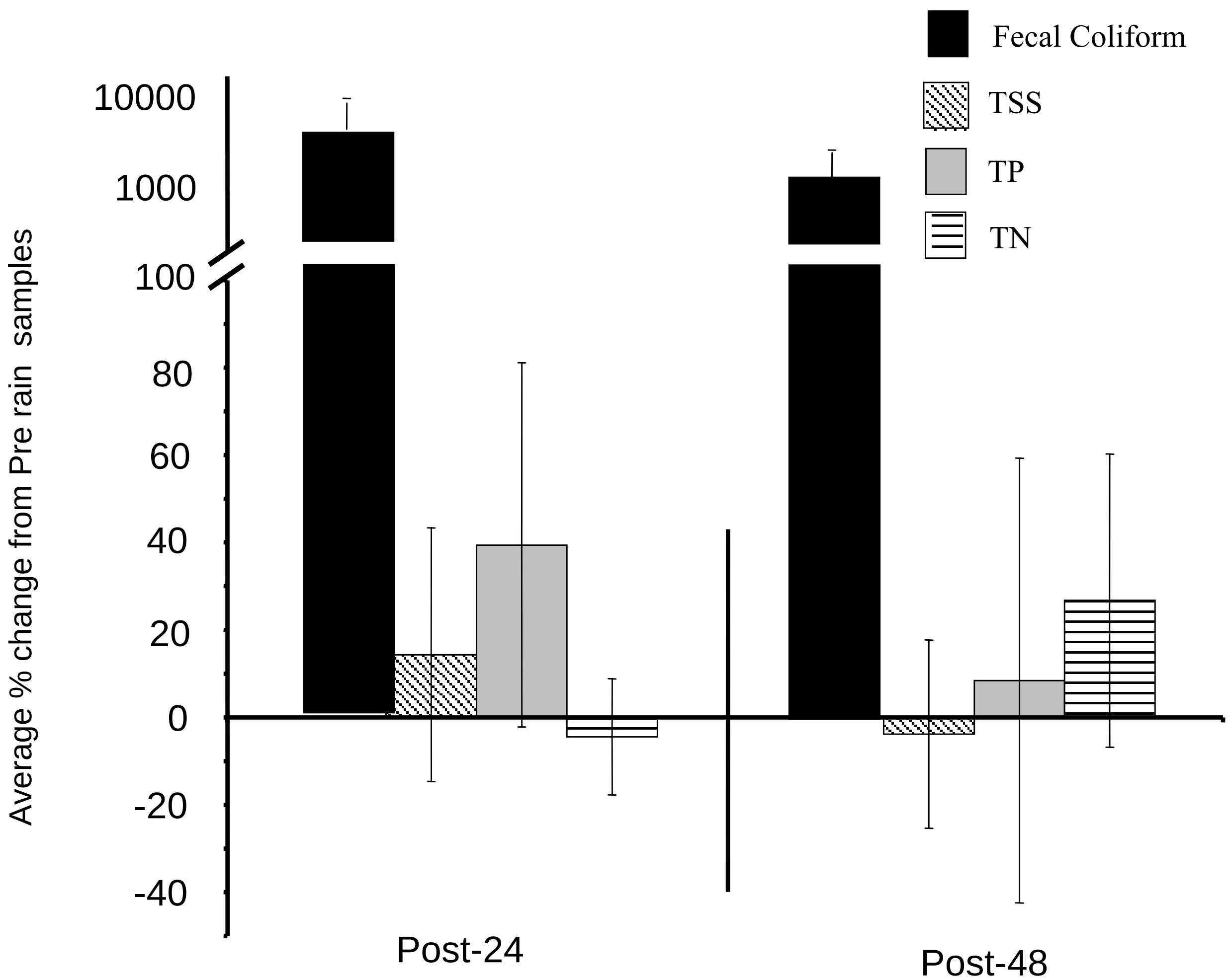
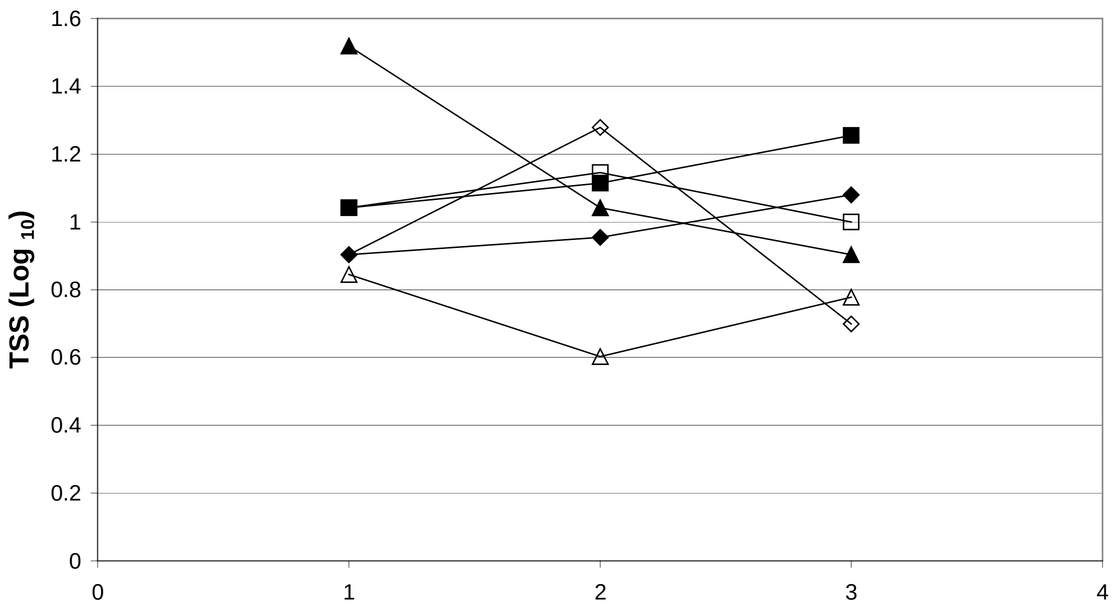


Figure 12. Average percent change from the pre rain event samples. Error bars are one standard error.

(A)



(B)

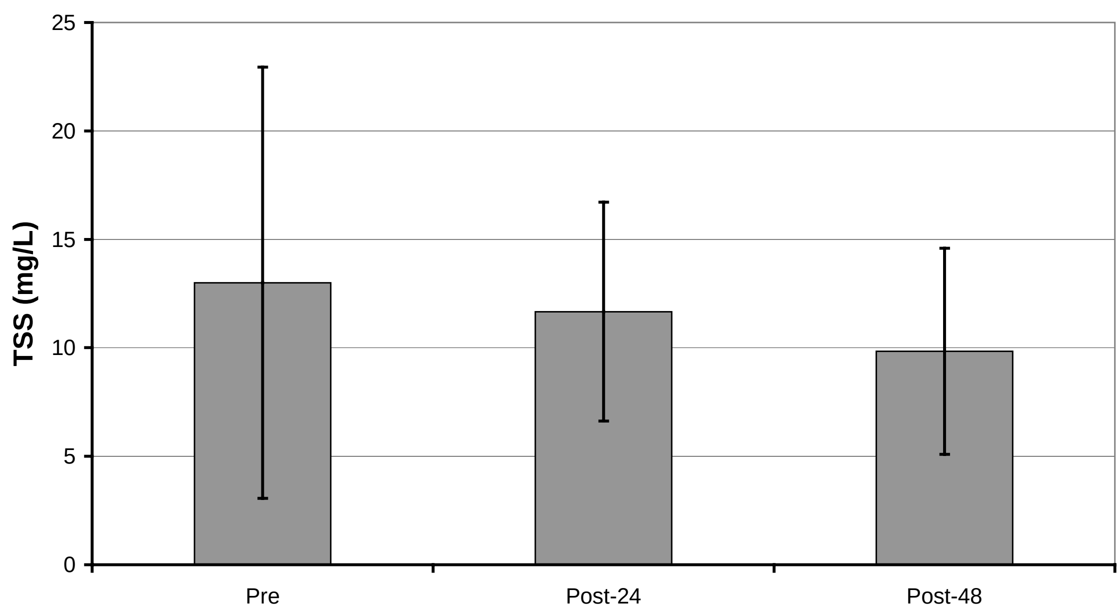
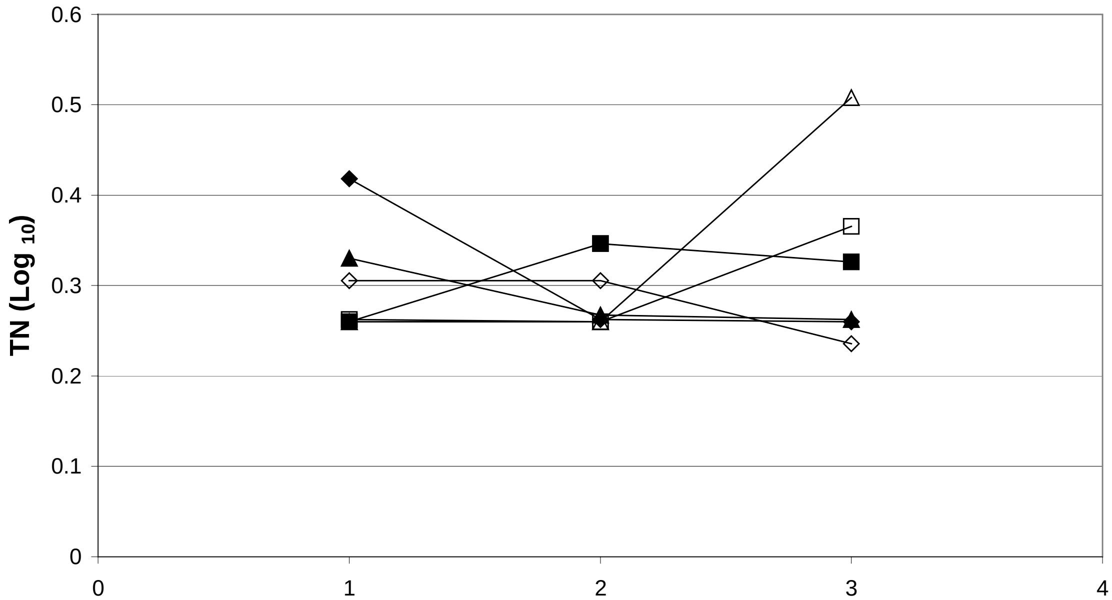


Figure 13. Total suspended solid point sampling concentrations for each event in each pond (A) and the average concentrations for both ponds combined with standard deviation bars (B).

(A)



(B)

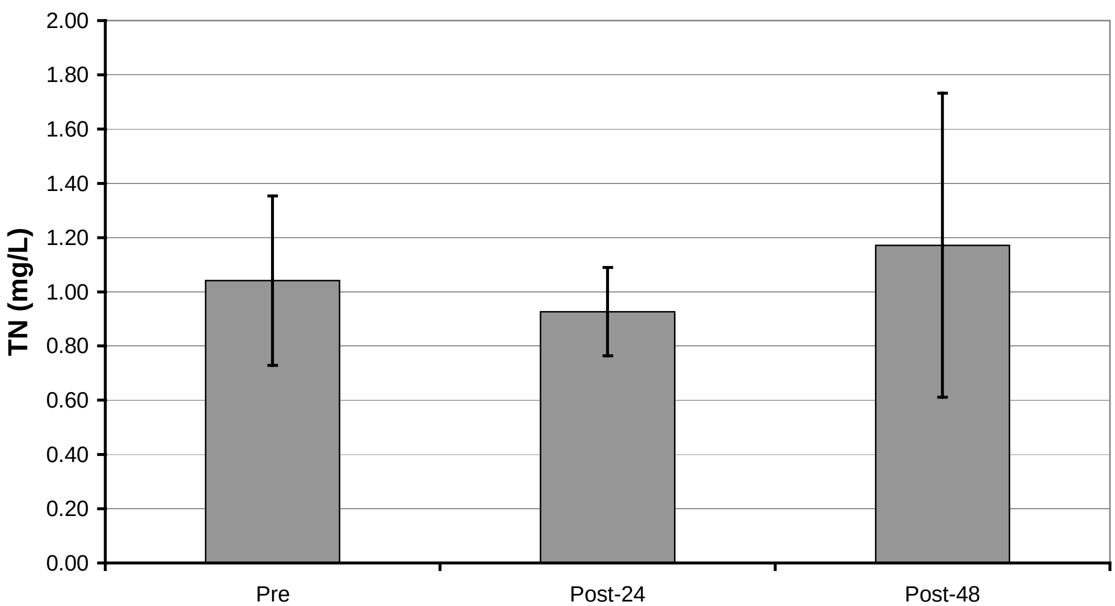
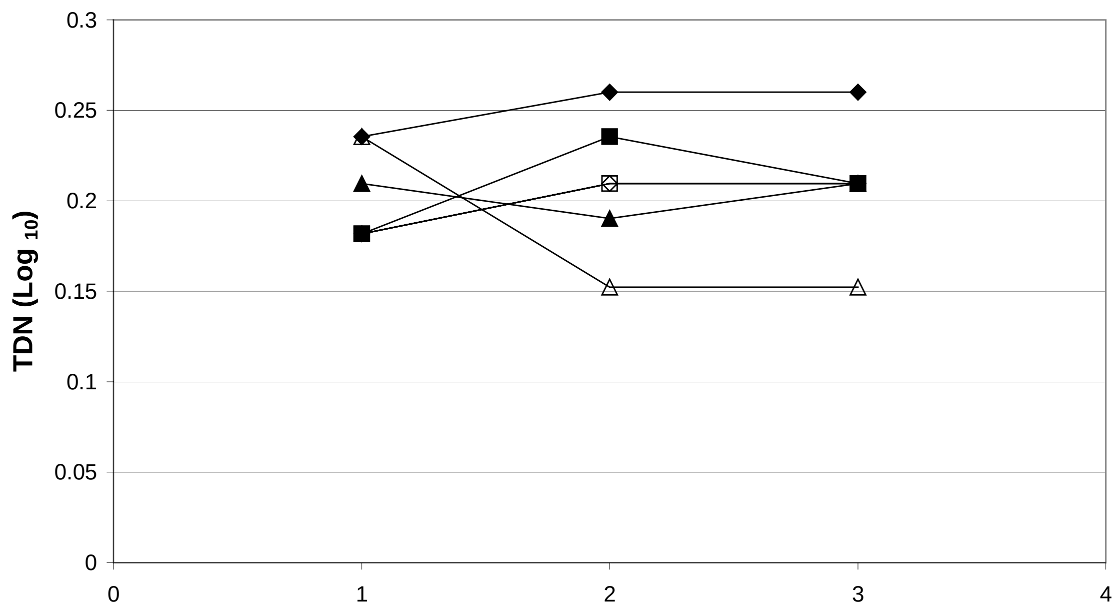


Figure 14. Total nitrogen point sampling concentrations for each event in each pond (A) and the average concentrations for both ponds combined with standard deviation bars (B).

(A)



(B)

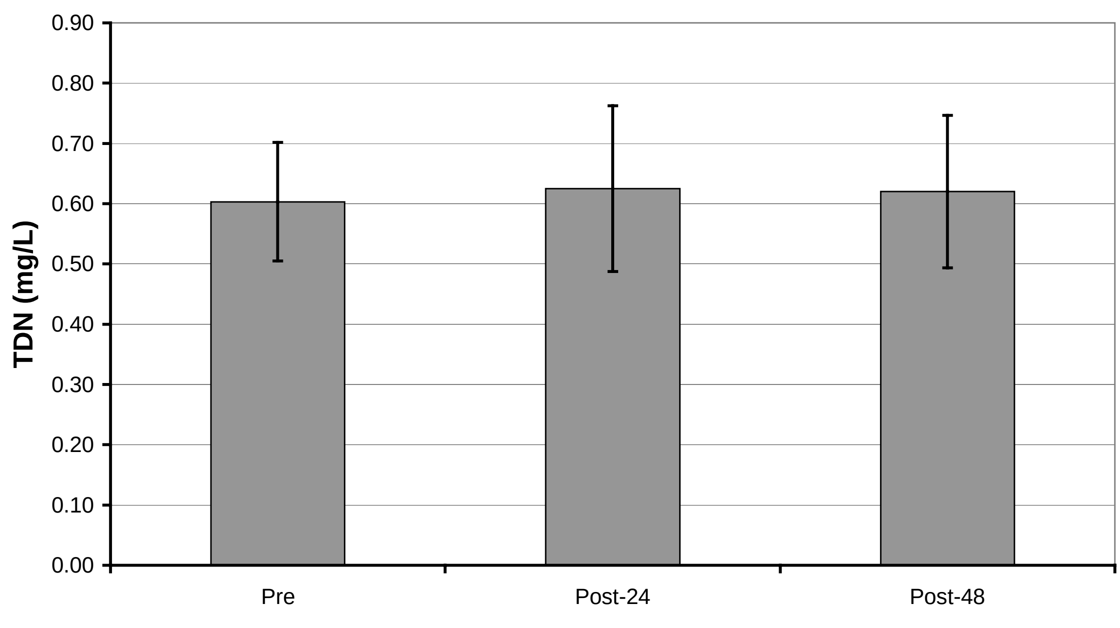
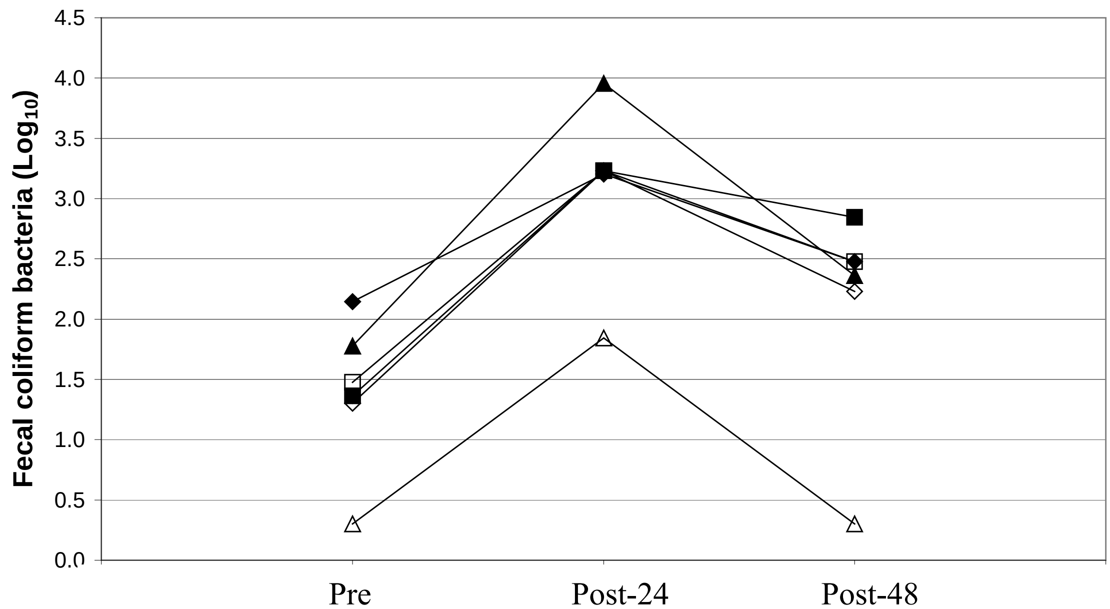


Figure 15. Total dissolved nitrogen point sampling concentrations for each event in each pond (A) and the average concentrations for both ponds combined with standard deviation bars (B).

(A)



(B)

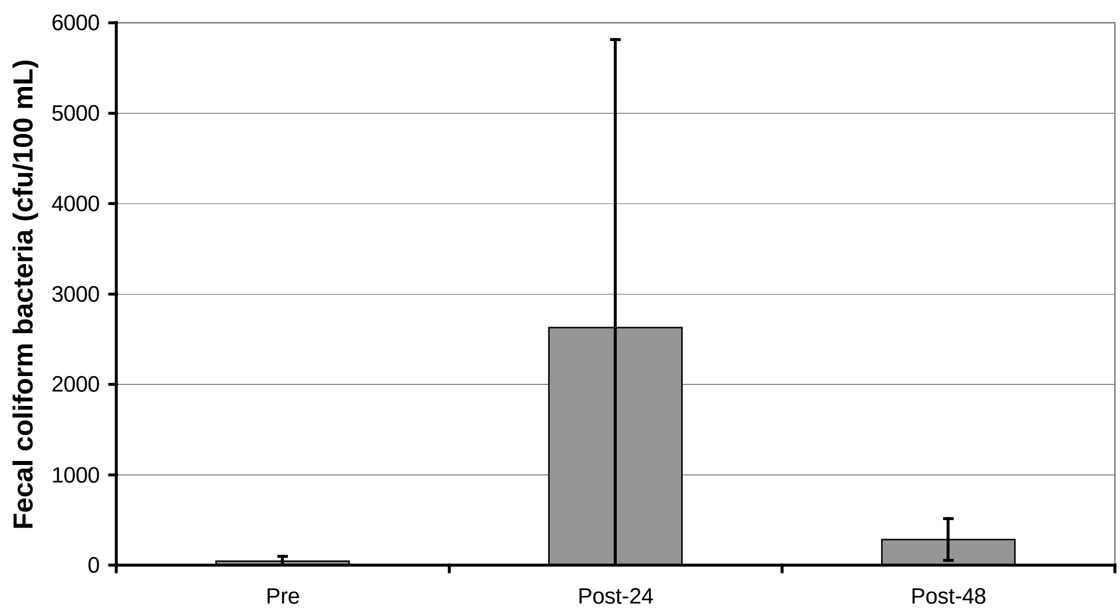
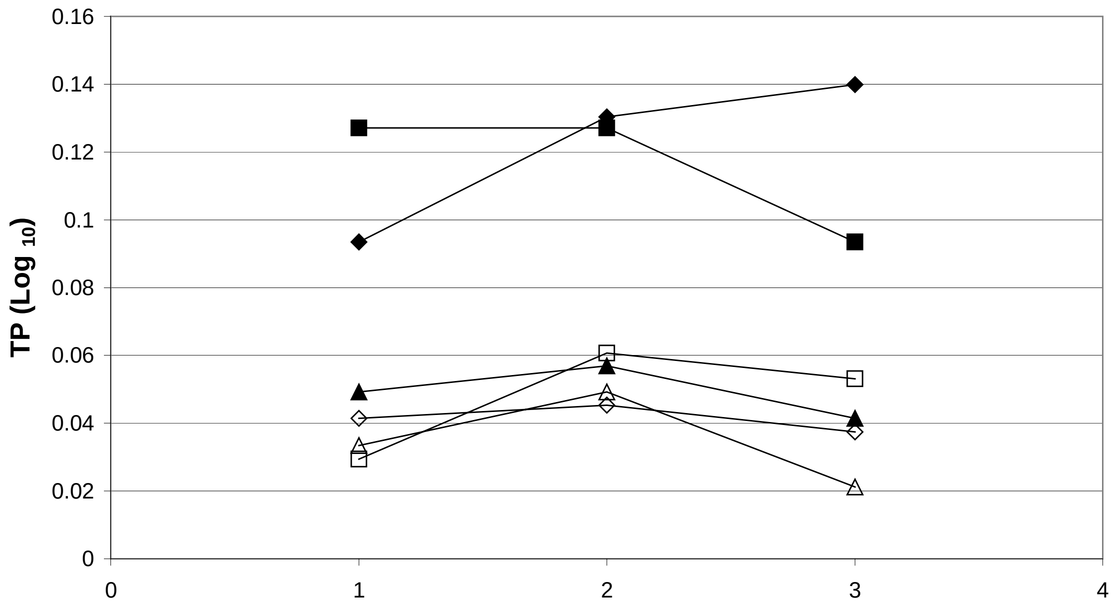


Figure 13. Fecal coliform bacteria point sampling concentrations for each event in each pond (A) and the average concentrations for both ponds combined with standard deviation bars (B).

(A)



(B)

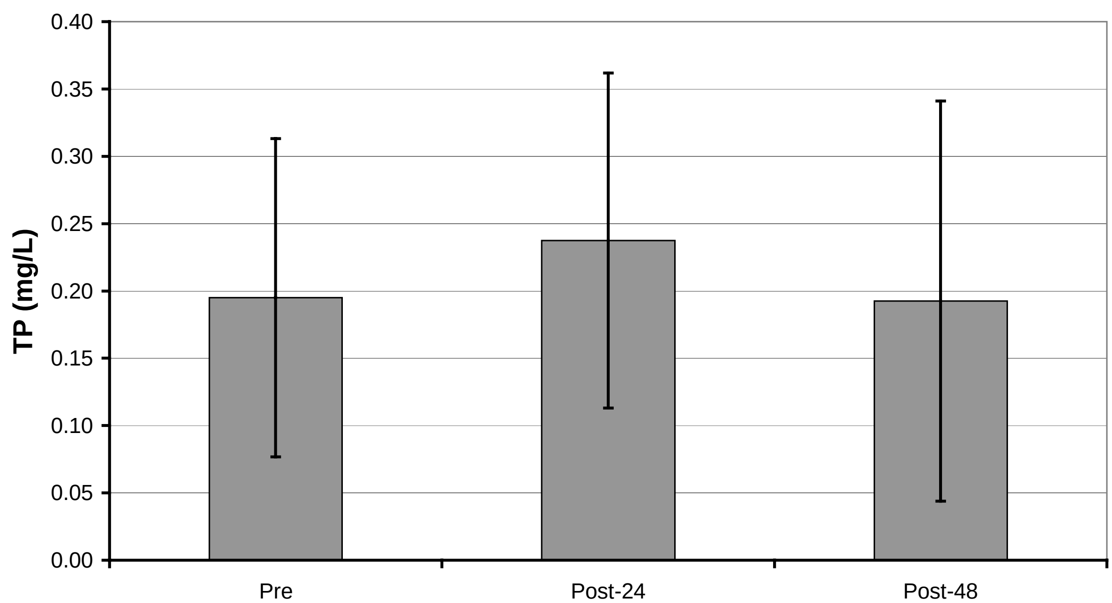
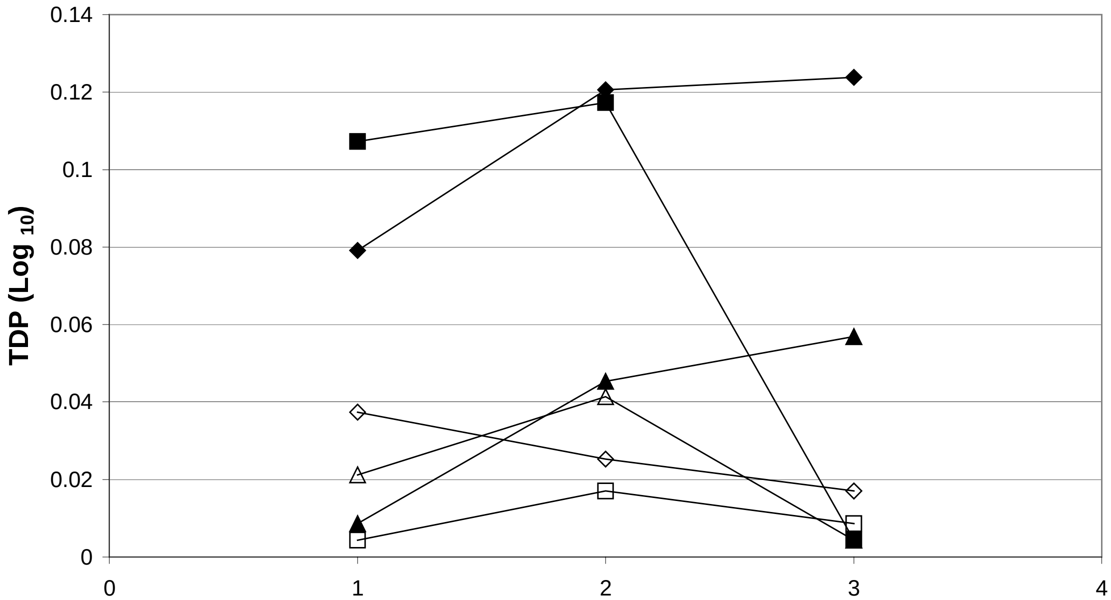


Figure 17. Total phosphorus point sampling concentrations for each event in each pond (A) and the average concentrations for both ponds combined with standard deviation bars (B).

(A)



(B)

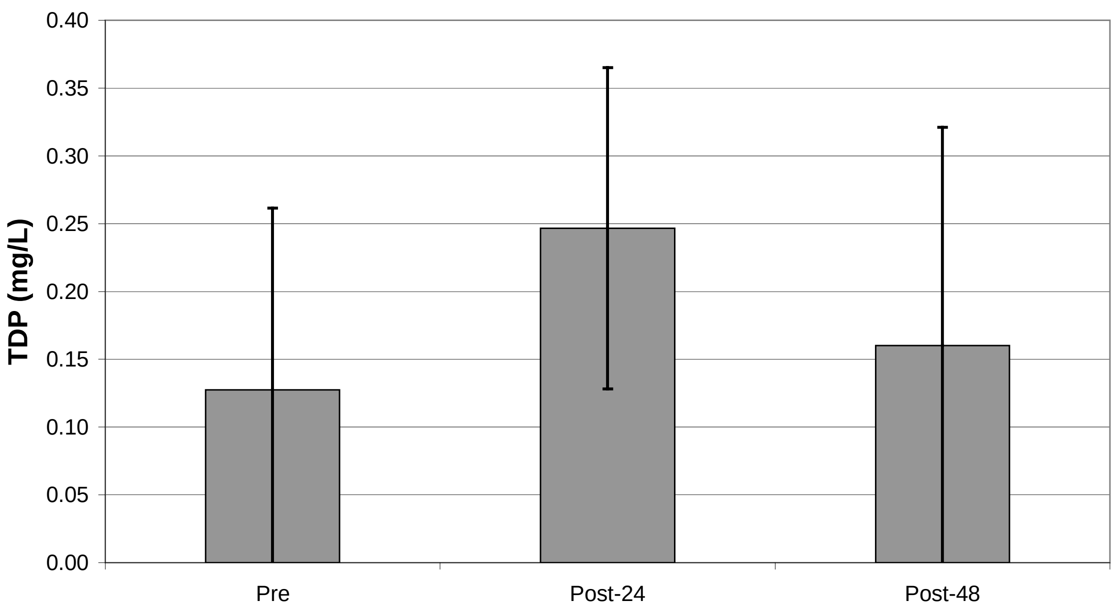
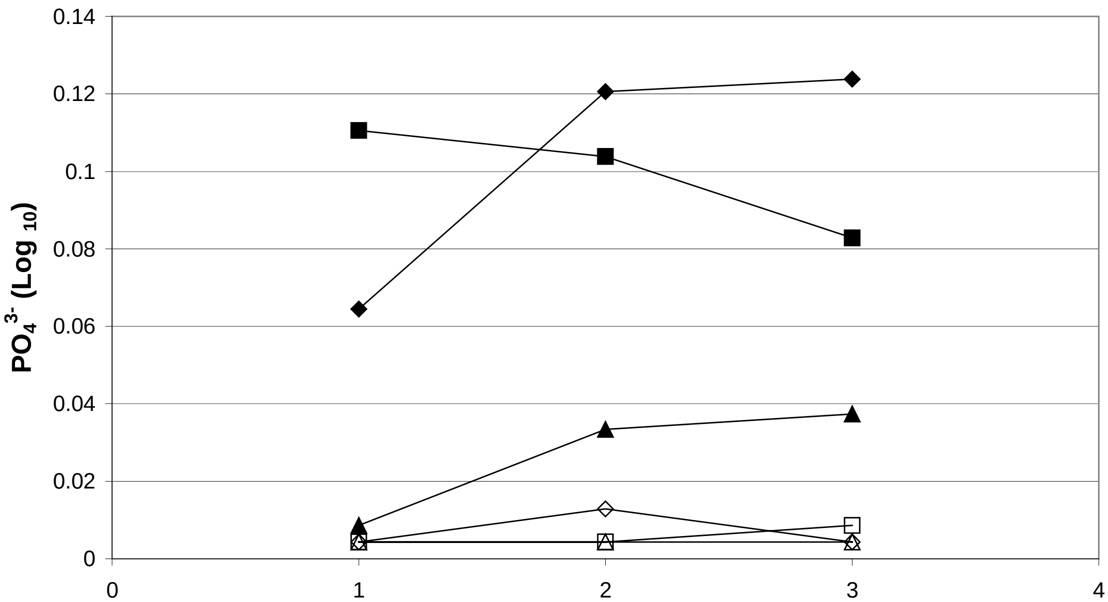


Figure 18. Total dissolved phosphorus point sampling concentrations for each event in each pond (A) and the average concentrations for both ponds combined with standard deviation bars (B).

(A)



(B)

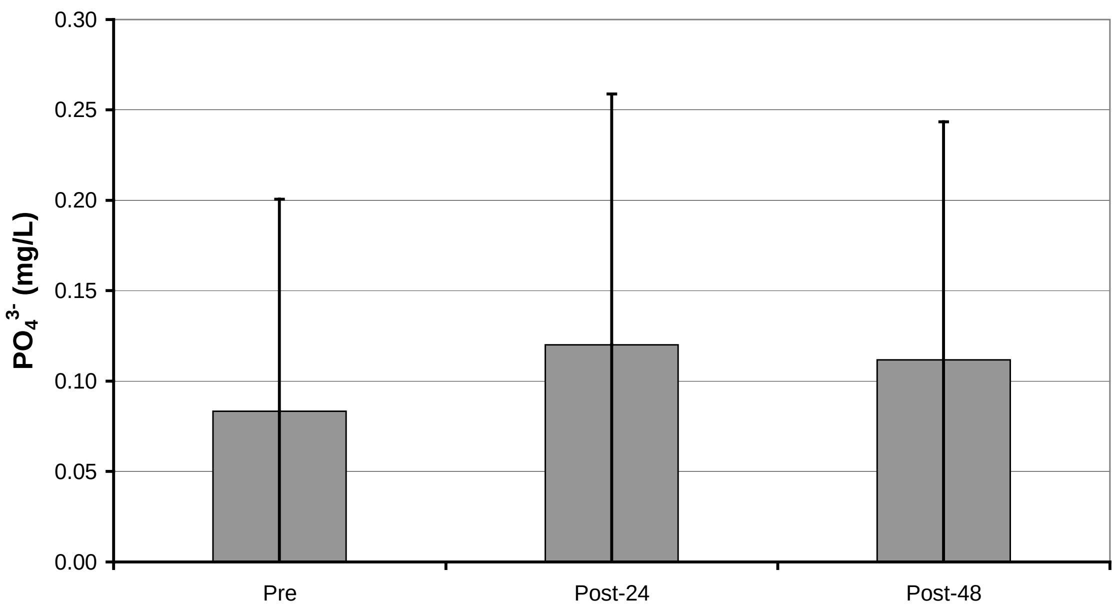


Figure 19. Orthophosphate point sampling concentrations for each event in each pond (A) and the average concentrations for both ponds combined with standard deviation bars (B).

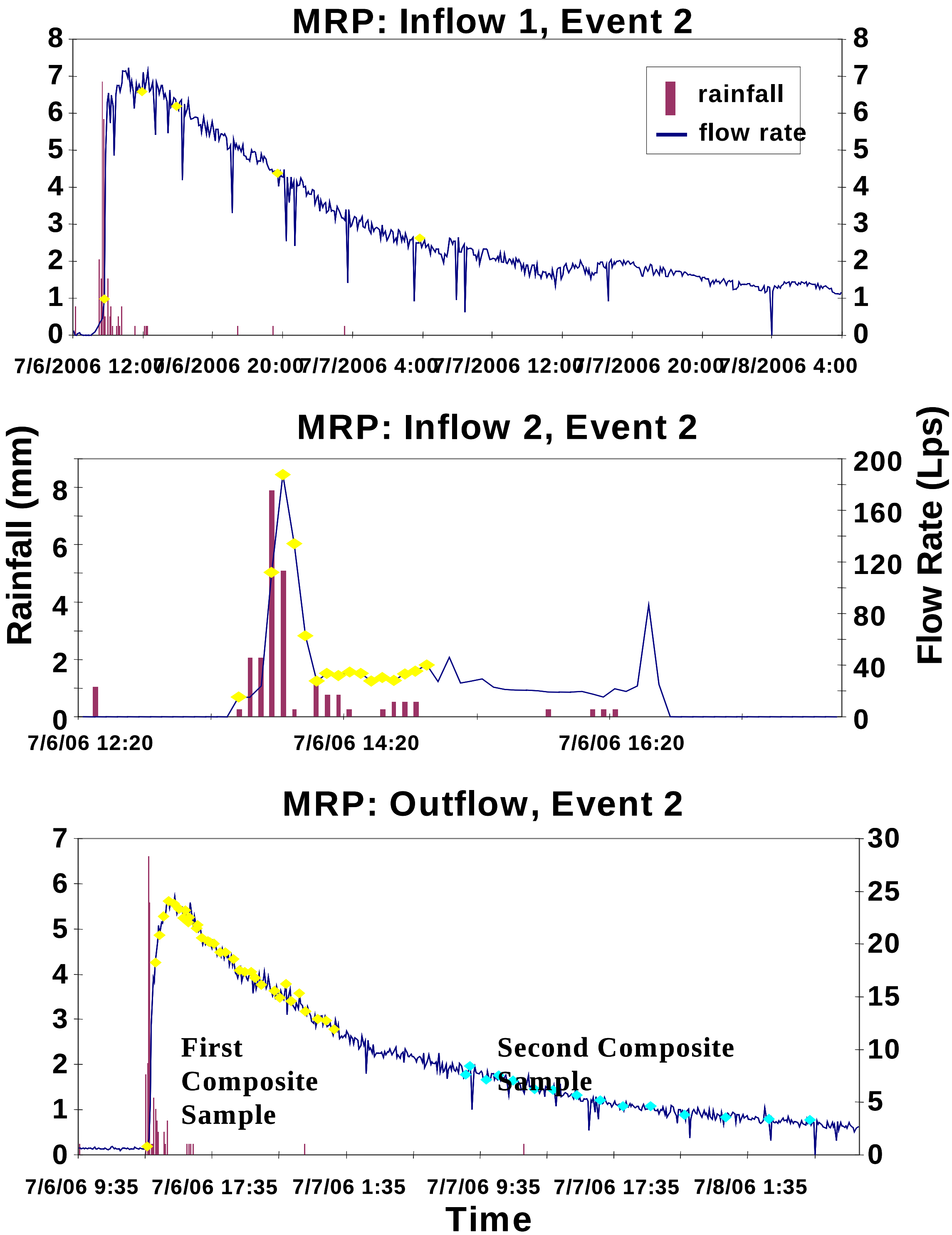


Figure 20. An example set of hydrographs, rainfall and sampling points for event 2 in MRP. Each point represents the time when samples were collected for the flow-weighted composite sampling.

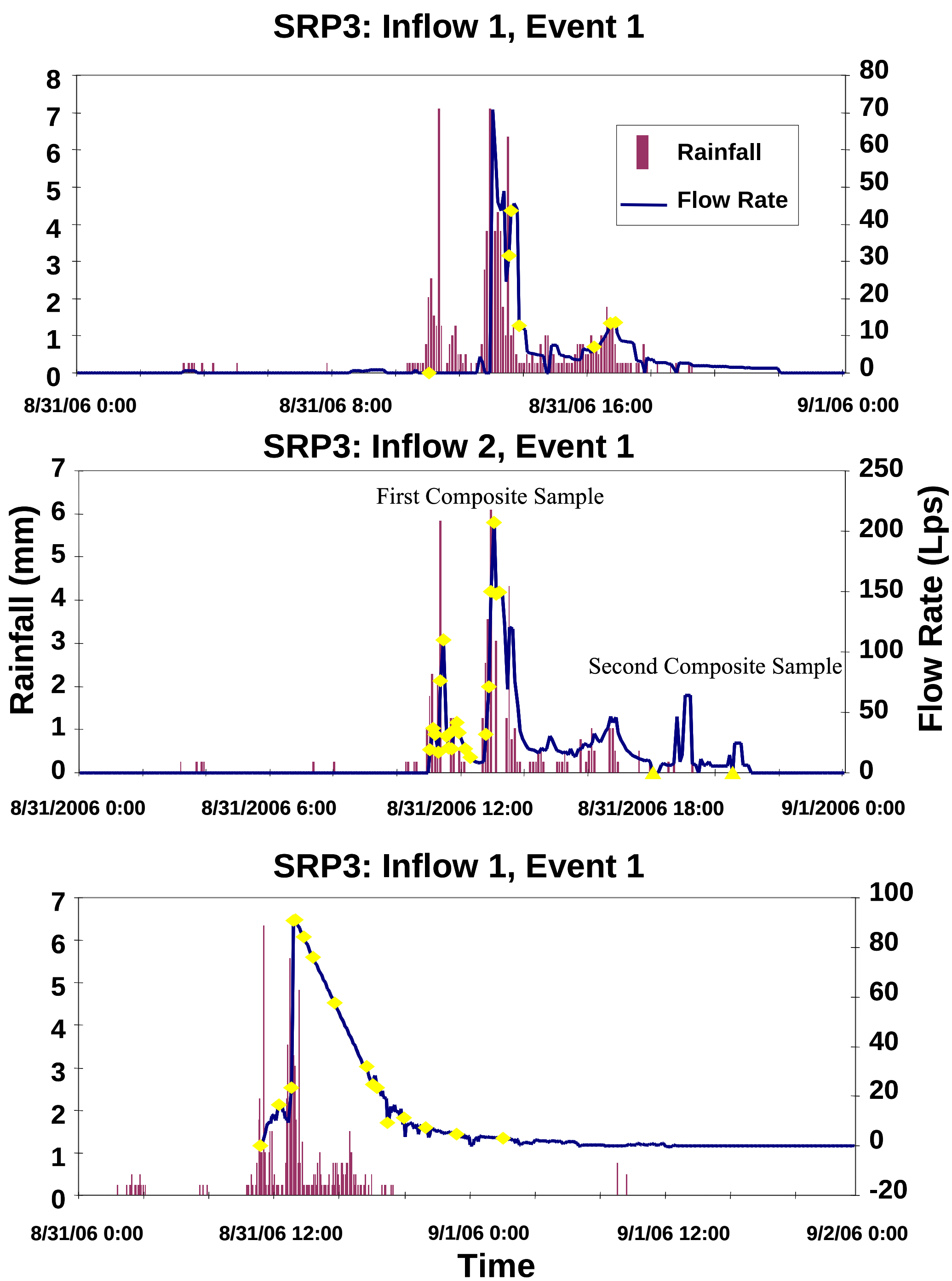


Figure 21. An example set of hydrographs, rainfall and sampling points for event 1 in SRP. Each point represents the time when samples were collected for the flow-weighted composite sampling.

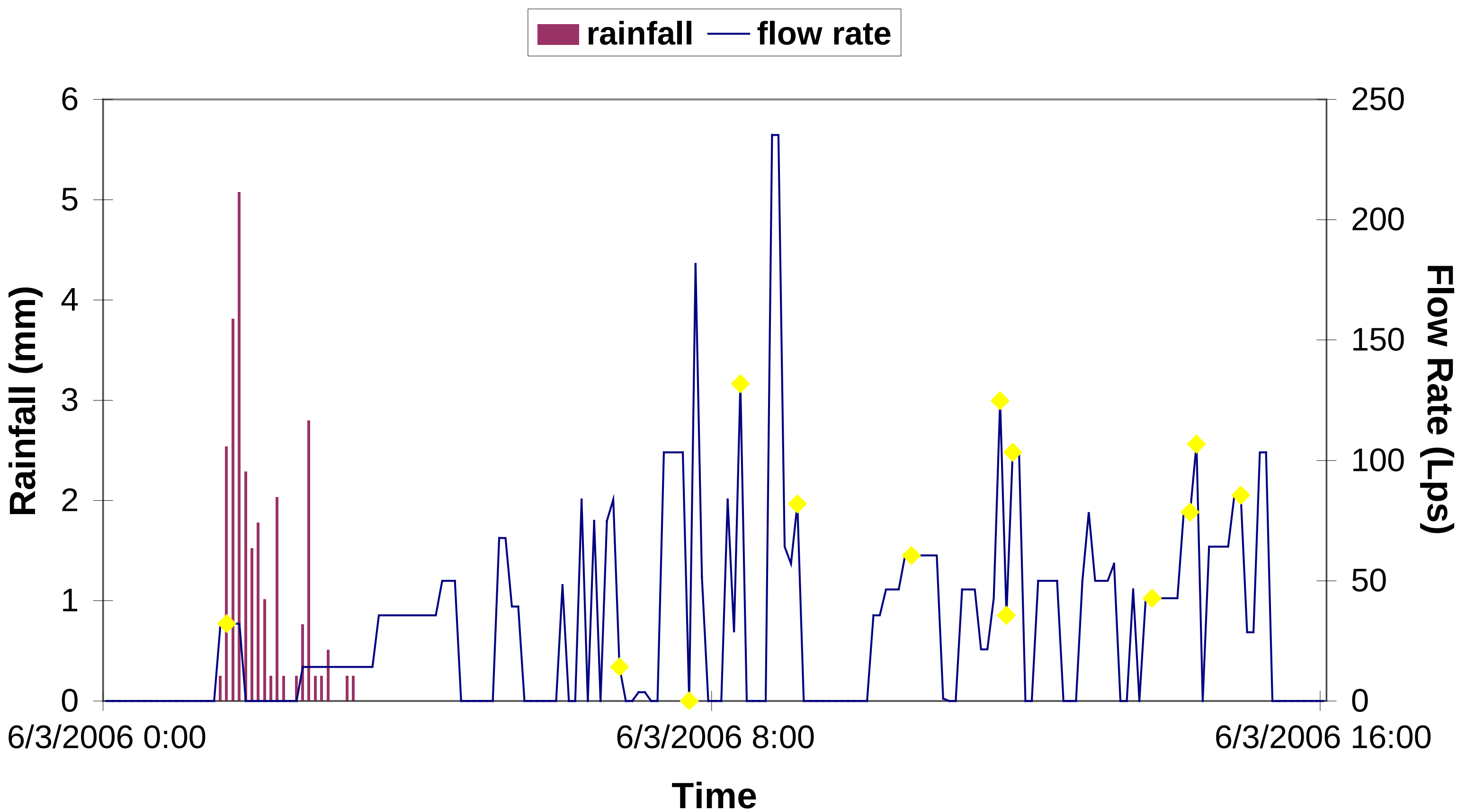


Figure 22. An example of a hydrograph that failed QA/QC from event 1 in MRP.

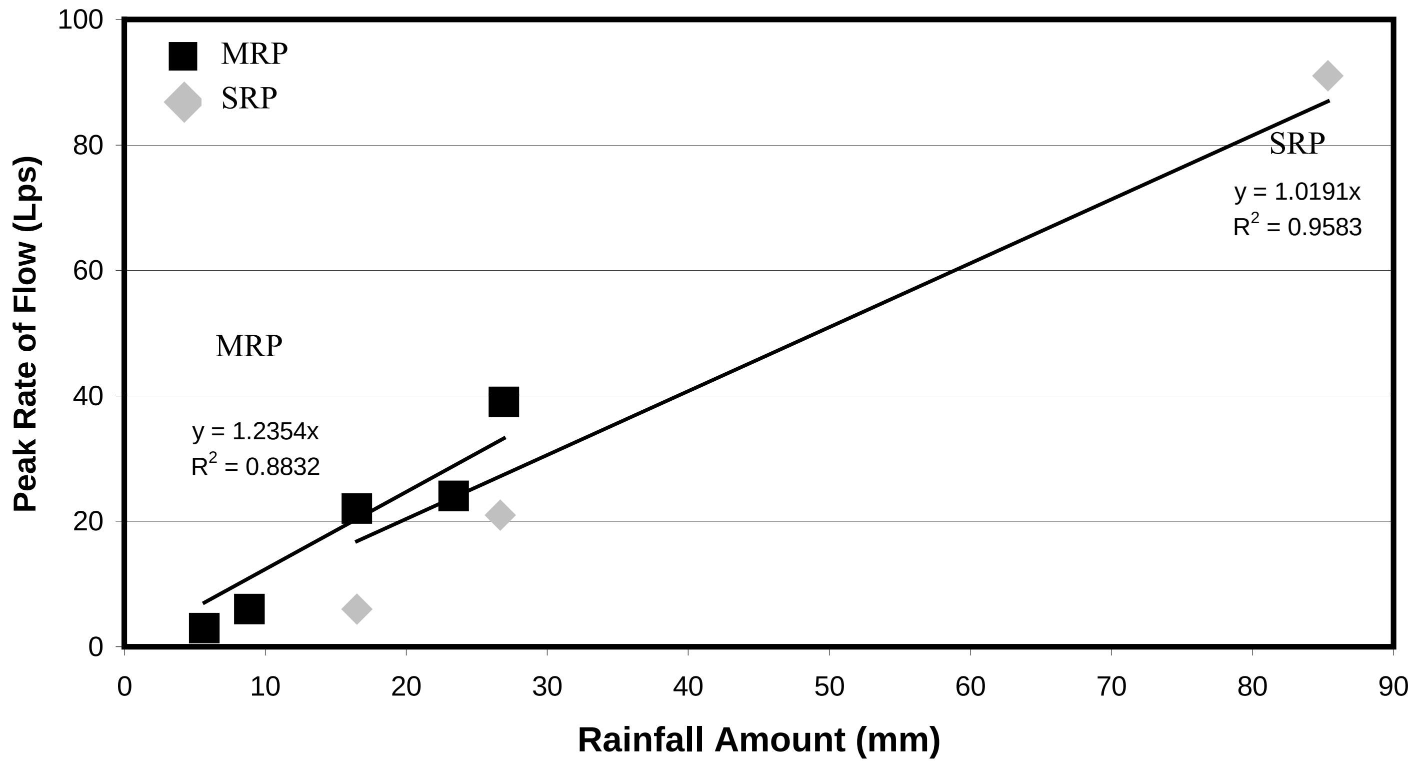


Figure 23. Relationship between peak rate of flow and amount of rainfall in MRP and SRP.

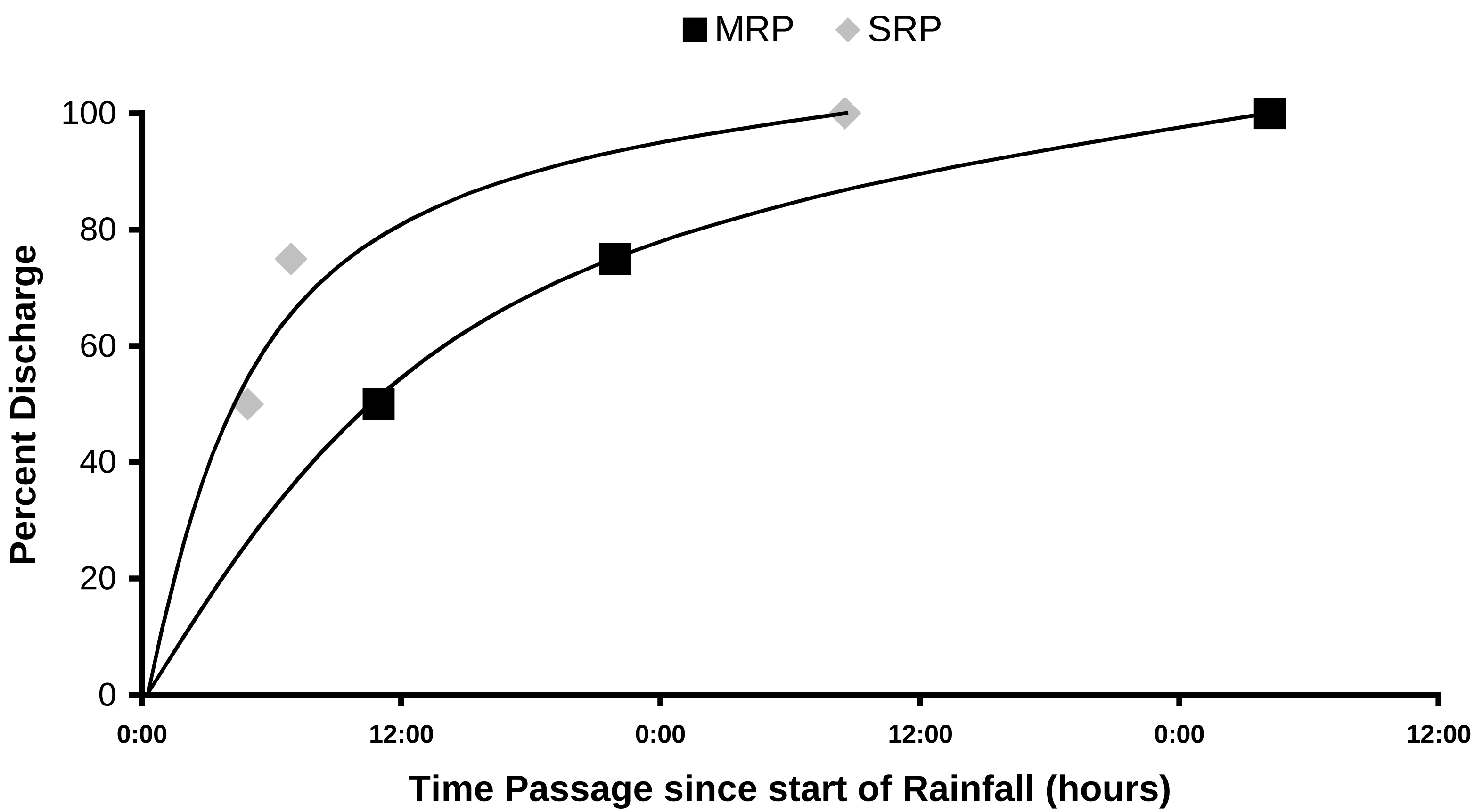


Figure 24. Average time for water to discharge from MRP and SRP.

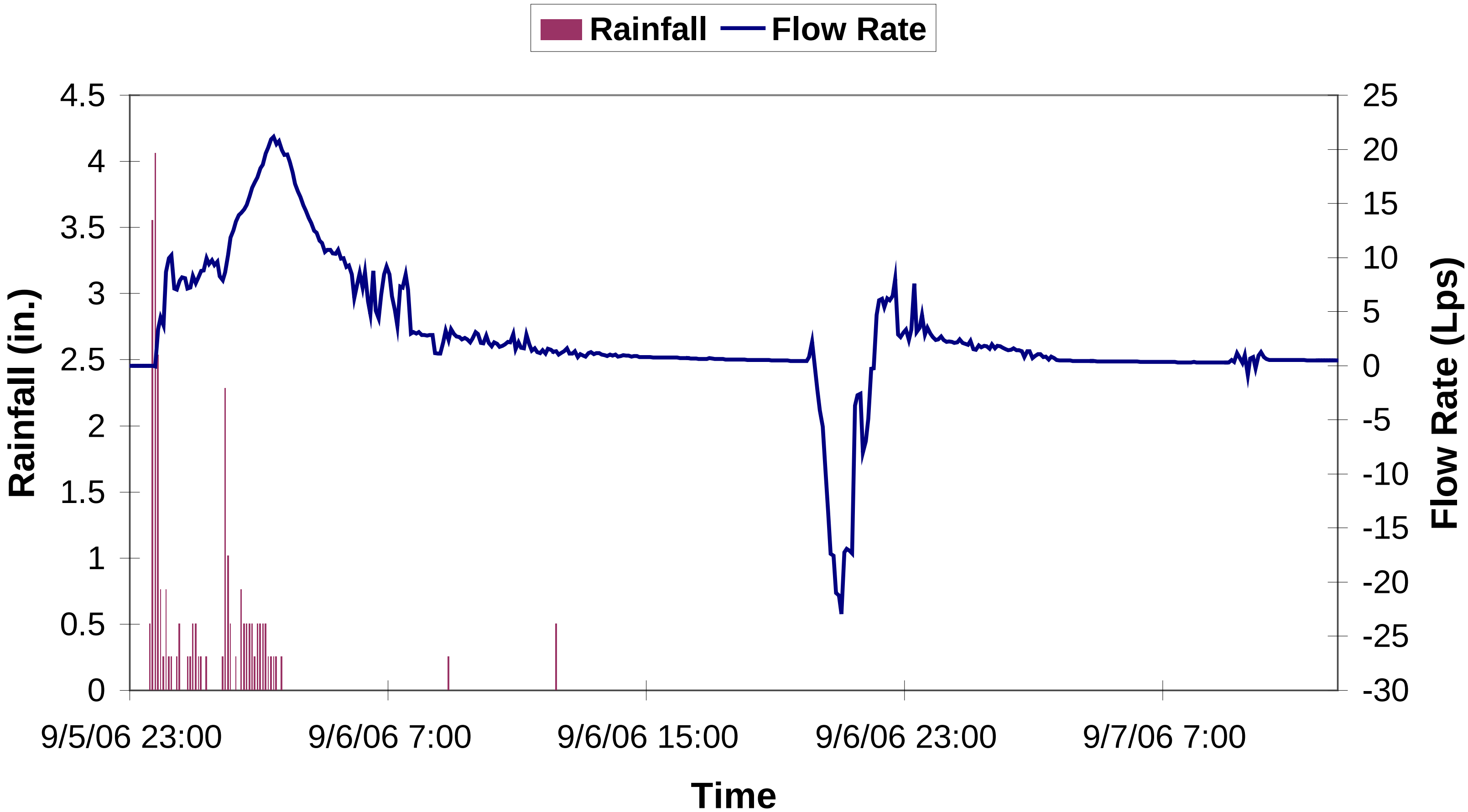
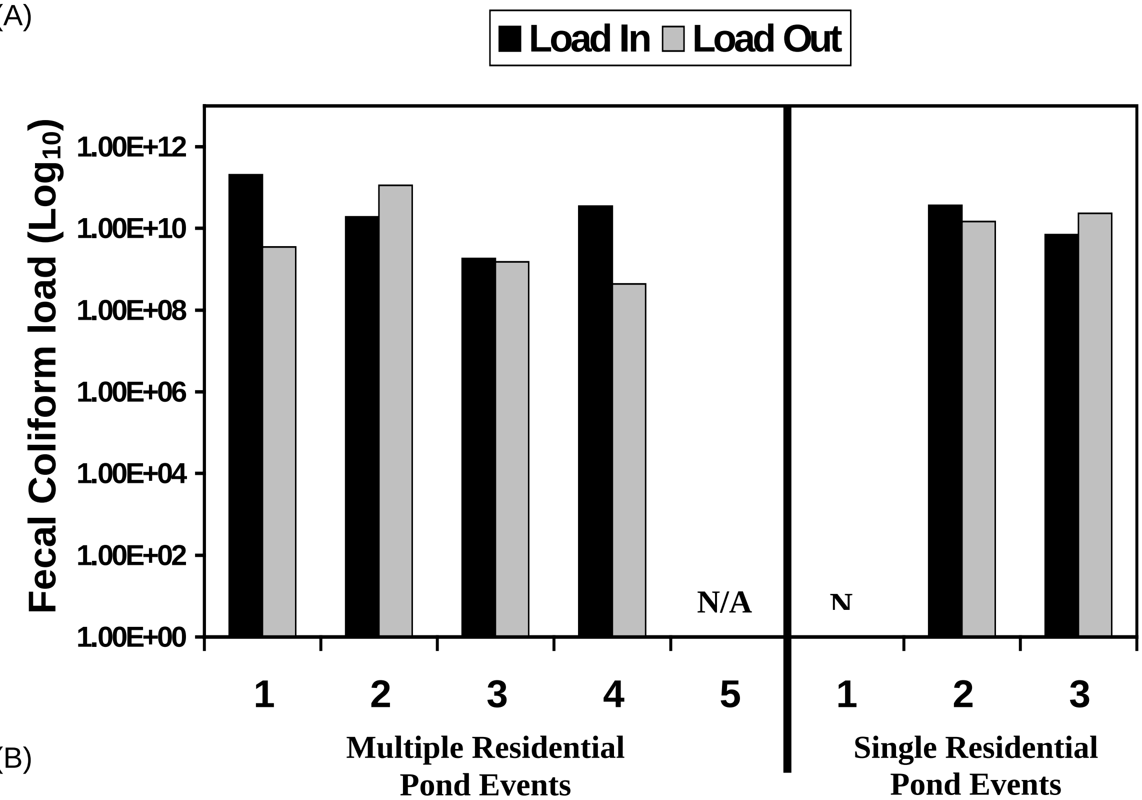


Figure 25. An example hydrograph displaying the negative flow rates occurring as a result of a spring tide during event 2 in SRP.

(A)



(B)

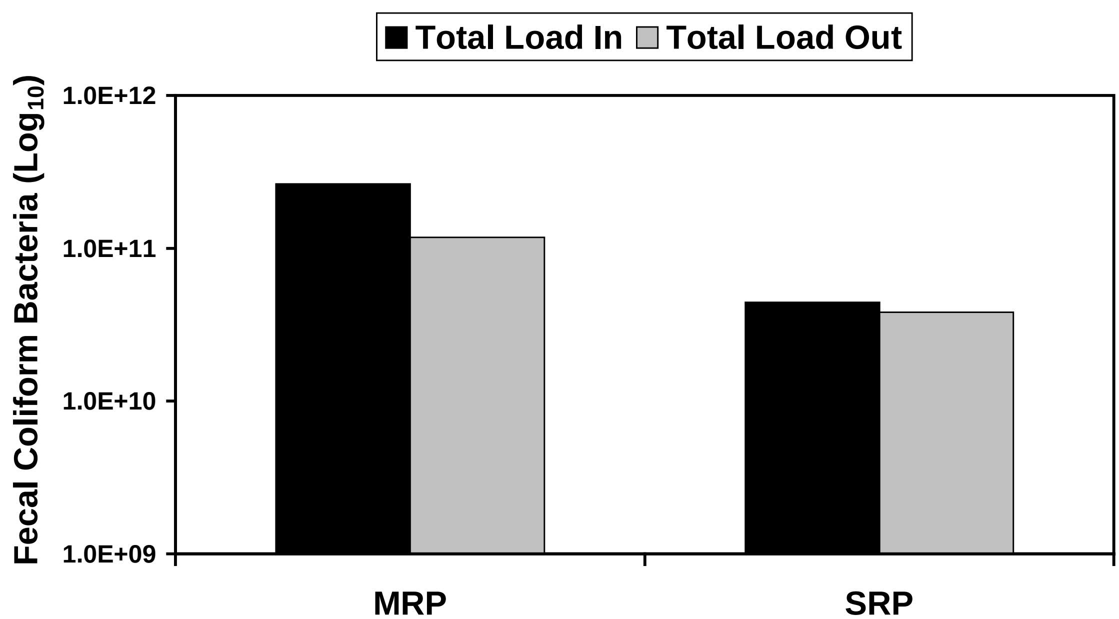


Figure 26. (A) Comparison of event inflow and outflow loads of fecal coliform bacteria (FC).

(B) Comparison of the total load of FC in and out from each pond over all rain events.

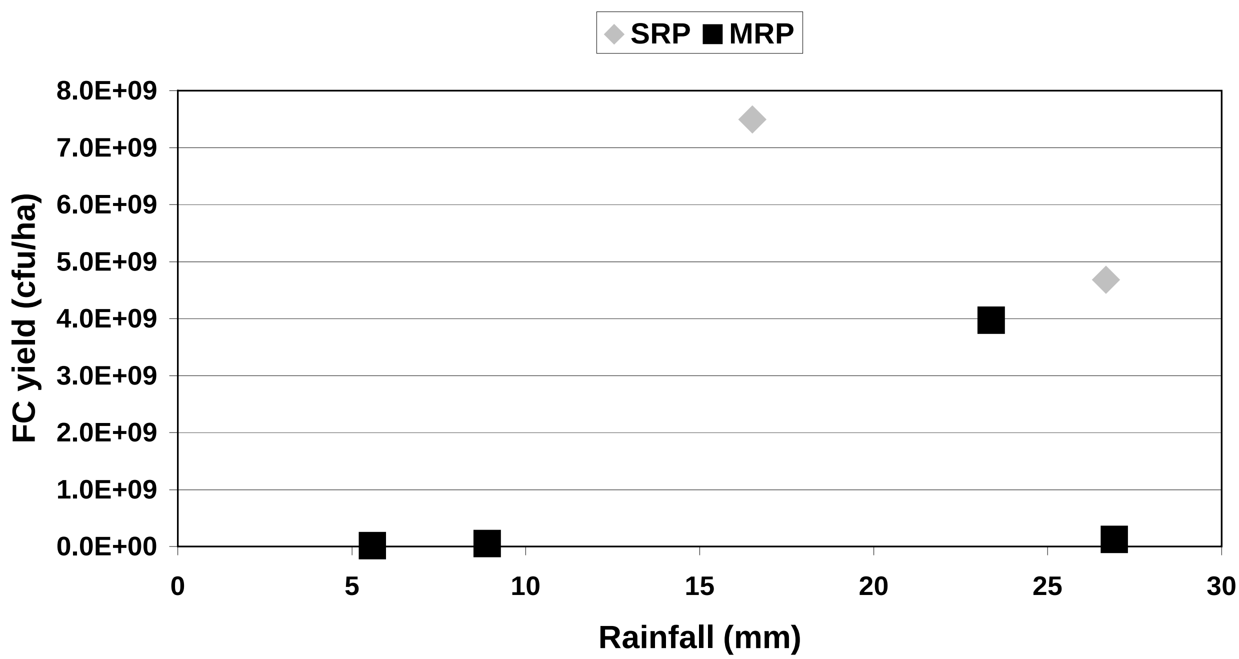
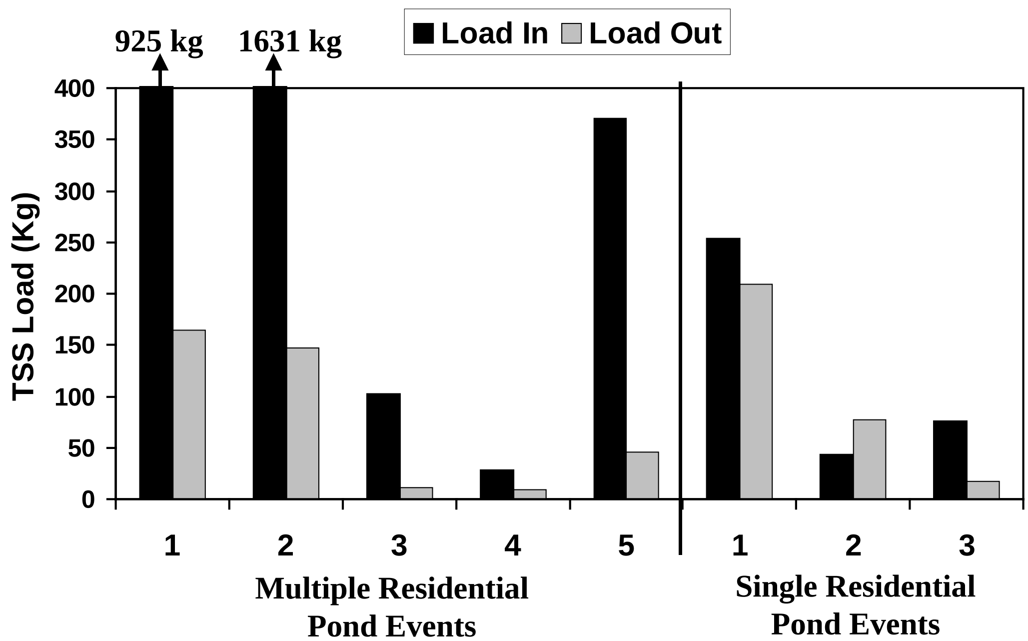


Figure 27. FC event yields from the drainage basins of MRP and SRP.

(A)



(B)

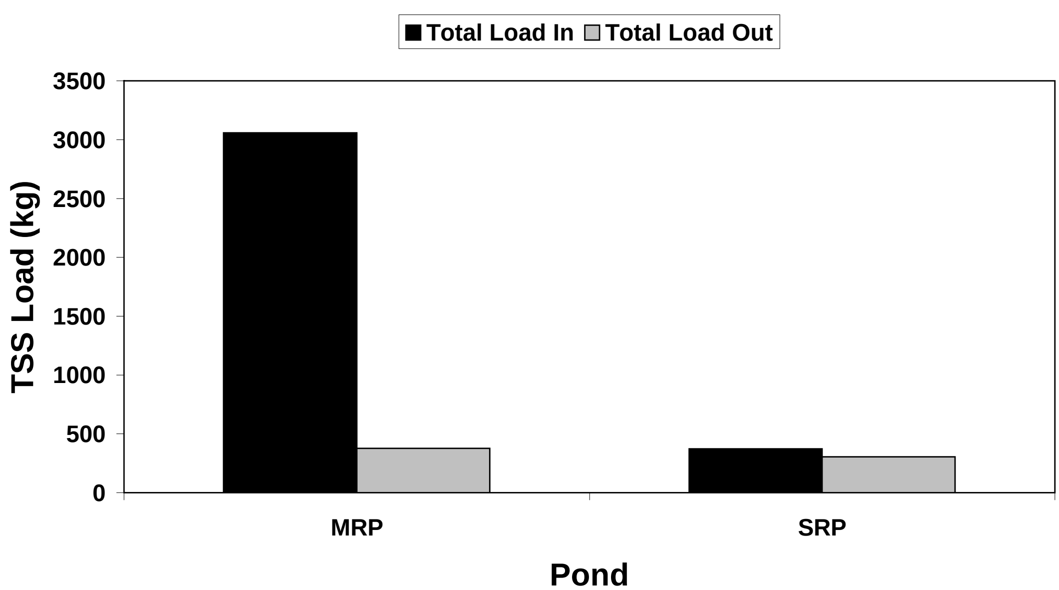


Figure 28. (A) Comparison of event inflow and outflow loads of total suspended solids (TSS). (B) Comparison of the total load of TSS in and out from each pond over all rain events.

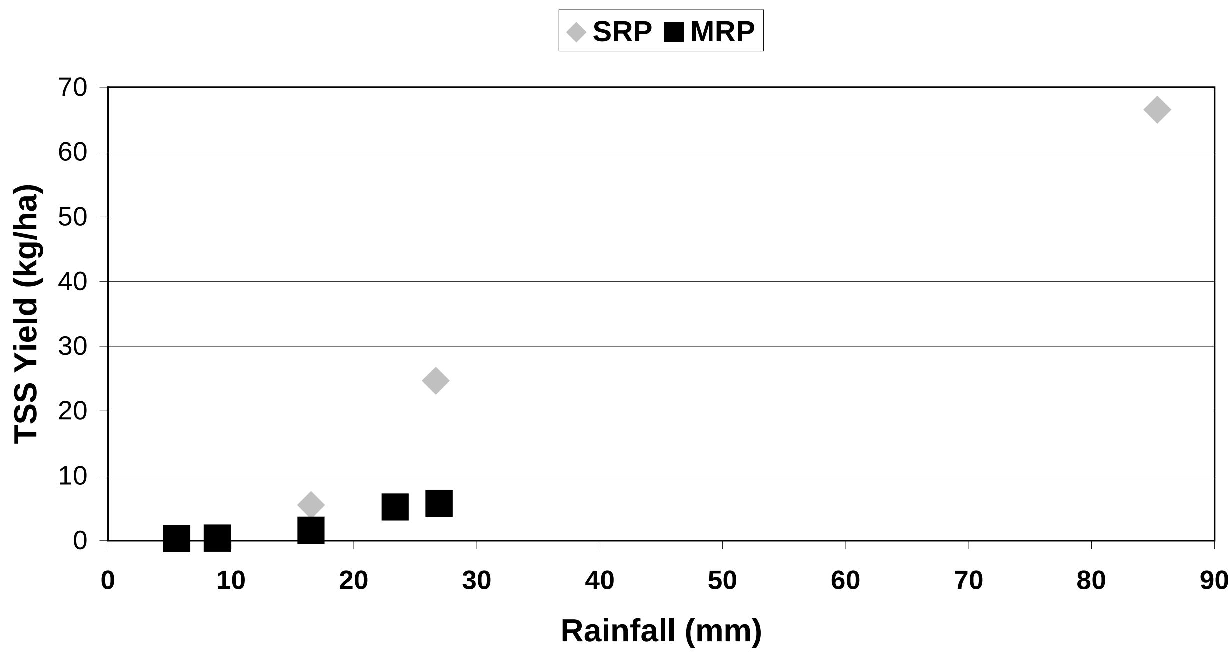
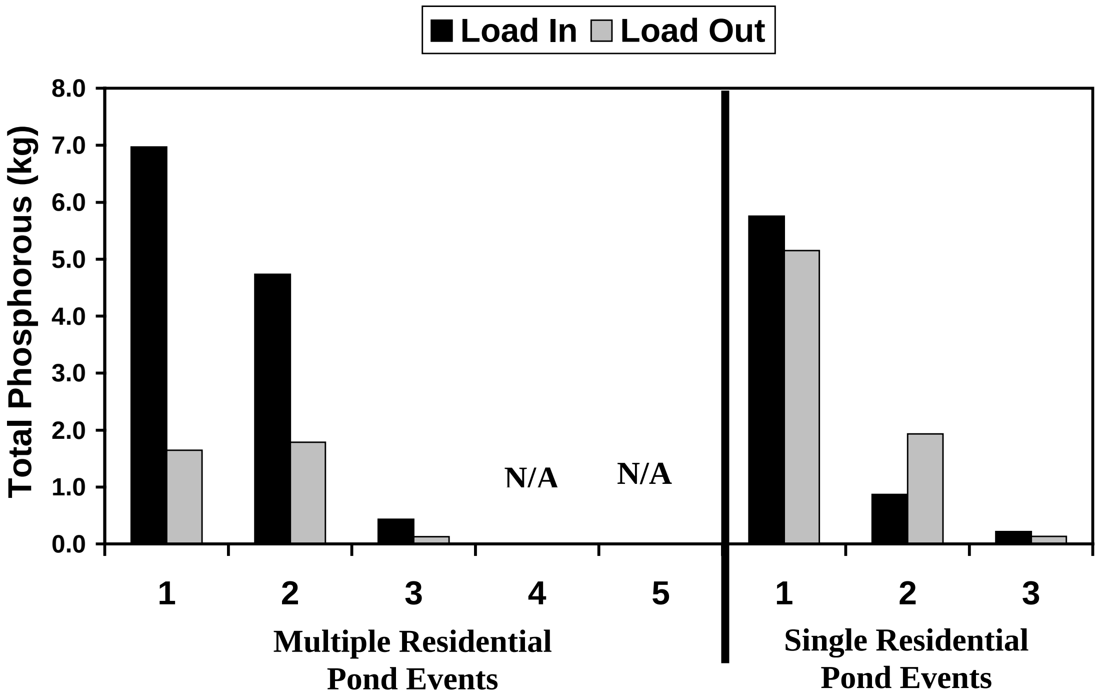


Figure 29. TSS event yields from the drainage basins of MRP and SRP.

(A)



(B)

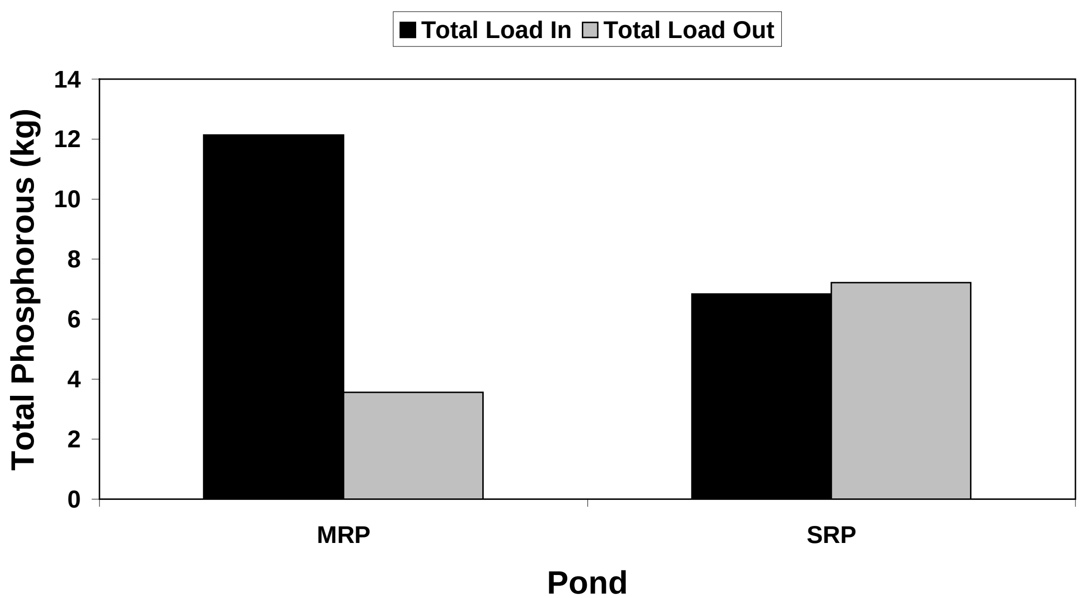


Figure 30. (A) Comparison of event inflow and outflow loads of total phosphorus (TP). (B) Comparison of the total load of TP in and out from each pond over all rain events.

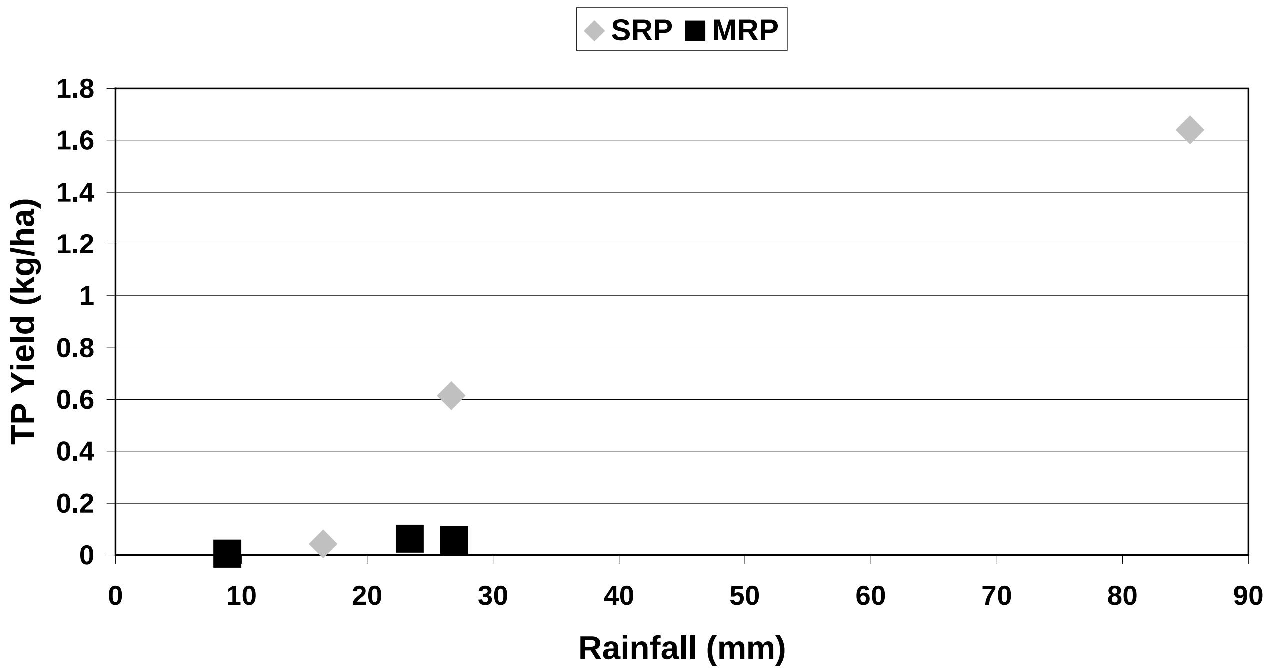
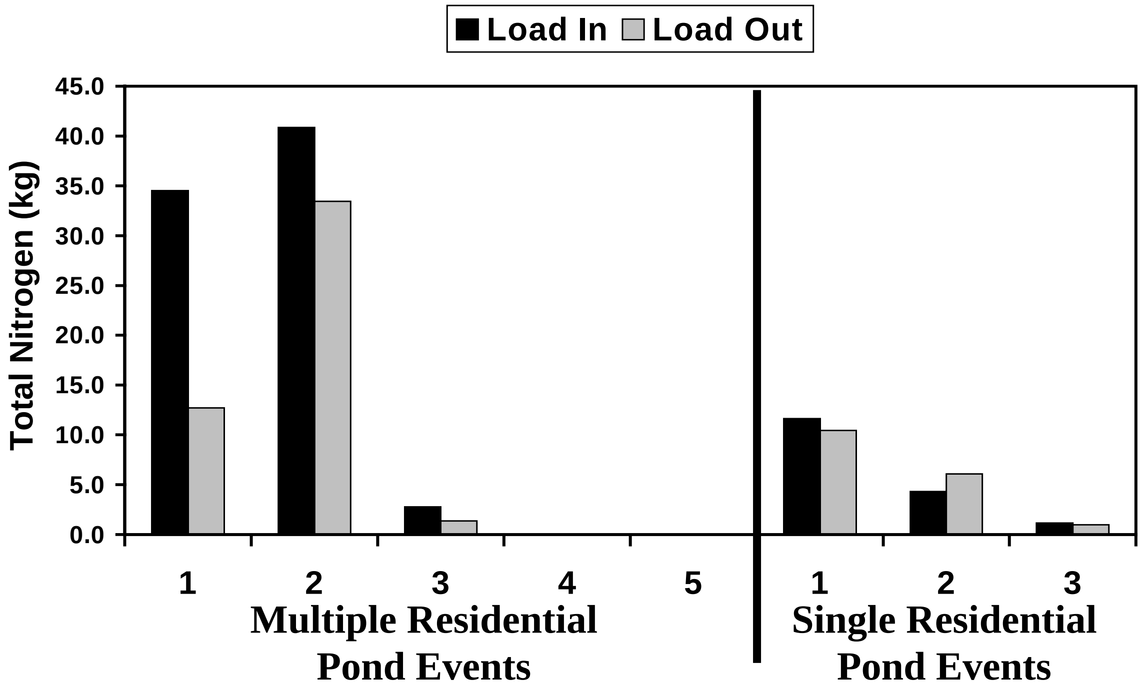


Figure 31. TP event yields from the drainage basins of MRP and SRP.

(A)



(B)

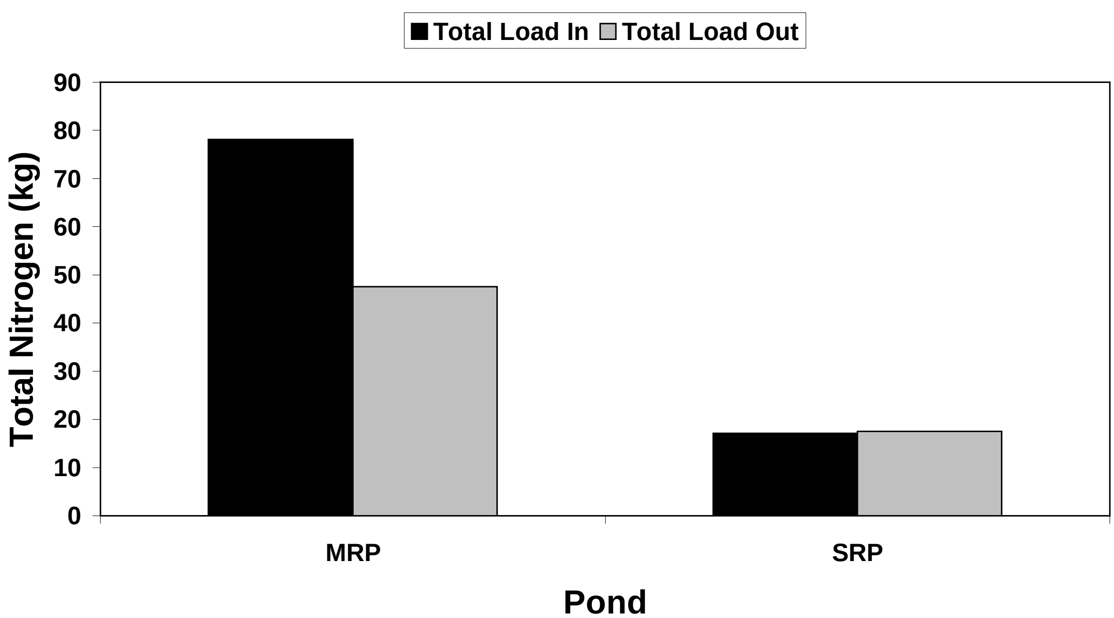


Figure 32. (A) Comparison of event inflow and outflow loads of total nitrogen (TN). (B) Comparison of the total load of TN in and out from each pond over all rain events.

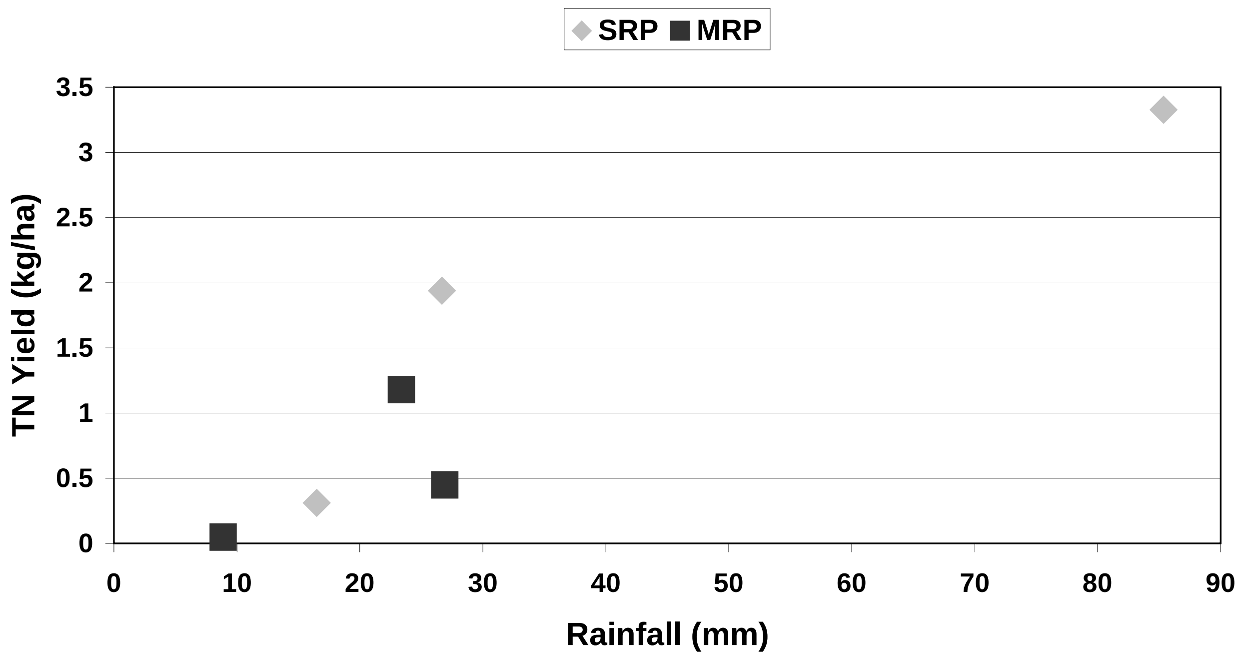


Figure 33. TN event yields from the drainage basins of MRP and SRP.

APPENDICES

Appendix A. Raw water quality data from MRP. All concentrations represented as mg/L, except for FC (cfu/100mL)

Station	Event	FC	TSS	TP	TDP	NO3+2	NH4	TKN	NH4 diss	NO3+2 diss	OP diss	TKN diss
mrp_pre	6/3/2006	<20	8	0.1	0.09	<0.02		1	0.1	<0.02	0.01	0.5
mrp_in1	6/3/2006	>16000	37	0.32	0.13	0.04		2.2	<0.1	<0.02	0.09	1.1
mrp_in2	6/3/2006	5000	70	0.47	0.18	<0.02		1.4	0.4	<0.02	0.17	0.9
mrp_out	6/3/2006	170	8	0.08	0.03	<0.02		0.6	0.1	<0.02	<0.01	0.5
mrp_24	6/3/2006	1700	19	0.11	0.06	<0.02		1.0	0.1	<0.02	0.03	0.6
mrp_48	6/3/2006	170	5	0.09	0.04	<0.02		0.7	<0.1	<0.02	<0.01	0.6
mrp_pre		2	14	0.18	0.04	<0.02		0.8	<0.1	<0.02	<0.01	0.6
mrp_pre	7/6/2006	30	11	0.07	<0.01	0.03	<0.1	0.8	<0.1	<0.02	<0.01	0.5
mrp_in1	7/6/2006	1300	24	0.18	0.07	0.02		1.8		<0.02	0.03	
mrp_in2	7/6/2006	1700	203	0.51	0.1	0.41	0.6	3.8	0.4	0.29	0.07	0.6
mrp_out	7/6/2006	16000	11	0.1	0.02	0.03	0.1	3.4	0.1	<0.02	<0.01	0.5
mrp_24	7/6/2006	1700	14	0.15	0.04	<0.02	0.1	0.8	0.1	<0.02	0.01	0.6
mrp_48	7/6/2006	300	10	0.13	0.02	0.02	<0.1	1.3	<0.1	0.02	0.02	0.6
mrp_out	7/6/2006	80	9	0.14	0.05	<0.02	0.6	1.2		<0.02	<0.01	0.6
mrp_pre	7/26/2006	2	7	0.08	0.05	<0.02	<0.1	0.8	<0.1	<0.02	<0.01	0.7
mrp_in2	7/26/2006	1100	61	0.26		0.46	<0.1	1.2				
mrp_out	7/26/2006	700	5	0.06	0.02	<0.02	<0.1	0.6	<0.1	<0.02	<0.01	0.6
mrp_24	7/26/2006	110	8	0.12	0.02	<0.02	<0.1	0.6	<0.1	<0.02	<0.01	0.5
mrp_in2	7/27/2006	>16000	13									
mrp_out	7/27/2006	350	4	0.03	0.02	<0.02	<0.1	0.6	<0.1	<0.02	<0.01	0.5
mrp_out	7/27/2006	30	4	0.04	0.02	<0.02	<0.1	0.6	<0.1	<0.02	<0.01	0.5
mrp_24	7/27/2006	70	4	0.12	0.10	<0.02	<0.1	0.8	<0.1	<0.02	0.01	0.4
mrp_48	7/27/2006	2	6	0.05	0.01	<0.02	<0.1	2.2	<0.1	<0.02	0.01	0.4
mrp_in2	7/29/2006	500	117	0.35	.11	0.15	<0.1	1.6	<0.1	<0.02	0.1	0.3
mrp_out	7/29/2006	300	6	0.05	0.01	<0.02	<0.1	1.4	<0.1	<0.02	<0.01	0.4
mrp_48	7/29/2006	80	5	0.06	0.03	<0.02	0.2	1.8	0.2	<0.02	0.01	0.3
mrp_in1	7/29/2006		15									

Appendix B. Raw water quality data from SRP. All concentrations represented as mg/L, except for FC (cfu/100mL)

Station	Event	FC	TSS	TP	TDP	NO3+2	NH4	TKN	NH4 diss	NO3+2 diss	OP diss	TKN diss
srp3_pre	8/31/2006	140	8	0.24	0.2	<0.02	<0.1	1.6	<0.1	<0.02	0.16	0.7
srp3_in2	8/31/2006	>1600	36	0.42	0.32	0.04	0.1	0.8	0.1	0.04	0.32	0.8
srp3_in1	8/31/2006	>1600	17	0.42		0.08	0.2	0.8				
srp3_out	8/31/2006	>1600	15	0.37	0.31	0.05	0.2	0.7	0.2	0.04	0.3	0.7
srp3_in2	8/31/2006	>1600	9									
srp3_24	8/31/2006	>1600	9	0.35	0.32	0.03	0.2	0.8	0.2	0.02	0.32	0.8
srp3_48	8/31/2006	300	12	0.38	0.33	<0.02	0.2	0.8	0.2	<0.02	0.33	0.8
srp3_pre	9/6/2006	23	11	0.34	0.28	<0.02	<0.1	0.8	<0.1	<0.02	0.29	0.5
srp3_in1	9/6/2006	500	6									
srp3_in2	9/6/2006	16000	17	0.29	0.26	0.14	0.2	1.3	0.1	0.14	0.26	1.3
srp3_out	9/6/2006	3000	24	0.58	0.3	<0.02	0.4	1.8	0.3	<0.02	0.3	1.2
srp3_out	9/6/2006	3000	12	0.35	0.29	0.03	<0.1	1.1	<0.1	<0.02	0.29	0.6
srp3_24	9/6/2006	1700	13	0.34	0.31	<0.02	<0.1	1.2	<0.1	<0.02	0.27	0.7
srp3_48	9/6/2006	700	18	0.24	0.01	<0.02	<0.1	1.1	<0.1	<0.02	0.21	0.6
srp3_out	9/6/2006	16000	12									
srp3_pre	9/13/2006	60	33	0.12	0.02	<0.02	<0.1	1.12	<0.1	<0.02	0.02	0.6
srp3_in1	9/13/2006	>16000	136									
srp3_in2	9/13/2006	300	21	0.17	0.06	0.02	0.2	0.9	0.2	0.02	0.06	0.8
srp3_out	9/13/2006	>16000	12	0.09	0.06	0.04		0.6		0.04	0.06	0.3
srp3_out	9/13/2006	>16000	10	0.09	0.08	0.05	0.1	0.8	<0.1	0.05	0.08	0.5
srp3_24	9/13/2006	9000	11	0.14	0.11	0.05	<0.1	0.8	<0.1	0.05	0.08	0.5
srp3_48	9/13/2006	230	8	0.1	0.14	0.03	0.2	0.8	<0.1	0.02	0.09	0.6
srp3_pre	9/19/2006	2	21	0.05	0.04	<0.02	<0.1	0.7	<0.1	<0.02	0.03	0.5
srp3_24	9/20/2006	<20	18	0.13	0.06	<0.02	0.1	0.9	0.1	<0.02	0.06	0.6
srp3_48	9/21/2006	8	21	0.13	0.06	<0.02	<0.1	1.2	<0.1	<0.02	0.04	0.7