ABSTRACT

MAZER, KATY E. Converting a Dry Pond to a Constructed Stormwater Wetland to Enhance Water Quality Treatment. (Under the direction of Dr. William F. Hunt, III).

Stormwater pollution degrades drinking water quality, impairs ecosystems, and can have negative economic impacts. Over the past 30 years, water quality treatment of runoff has become an integral part of stormwater management. Second generation stormwater control measures (SCMs) are designed with pollutant removal mechanisms that treat runoff. Dry ponds, which poorly treat water quality, are older SCMs that were used prior to water quality control becoming a priority. However, a dry pond's simple design provides ample opportunity to make alterations within the structure that enhance water quality treatment.

In this study, three dry ponds were monitored for water quality treatment capability and the potential for treatment enhancement through basin alterations, or retrofits. Two dry ponds, MOV1 and MOV2, were studied as a paired watershed in a residential area in Morrisville, North Carolina. Directly connected impervious areas and underlying soils (HSG D) for both catchment areas were similar. A third dry pond, WS, was monitored at an elementary school in Winston-Salem, North Carolina, with a mostly pervious watershed. Beginning in February 2017, all ponds were monitored for influent and effluent water quality of total suspended solids (TSS), total phosphorus (TP), orthophosphate ($O\text{-}PO_4^{3-}$), total nitrogen (TN), total ammoniacal nitrogen (TAN), total Kjeldahl nitrogen (TKN), and organic nitrogen (ON). Significant water quality changes between inlet and outlet in MOV1 were only detectable for TP, TN, and ON, where effluent loads *exceeded* influent loads. TP and $O\text{-}PO_4^{3-}$ loads were also *exported* from MOV2. In WS, there were significant load reductions of 43%, 61%, and 49% for TSS, TN, and TP, respectively. Concentrations discharged by MOV1 and MOV2 were higher than those typical of

other SCMs, including dry ponds, while concentrations at WS were comparable to those of other dry ponds.

After water quality treatment of each dry pond was assessed for existing conditions, MOV1 and WS were retrofitted to incorporate features of constructed stormwater wetlands (CSWs) in September 2017 and March 2018, respectively. MOV2 remained un-retrofitted as a control pond for MOV1. Each pond's outlet was modified to create a permanent pool in the basin, and wetland vegetation was installed. At MOV1, the outlet orifice's diameter was also reduced to increase detention time. Water quality monitoring of all ponds continued post-retrofit, and data from both monitoring periods were used to detect changes in treatment in these ponds-turned-wetlands using analysis of covariance (ANCOVA). WS's post-retrofit sample size was too small (n = 4) to detect significant changes. At MOV1, significant improvement in effluent concentrations and annual loads were found; post-retrofit annual effluent loads were reduced from those of pre-retrofit by 89%, 60%, 57%, 71%, 75%, 69%, and 75% for TSS, TP, O-PO_4^{3-}, TN, TKN, TAN, and ON, respectively. There was no significant difference in treatment of $NO_{2,3}$-N, though effluent concentrations of $NO_{2,3}$-N only exceeded target effluents in 56% of storms. Additionally, during periods of baseflow, there was evidence of significant $NO_{2,3}$-N reduction, which is characteristic of wetland systems. Effluent concentrations in MOV1 were lower than those of other dry ponds but remained higher than those of CSWs.

This dry pond retrofit, which is low-cost and easy to implement, appears to have major water quality benefits. Such a simple design could easily be replicated in many other dry ponds throughout the country. Minimal investment and effort at individual ponds could lead to large reductions in pollutant loads transported downstream and perhaps yield financial incentives to 'nutrient bankers' in regions of America where nutrient removal is required.

Converting a Dry Pond to a Constructed Stormwater Wetland to Enhance Water Quality Treatment

by
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DEDICATION

"Never let success get to your head and never let failure get to your heart."

-Zaid K. Abdelnour

To learning how to fail gracefully and allowing those lessons to lead us to greatness.

BIOGRAPHY

Katy Elizabeth Mazer was born to Stephen and Mary Mazer on November 9, 1990. She spent her childhood playing outside and climbing trees. She loved nature, the outdoors, and had the spirit of an environmentalist. Even as a child, she would become upset about the repercussions of irresponsible development, like ecosystem degradation and a disruption of hydrologic processes. A geography class she took in ninth grade steered her towards a career in environmental stewardship. She graduated with a B.S. in Biological and Agricultural Engineering from NC State in 2013. Dr. Bill Hunt offered her a position as a graduate research assistant, but the call of the world was pulling her in a different direction. She spent two years as a water resources Peace Corps volunteer in Panama, assisting in the development of sustainable drinking water systems. Living out of her little thatched hut in Panama, Katy learned how to fail and how to talk to people to solve even the biggest of problems.

She then returned to NC State as a graduate student under Dr. Hunt and has spent many hours diving deep into dry pond literature and even deeper into work in the field. She was happy to be back in the heart of Raleigh, watching the city grow and spending nights out with friends. She looks to the future, where she will use the skills she learned at NC State to make a difference in water resources and in the lives of the people she encounters.

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I feel very grateful for having had the opportunity to return to NC State to pursue my M.S.; it would not have been possible without the help and support of many people. I'd like to start by thanking Dr. Bill Hunt, whose advising and guidance have been integral in shaping my career. I appreciate your passion for your work, the contagious enthusiasm you bring to even mundane tasks, and the broad perspective you bring to research goals. Thank you for keeping your expectations high and for feedback you've provided, which have helped me grow as a professional. I would like to thank Dr. Mike Burchell and Dr. Mike Vepraskas for serving on my committee and providing guidance on creating a well-rounded research package. This project was made possible through support from the Clean Water Management Trust Fund. I would also like to thank project collaborators, including the Town of Morrisville, Forsyth County Schools, and Wendi Hartup. I'd like to express special gratitude to Josh Baird, who helped push this project along and provided an industry perspective to the intricacies of the retrofit. Thank you to Shawn Kennedy, whose skills with tools and ingenuity of design make up the cornerstone of the field work we do. To Sarah Waickowski, I am continuously astounded by your hard work, ability to multi-task, and your responsiveness to questions and concerns that I had. Thank you for modeling what it's like to be on top of things. I would also like to thank Mitch Woodward for being both a willing volunteer and an eager mentor to satisfy my extension interests.

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1.1: Introduction

Currently, the world's population is 7.3 billion and is expected to reach 9.7 billion by 2050 (UN, 2017). Of the current population, approximately 54% live in urban areas. By 2050, that number will grow to 66% (UN, 2014). With increased urban growth comes the need for more housing, infrastructure, and amenities, such as schools and grocery stores. All of these lead to a change in land use, from pervious to impervious surfaces, impacting vegetation, wildlife, and the hydrologic cycle.

The main components of the hydrologic cycle are precipitation, evapotranspiration, infiltration (both deep and shallow), and runoff. The hydrologic cycle also includes interception, condensation, percolation, and depression storage (NOAA NRFC, 2018). In a forested landscape, the majority of precipitation is infiltrated or evapotranspired, and only 10% becomes runoff (NRCS, 1998). As the impervious percentage of a watershed increases, so does the fraction of runoff (*Figure 1-1*, NRCS, 1998) and, consequently, increased imperviousness yields hydrographs with higher peak flows and shorter lag times, a greater total volume of runoff, and shorter runoff duration overall (Leopold, 1973; Leopold, 1994). These "flashier" flows cause increased flood frequency, increased erosive forces in streams, and decreased baseflows during inter-event periods (Hammer, 1972; Hollis, 1975; Leopold, 1973).

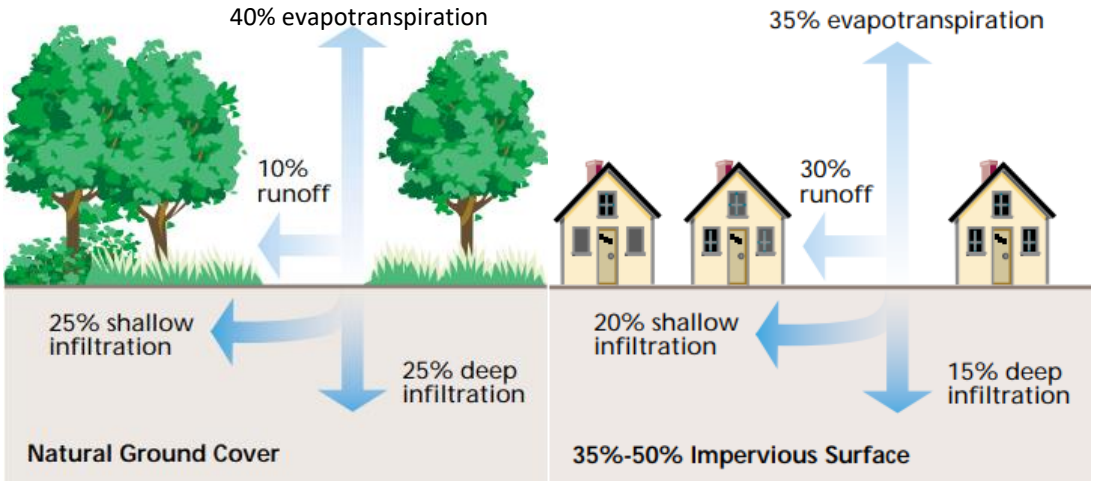

Figure 1-1: Increased impervious cover changes the hydrologic cycle, and more precipitation is converted to runoff (modified from NRCS, 1998).

1.2: Pollution in Stormwater

Urbanization impairs water quality, from land disturbance and sediment buildup to an introduction of anthropogenic materials that do not exist in a natural watershed (Line et al., 2002; Line and White, 2007). Runoff from urban areas alters stream flows and causes stream bank erosion, sediment deposition that degrades macroinvertebrate habitat, increased water temperatures, and added nutrient loads (Paul and Meyer, 2001; Walsh et al., 2005). An overabundance of nutrients, or eutrophication, triggers algal blooms, which in turn lead to decreased dissolved oxygen concentration in the water column and, ultimately, fish kills. Heavy metals accumulate in the biomass of fish (Rashed, 2001; Vinodhini and Narayanan, 2008), which can render them unsafe for human consumption. Fish health has a direct impact on fishing-based economies. Aquatic species are also negatively impacted by species loss through toxicity of polluted stormwater and disruption of the food chain (Halder and Islam, 2015). Increased water temperatures have been linked to poor trout survivability (Coutant 1977).

Stormwater pollution creates unsafe conditions for recreation activities. Pathogens transported by stormwater can harm humans. Swimming in polluted waters can cause rashes, infections, exposure to harmful bacteria, and other health problems (Halder and Islam 2015).

Stormwater pollution comes from two main sources: atmospheric deposition and land-based sources. Atmospheric deposition is composed both of dry deposition (dustfall) and wet deposition. Barkdoll et al. (1977) found dustfall to be the main contributor of COD, Hg, Cl, As, and PO_4 in stormwater. Andren and Lindberg (1977) found that 16.5% or greater of the total contaminant load for Cr, Hg, Mn, and Pb was due to dry deposition. Atmospheric deposition was the source of 20% of TSS, 70-90% of nitrogen, and 29% of total phosphorus loads in highway stormwater runoff in the piedmont of North Carolina (Wu et al., 1998). Line et al. (2002) found that, over several types of urban land uses, pollutant loading rate in rainfall ranged from 44 – 287% of that in runoff for $NO_{2,3}$-N and between 104 – 951% for TAN. Percentages over 100 are indicative of an overall use or conversion of these N species in the watershed through processes like plant uptake. Much of atmospheric deposition comes from anthropogenic sources (Bertine and Goldberg 1971).

Though atmospheric deposition contributes to pollutant loads, land-based anthropogenic sources are a major source of pollution in natural water bodies. Pollution contributed from point sources like factories and wastewater treatment plant effluents is highly regulated through the NPDES permit program (USEPA, 2017). Nonpoint source pollution typically occurs when runoff from a storm event carries pollutants that have been built up in areas throughout the watershed.

Although impervious surfaces themselves are not “pollution”, there is a strong link between the level of imperviousness and pollution of waterways (Arnold and Gibbons 1996).

Many pollutants build up on impervious surfaces. Impervious surfaces, particularly asphalt, have been shown to have temperatures exceeding 60° C (Diefenderfer et al., 2006). Therefore, runoff from impervious surfaces can reach 30° C, which cause temperature surges in streams (Jones and Hunt, 2009; Sansalone et al., 2005).

Other common pollutants in stormwater include total suspended solids (TSS), nitrogen, phosphorus, heavy metals, pathogens, pesticides, and hydrocarbons (Makepeace, 1995). Directly connected impervious area (DCIA) in a watershed effectively transports pollutants to streams (Hatt et al., 2004). In residential areas, roads and streets disproportionately contribute to total pollution in a watershed (Bannerman et al., 1993).

Increased contaminant concentrations may affect drinking water reservoirs. This can cause risks to public health and aquatic ecosystems (Makepeace, 1995). Line et al. (2002) found that export loads of TSS in runoff from construction zones were between 6.5 – 23 times greater than TSS loads in wooded areas. In the same study, export loads for TN in developed areas were more than 250% higher than those of wooded areas. Pb, Zn, and Cu concentrations are higher in urban watersheds than in forested watersheds (Helsel et al., 1979).

Goonetilleke et al. (2004) monitored six types of catchments, with impervious areas ranging from 9% to 70% of the watershed. They found that percent impervious cover was not directly correlated to pollutant loads. Land use type also had an effect on pollution loads. Pollutant concentrations from a watershed of single-family homes was much greater than that of multifamily units. The abundance of landscaped areas around the single-family homes, which use nutrient-heavy fertilizer, likely contributed to this increased pollution.

Line and White (2007) monitored two similar drainage areas in the piedmont of North Carolina over 5 years. During this time, one of the watersheds was developed into a residential

area, while the other remained woods and agricultural land. Approximately 55% of the rainfall in the developed watershed was converted to runoff, while only 21% of rainfall from the undeveloped watershed was converted to runoff. Total runoff volumes for the developed area were 68% greater than from the undeveloped area. In the urban watershed, annual loads of TSS, TP, and TN were 95%, 74%, and 69% greater, respectively, than those in the undeveloped watershed.

Pollutants, like heavy metals and phosphorus, can be sediment-bound. Line et al. (2002) found that total phosphorus loads exported from urban areas ranged between 30% and 430% greater than phosphorus loads leaving wooded areas.

1.3: Stormwater Policy and Regulation

The first law enacted in the United States to regulate water pollution was the Federal Water Pollution Control Act (FWPCA) in 1948. The Clean Water Act (CWA) expanded the FWPCA in 1972 (USEPA, 2017). The CWA's original objective was to restore and maintain "the chemical, physical, and biological integrity of the nation's waters." The CWA regulates point source pollution, requiring industrial, municipal, or commercial facilities to obtain a National Pollutant Discharge Elimination System (NPDES) permit to discharge directly to surface waters (USEPA, 2017). This permit aims surface water quality to ensure it is "fishable" and "swimmable." An amendment to the CWA in 1987 added nonpoint source pollution, including stormwater runoff, to NPDES permitting (Copeland 2016).

The amendment to the NPDES permit included stormwater runoff from industrial activities, separate storm sewers from municipalities (MS4), and construction activities. MS4s are sewer networks that discharge to surface water without treatment. In order to operate in a town or city with an MS4, an organization must obtain a NPDES permit and develop a

stormwater management program (SWMP). This SWMP creates an outline for how the MS4 will minimize pollution and incorporate stormwater control measures. Phase I MS4 (1990) permits include those for cities or some counties with populations greater than 100,000. The Phase II permit was initialized in 1999 and includes smaller and non-traditional MS4s (i.e. universities, hospitals, and prisons, etc.) (USEPA, 2017). MS4s are required to incorporate SCMs sized to achieve a "runoff treatment" for the first 25 mm of stormwater, or the "first flush volume" to treat sediment (NCDEQ, 2017a).

Additionally, some areas must also follow rules that pertain to Nutrient Sensitive Waters or Special Watershed Programs, which often limit discharge of nitrogen and phosphorus into these watersheds (NCDEQ, 2017b). Nutrient Sensitive Waters (NSWs) have been identified as areas needing extra nutrient management because they serve specific purposes like drinking water reservoirs or critical habitat. In NSWs, nutrient limits are applied to stormwater effluent from developments and can be met by treating runoff or purchasing nutrient offset credits from the NC Department of Mitigation Services or private mitigation banks (NCDEQ, 2018b). These offset credits can be costly and incentivize developers to implement stormwater treatment (NCDEQ, 2018a).

1.4: First Flush

Regulations commonly require water quality treatment of the first 25 mm of runoff (NCDEQ, 2017c; CWP, 2009; VDEQ, 2013). This is partially derived from the first flush phenomenon, which assumes the majority of pollutants exist in the first portion of runoff from a storm (Tucker 2007). There is no consensus in the literature on the exact definition of first flush. Many definitions are vague and broad, while others attempt to precisely identify the

phenomenon. *Table 1-1* reports first flush definitions from various authors and the existence of first flush according to each author(s)'s definition.

Figure 1-2 graphically depicts the ratio definition of first flush. First flush is considered to have occurred when the cumulative pollutant load ratio curve is positioned above the cumulative runoff ratio curve (*Figure 1-2, Line 1*). Line 2 depicts an even distribution of pollutants throughout an event, while Line 3 represents when runoff proceeds the majority of pollutant loads in an event (i.e. the opposite of first flush). The ratios described in *Figure 1*-2 can be defined as:

$$R_{runoff} = \frac{\sum_0^n V_{runoff}}{Total\ Storm\ Runoff}$$

where

R_{runoff}: Runoff ratio

V_{runoff}: Volume of runoff

n: Any point in the storm

This is also applicable to Rload. Many authors utilize these ratios in some form to describe first flush.

Table 1-1: Definitions of first flush (FF) found in the literature and the prevalence of this FF based on the various definitions.

Source	**Category**	**Definition**	**Findings**
Thornton and Saul (1987)	General	The initial period of storm flow during which pollutant concentration is significantly higher than in the latter stages	NA
EPA (1993)	General	some portion of total overflow determined to contain a major fraction of pollutant load	NA
Gupta and Saul (1996)	General	used definition provided by Thornton and Saul (1987)	FF was correlated with rainfall intensity, storm duration, and antecedent dry period
Line et al. (1997)	Time	first 30 minutes of runoff	Higher than median concentrations of heavy metals and nutrients in FF
Kim et al. (2007)	Time	first 20 - 50 minutes of storm	Concentration declines rapidly after first hour of storm
Saget et al. (1996)	Runoff fraction	80% of the mass transported in first 30% of runoff	FF of suspended solids occurred in 1 of 197 events
Stanley (1996)	Runoff fraction	majority of mass in first 20% of runoff	25% of pollutants present in FF
Deletic (1998)	Runoff fraction	first 20% of runoff	25.5% of suspended solids present in FF
Stenstrom and Kayhanian (2005)	Runoff fraction	first 20% of storm event	FF effect was greatest for Ni and lowest for $O\text{-}PO_4^{3-}$
Flint and Davis (2007)	Runoff fraction	50% of mass in the first 25% of runoff	Occurred in less than 1/3 of events for all pollutants
Helsel et al. (1979)	Ratio	cumulative load ratio exceeds cumulative runoff ratio	FF of heavy metals occurred in 90% of events
Geiger (1987)	Ratio	slope of (percentage total pollutant load)/(percentage cumulative flow) curve greater than 1	FF dependent on pollutant, existed for TSS
Sansalone and Buchberger (1997)	Ratio	similar to Helsel et al. (1979)	FF of Zn and Cu pronounced in sheet flow
Lee and Bang (2000)	Ratio	similar to Helsel et al. (1979)	FF occurred when impervious cover exceeded 80%
Lee et al. (2002)	Ratio	similar to Helsel et al. (1979)	Did not occur uniformly among pollutants

Although Bertrand-Krajewski et al. (1998) did not detect first flush using the definition provided by Saget et al. (1996), for at least half of events, 50% of the pollutant mass was transported in the first 38% of the volume. This aligns with the definition of first flush proposed by Helsel et al. (1979). Flint and Davis (2007) found in at least 17% of storms for all pollutants, larger loads were present in later portions of the storm than the first flush portion.

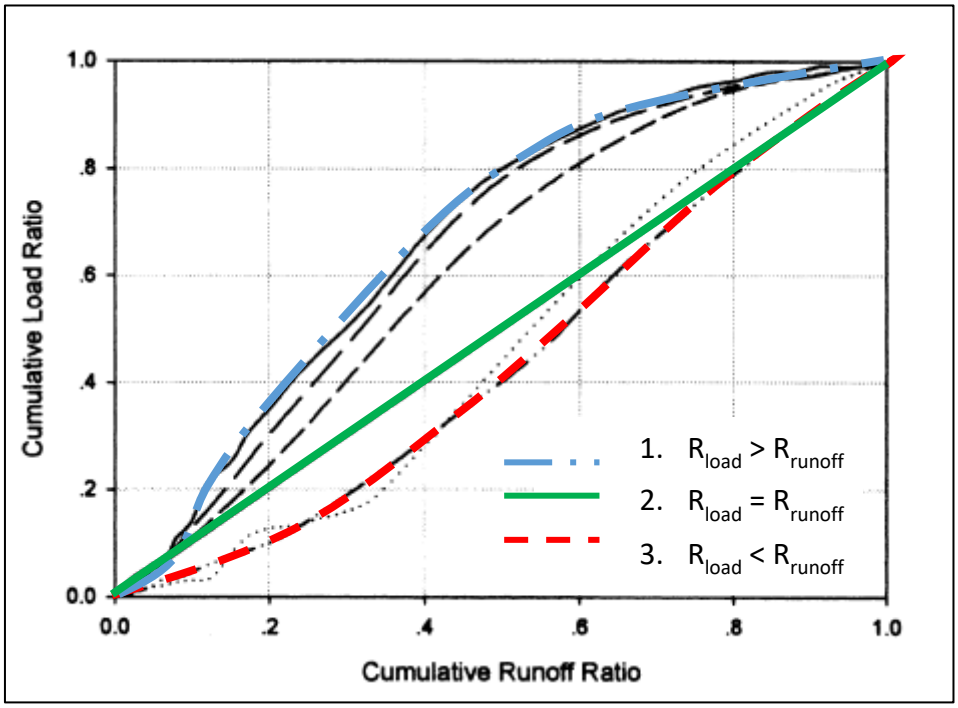

Figure 1-2: Cumulative runoff ratio plots pollutant load over time. When $R_{load} > R_{runoff}$, first flush occurs (Lee and Bang, 2000).

Often, multiple factors contribute to the existence of a first flush or an uneven distribution of pollutant loads per stormwater volume in general. In Deletic (1998), storms with larger runoff volumes had cleaner flows by the end of the storm than at the beginning. It is reportedly due to a lack of available sediment on the surface to be picked up in runoff after a certain point. Deletic also suggested that maximum rainfall intensity (I_{max}), time of occurrence of I_{max}, and land use area were factors contributing to pollutant load distribution throughout a storm. In a study of 14 catchments, both storm sewer and combined sewer, none of the tested basin

characteristics (time of concentration, slope, and active area) were found to have significant effects on pollutant loads throughout the catchments (Saget et al., 1996). The US EPA (1993) reported more frequent occurrence of first flush in small catchments, in areas with low impervious fractions, and with flat slopes. However, this study focused on combined sewers, which build up pollutants during dry period flows (FNDAE 1988). Some studies have shown that there is no noticeable difference in the first flush phenomenon between combined sewers and separate storm sewers (Saget et al., 1996).

Whether or not first flush exists, this ideology helps to identify treatment volume needed for sizing SCMs (Stanley, 1996). In NC, SCMs are designed to fully capture and treat the first 25 mm of rainfall (38 mm in coastal areas). In the Piedmont and Mountains of North Carolina, approximately only 20% of daily rainfall totals exceed 2.54 cm (*Table 1-2*). Thus, 100% of runoff from 80% of storms is treated annually (Bean, 2005). Even for storm events that exceed this rainfall depth, the first portion is still captured, under the principle that "first flush" of pollutants will occur. In this case, regardless of occurrence of first flush, SCMs capture and treat the majority of annual runoff.

Table 1-2: Calculated percent from 10 to 90 of daily rainfall depths for municipalities throughout North Carolina (Bean, 2005).

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

1.5: Treatment: Pollutant Removal Mechanisms

Structural SCMs are specially designed to remove pollutants in runoff by implementing one or more pollutant removal mechanisms (PRMs). PRMs are natural physical, chemical, or biological processes that reduce pollutant concentrations in runoff. Volume reduction, sedimentation, filtration, gross filtration, sorption, uptake, microbial processes, desiccation, and UV radiation are all types of PRMs utilized in SCMs.

Evapotranspiration (ET) is the process through which liquid water becomes vapor both from open water (evaporation) and the surface of plant material (transpiration). Main factors affecting evaporation rates are surface area of open water and length of time these water bodies are exposed to sunlight. Rates of transpiration increase with increased plant biomass. Evapotranspiration rates are highly dependent on the time of year and vary by SCM (Merriman et al., 2016; Rezaei et al., 2005).

Infiltration, the process by which water moves through pore space in the soil (NOAA NWFC, 2018), is the other mechanism for volume reduction in SCMs (Lenhart and Hunt 2011; Wilson et al. 2015). Infiltration rate is affected by underlying soil properties. Sandy soils have high infiltration rates that allow large volumes of water to pass through quickly. Soils in hydrologic soil group D, which are composed of at least 40% clay, have restricted water movement through the soil, and therefore, very low infiltration rates (NRCS, 2007).

Other stormwater practices temporarily store water for later use. In rainwater harvesting, rainwater is collected in a cistern and can then be used for irrigation or in-house water needs (Jones and Hunt, 2010). Water stored in wet ponds can also be used for irrigation (Camnasio and Becciu, 2011).

Each SCM serves particular sets of needs and uses specific PRMs to achieve these needs. Next generation SCMs, like bioretention cells and stormwater wetlands, are designed for newer stormwater management goals, including hydrologic matching and water quality improvement. However, some SCMs have more simple designs that implement fewer PRMs.

Sedimentation is a mechanism for settling particles out of water. Sedimentation occurs as velocity is decreased, when water loses the energy required to hold these particles (Greenway 2004). Settling velocity is dependent on particle size, with rates ranging from minutes for large particles to days for smaller particles (Garde, et al., 1990). In order for sedimentation to occur at this capacity, water must remain still for a duration of time. Particles settled will remain at the base of the water column, often in areas where velocity is suddenly decreased, like in pools. However, particles can be resuspended if the main flow reaches the area of sedimentation, which can be an issue particularly in areas without a permanent pool of water (Shammaa et al., 2002). Many pollutants, like petroleum hydrocarbons, metals, phosphorus (Ballestero et al., 2012), and pathogens can be attached to sediments and removed through sedimentation (Keraita et al., 2008).

Sediment can also be removed through filtration and gross filtration. Gross filtration is the filtration of gross solids, which are particles larger than 5 mm in diameter (Urban Water Resources Research Council, 2010). This process is achieved by obstructions that “sieve” water, like trash racks, orifices, and plants. have the ability to filter out gross solids. Filtration is the process by which obstructions remove smaller particles. In stormwater, filtration often refers to water passing through soil or an engineered media, where particles are trapped (Clark and Pitt, 2012; Davis et al., 2010). This process prevents smaller particles and pollutants bound to them

from entering streams. Fine particles may also adhere to sticky biofilms present on the surface of many emergent and submerged macrophytes (Greenway, 2004).

Sorption is the process by which soluble ions, including heavy metals and phosphorus, become bound to charged soil particles. Soluble phosphorus (orthophosphate), which is negatively charged, can be bound to negatively-charged soil particles through iron (Fe^{3+}) bridging (Clark and Pitt, 2012; Correll, 1998; Davis et al., 2001). Additionally, there is a limited capacity for soils to sorb cations or anions because of the limited number of charged soil particles. The P index assesses the ability of the soil to sorb additional phosphorus (Hardy et al., 2009). Fe^{3+} can bridge phosphorus anions to negatively charged soil particles. However, in highly anaerobic conditions, solid Fe^{3+} can be reduced to soluble Fe^{2+}, which breaks the bridge and releases ortho-P from the soil particle (Stumm and Morgan, 1996). This process can occur after the soil is inundated for 4-6 weeks (Greenway, 2004).

Plants that are present in SCMs have the ability to uptake nutrients, namely nitrogen and phosphorus (Greenway and Woolley 2001). Nutrient uptake can occur in the soil or built up sediment (Chambers et al., 1989) and the water column (Cooper and Cooke 1984; Vincent and Downes 1980), though always through plant roots (Carr 1998). Assimilation by algae also accounts for a large percentage of nutrient uptake (Bachand and Horne, 1999b). Greenway and Woolley (2001) monitored a constructed surface wetland for nutrient accumulation in plant biomass, and it was found that between 47 – 65% of P removal and 27 – 47% of N removal was due to plant uptake. Increased macrophyte biodiversity can increase nutrient uptake (Bachand and Horne, 1999b). Carbon is also sequestered from the atmosphere when it is accumulated as plant material in wetland biomass (Merriman et al., 2016). However, these nutrients can be released as plants decompose (Chimney and Pietro 2006).

There are a variety of processes carried out by microorganisms that improve water quality. Microorganisms play a large role in the nitrogen cycle through anaerobic respiration, which removes nitrogen from the water column. Inorganic NH_3-N is converted to $NO_{2,3}$-N through nitrification. Then, through a process known as denitrification, which requires a carbon source, anaerobic conditions, and denitrifying bacteria (Lefevre et al., 2015), $NO_{2,3}$-N is converted to N_2O or N_2 and released from the wetland (Lee et al., 2009). Denitrification occurs in anaerobic conditions, often in deeper sediment (Greenway, 2004) and is the most permanent form of nitrogen removal from a wetland. In a macrocosm study by Bachand and Horne (1999a), average nitrate removal was found to be between 1000 and 1500 mg NO_3-N/m^2/d, most likely due to denitrification. Microorganisms also oxidize many metal ions and reduce of sulfate ions, precipitating them out of the water column (Greenway, 2004).

Sunlight and dry areas in SCMs play a role in pollutant removal. UV radiation kills pathogens in stormwater exposed to sunlight (Davies and Evison 1991) On surfaces that dry out where microorganisms have accumulated, desiccation is a method of pathogen removal.

1.6: Stormwater Control Measures

This variety of pollutant removal mechanisms have been combined in many ways to create structures that remove pollutants. Both managerial and structural SCMs have been implemented to treat stormwater both by stormwater quantity reduction and stormwater quality improvement. Many structural SCMs were originally designed solely for flood control, mitigating peak flows of larger storm to match peak flows from predevelopment hydrographs. Much of stormwater infrastructure also conveyed water away from areas as quickly as possible without consideration for water quality or downstream impacts on stream channels, habitat, or biota. The goals for SCMs have changed; now post-development hydrology is more closely

matched with pre-development hydrology in what is known as low-impact development (LID) (Perrin et al., 2009). A principal component of LID is volume reduction of runoff. Essentially, these SCMs are designed to mimic hydrologic characteristics of natural systems, where the majority of water is evapotranspired or infiltrated, and little remains as runoff.

1.7: Dry Ponds

Dry ponds, also known as dry detention basins (*Figure 1-3, Figure 1-4*), are a type of SCM, constructed as a basin to capture and detain a certain volume of runoff with an outlet/orifice structure at the lowest elevation of the basin (NCDEQ, 2017c). Between storms, dry ponds completely dewater and remain dry during inter-event periods (Shammaa et al. 2002). Flood control dry ponds, an older design, were built solely for water quantity control by detaining runoff from 10- and 25-year storm events (USEPA, 1983). Because flood control ponds only controlled large quantities of water, the outlet orifices (*Figure 1-5*) were sized too large to detain runoff associated with smaller, more frequent storm events for an adequate duration (Goff and Gentry 2006), hence providing little treatment.

Figure 1-3: Typical dry pond located in a residential area in Morrisville, NC.

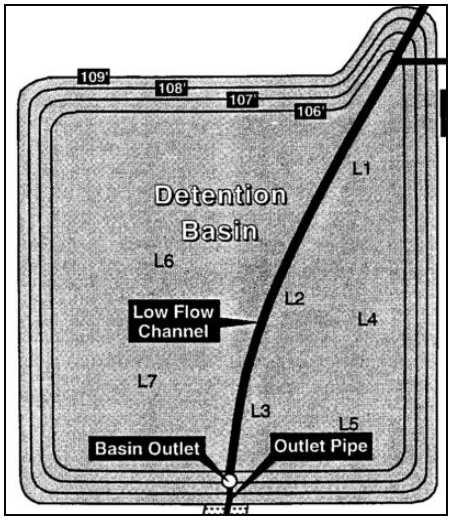

Figure 1-4: Basic design of a dry pond basin, though many do not include low flow channels (Guo, 1997).

Figure 1-5: The outlet orifice of a flood control dry pond does not greatly detain flows for smaller storm events, as it is relatively large (Carpenter et al., 2014).

In an attempt to control outflow from more stormwater events, extended detention dry ponds were created (U.S. Environmental Protection Agency, 1983). Extended detention dry ponds are designed to mitigate peak flow of multiple design storms (i.e. 2-yr, 5-yr, and 10-yr storms) (Goff and Gentry, 2006). This is achieved by creating a multi-level outlet (Shammaa et al., 2002, *Figure 1-6*). The outlet (*Figure 1-6, noteA*) sets the base water elevation and detains runoff from small storms. During larger storm events, water levels in the pond can reach the weir (*Figure 1-6, note B*) or the riser crest (*Figure 1-6, note C*) and begin to flow out at higher rates. These parts of the outlet structure are usually designed to match peak outflows to those of

predevelopment. By detaining water from small events, more effective removal of particle-bound pollutants occurs, particularly for smaller storms and the “first flush” of larger storms (Whipple 1981). Because flood control dry ponds do not manage this portion of a storm event, they are less capable of mitigating pollutants (U.S. Environmental Protection Agency, 1983).

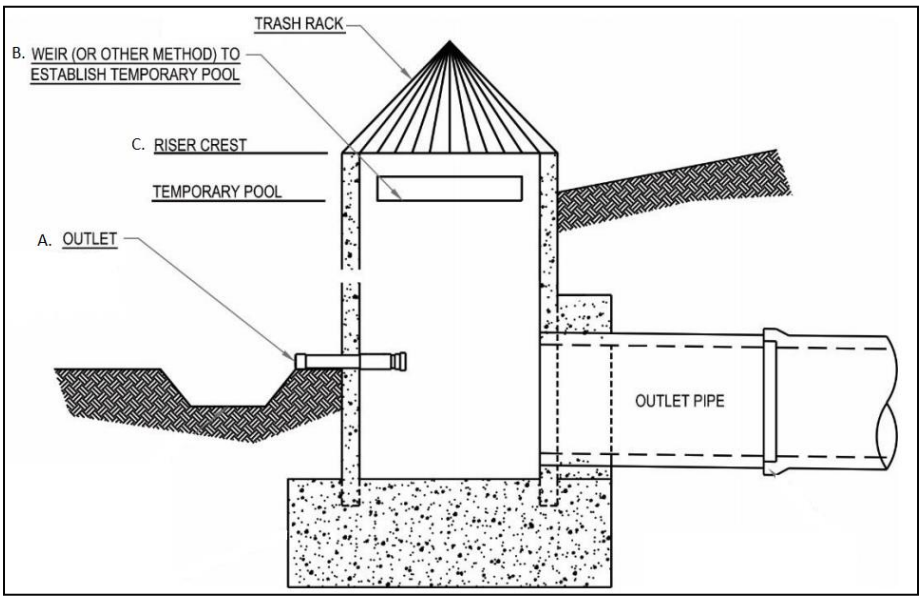

Figure 1-6: Dry pond outlet structure modified from NC Stormwater Design Manual (NCDEQ, 2017c).

Dry ponds were initially one of the most common SCMs implemented and remain ubiquitous throughout the US (Stanley, 1996). Because of ease and simplicity of design, they are among the most used (Ballestero et al., 2012). As of 2014, some jurisdictions dry ponds still implemented were primarily for flood control (Carpenter et al., 2014). Flood control dry ponds commonly have a concrete-lined channel from the inlet to the outlet for low flow conditions to minimize erosion, frequently preventing water from interacting with the soil, where some treatment could have occurred (Hogan and Walbridge, 2007).

Extended detention dry ponds detain water for longer periods than flood control ponds, and provide more time for sedimentation (Middleton and Barrett 2008). However, sedimentation is generally ineffective for smaller particles with slower settling velocities (Garde et al., 1990).

The question can be raised about detention time required to achieve sufficient sedimentation. Stanley (1996) found that pollutant removal was not greatly changed between a drawdown time of 6-12 hours and 74 hours. Also, this sedimentation may be in vain, as settled pollutants can be resuspended during subsequent storm events (Carpenter et al. 2014).

Sedimentation is the only PRM active in dry ponds, which limits potential for pollutant reduction. However, some research shows that in periods of extended detention, there are possibilities for nutrient removal, especially of nitrogen (Carpenter et al. 2014). Ponded water creates anaerobic conditions at the soil interface, therefore providing opportunity for nitrification/denitrification (Lee et al., 2009). In many cases, dry ponds even take on wetland-characteristics, like saturated soil and the growth of some wetland vegetation in central areas (Rosenzweig et al., 2011). After adding a sluice gate to the outlet to increase runoff detention times, Carpenter et al. (2014) reported an increase of NH_3-N removal efficiency from 10 to 84%, although a specific reason was not mentioned. In the study, the basin was completely closed off to detain water for up to 102 hours at a time, which had the potential to create anaerobic soil conditions. More research needs to be done in this area to understand better the processes that are occurring.

1.8: Dry Pond Hydrologic Mitigation and Water Quality Treatment

Surprisingly, there is little research on dry pond performance, especially in regards to soluble pollutants. However, extended detention dry ponds appear to offer more pollutant removal than that of flood control dry ponds. Due to the flow-through nature of flood control dry ponds, they have low capabilities for removing even particle pollutants (U.S. Environmental Protection Agency, 1983).

1.8.1: Dry Pond Hydrologic Mitigation

Two of the main metrics for hydrologic change are peak flow mitigation and runoff volume reduction. Peak outflow mitigation provided by SCMs is a factor of storm characteristics, basin size, and components of the outlet structure including: number and size of the orifice(s) and height and length of the overflow weir or emergency spillway. Runoff volume reduction is dictated by the SCM water balance: precipitation depth, inflow, infiltration, and ET.

Only two dry pond studies discuss water quantity control. Ballestero et al. (2012) reported an annual average peak flow reduction of 93% for a dry pond on the campus of University of New Hampshire. Pope and Hess (1988) found that for storms with rainfall depths less than 13 mm, median volume reduction in their studied dry pond was 13%. The median volume reduction for storms with depths greater than 13 mm decreased to 0%.

1.8.2: Dry Pond Water Quality Treatment

Limited data exist for pollutant removal capabilities of dry ponds, particularly for soluble pollutants. Treatment is often expressed in terms of event mean concentration (EMC) or mass removal. Event mean concentration can be defined by the following equation:

$$EMC = \frac{\sum_0^T M}{\sum_0^T Q}$$

where

M: mass of pollutant (mg)

Q: Volume (L)

T: duration of the storm (hrs)

Total influent and effluent loads can be calculated by multiplying EMC by storm volume. Removal efficiency (RE) describes the ability of an SCM to remove pollutants from runoff, either expressed as a percent of EMC or mass removal (*Table 1-3*). RE of EMC is expressed as follows:

$$RE = \frac{EMC_{inflow} - EMC_{outflow}}{EMC_{inflow}} * 100$$

Often, EMC RE can be misleading because it does not take into account volume reduction or irreducible concentrations. Mass load reduction calculations are beneficial because they account for volume changes (Lenhart and Hunt 2011). *Table 1-3* reports dry pond RE from available studies. In several studies, only data for EMC RE were available. When possible, load removal efficiencies were also reported. Pope and Hess (1988) and Schueler and Helfrich (1988) did not report RE of EMC.

Table 1-3: Mean EMC and load removal efficiencies of dry ponds for several pollutants.

Study	Location	TSS %RE Conc./Load	TP %RE Conc./Load	TN %RE Conc./Loads	$NO_{2,3}$-N %RE Conc./Load	TAN %RE Conc./Load
Carpenter et al. (2014)*	Quebec City, CA	46/39	-/-	-/-	-/-	24/16
Schueler and Helfrich (1988)*	Lake Ridge, WA	-/14	-/20	-/10	-/-	-/-
Stanley (1996)	Greenville, NC	72/71	14/14	-/26	4/-	7/-
USEPA (1983)	Stedwick, WA	63/64	11/<15	-/24	13/10	-/-
Rozensweig et al. (2011)	Princeton, NJ	-/-	-/-	3/17	3/17	3/23
Birch et al. (2006)	Sydney, Au	40/-	-5/-	28/-	-46/-	-/-
Pope and Hess (1988)	Topeka, KS	4/-20	-/20	-/-24	-/18	-/63
Hathaway et al. (2007a)	Charlotte, NC	65/-	-13/-	10/-	-11/-	14/-
Hathaway et al. (2007b)	Charlotte, NC	39/-	-15/-	13/-	29/-	31/-

Studies, which each used different methods of data collection and analysis, should not be directly compared.
**flood control dry pond*

Dry ponds remove some particle pollutants, though REs are lower than those of other SCMs. In a study by Guo (1997), an estimated 11.6 kg of lead, 3.3 kg of copper, and 22.8 kg of zinc accumulated in a dry pond over eighteen years, which was ten times less than what was expected. Because metals are often particle-bound, this may be evidence for lower than expected sediment removal rates.

Dry ponds are not credited with high removal rates of soluble pollutants (Stanley, 1996). The few studies that quantify this removal are listed in *Table 1-3.* With one exception, REs of soluble nitrogen do not exceed 31%. This limited information suggests room for improvement for soluble pollutant treatment in dry ponds.

1.9: Dry Pond Maintenance

Improper maintenance of dry ponds can lead to reduced treatment or system failure (Ballestero et al., 2012). Typical annual maintenance tasks include removal of accumulated sediment, control structure repair, and side slope stabilization (USEPA 1999). Accumulated sediment reduces storage capacity, impairing hydrologic mitigation (Guo, 1997). Dry ponds are often neglected, and maintenance occurs once or less per year (Erickson et al., 2010). More than 2/3 of maintenance costs in dry ponds are associated with periodic maintenance, while the other 1/3 is split between proactive and reactive maintenance (Houle et al., 2013). Caltrans (2004), reported that vegetation management and vector control in dry ponds required the greatest time commitment. Erickson et al. (2010) reported a buildup of sediment, litter, and debris as the main maintenance concern. There were also several instances of pipe clogging, invasive vegetation, and bank erosion.

1.10: Dry Pond Retrofits

Although modern stormwater management goals include water quality control, many old dry ponds remain in place. Attempts have been made to change dry pond structures, or retrofit them, to enhance pollutant removal. Many retrofit techniques have been studied. Reduction of the outlet structure orifice increases detention time, which can induce sedimentation, vegetative uptake, and microbial activity (Ballestero et al., 2012; Traver, 2000; Guo et al., 2000; Marcoon and Guo, 2004). Maintaining a permanent pool elevation at the bottom of the basin limits resuspension of deposited particles (Carpenter et al., 2014). The effects of studied dry pond retrofits are reported herein.

Carpenter et al. (2014) retrofitted a suburban dry pond in Quebec City by reducing the outlet orifice's diameter and installing a sliding door to completely close the outlet. Samples were taken of TSS, NH_3-N, and heavy metals before and after outlet modification. Before the retrofit, average load REs were 39% for TSS, 16% for NH_3-N, and 22% for total zinc. After retrofit, runoff from an event was stored in the basin between 36 and 102, after which the gate was opened to completely dewater the pond. TSS RE significantly increased to an average of 90%. RE of NH_3-N and Zn significantly increased to 84% and 42%, respectively. Additionally, RE of manganese significantly decreased from pre- to post-retrofit, changing from -21% to -429% (Carpenter et al. 2014). Both nitrogen reduction and increased manganese export suggest that anaerobic conditions were created. This may be evidence supporting the possibility of denitrification in dry ponds. Dewatering of the pond also resulted in some particle resuspension. It was suggested that leaving a shallow permanent pool could prevent this resuspension. However, only five storm events were sampled post-retrofit.

Middleton and Barrett (2008) converted an Austin sand filter to a “batch-type” extended detention dry pond, which detained water similarly to the methods of Carpenter et al. (2014). Middleton and Barrett (2008) fitted the basin’s outlet with an automated valve that sensed the start and end of storm events. The valve was programmed to open 12 hours after a storm’s completion to allow the pond to dewater over the course of 12 hours. Only post-retrofitted water quality was monitored in this study. TSS RE was significant, at 91%, though improvement over initial condition cannot be quantified. EMC RE of total zinc was 62%. $NO_{2,3}$-N EMC was reduced by 58%, which is much higher than the that of other dry ponds (Stanley 1996). It is possible that anaerobic conditions were achieved, which encouraged nitrification and denitrification. This treatment method captured first flush flow and eliminated short-circuiting.

While some SCM alterations are intentional, others stem from neglect and lack of maintenance. Clogging and standing water can create wetland-like conditions in dry ponds. (Rosenzweig et al., 2011). These conditions may increase PRMs and thus removal in an SCM (Natarajan and Davis 2016).

Rosenzweig et al. (2011) monitored seasonal nitrogen removal in an extended detention dry pond. Clogging and a lack of maintenance, along with a constant baseflow, led to proliferation of a wetland-like environment in the basin. Water quality samples were collected for 21 days during each of the four seasons, testing for $NO_{2,3}$-N NH_3-N dissolved organic nitrogen (DON), and particulate nitrogen (PN), though only $NO_{2,3}$-N was sampled for all four periods. RE EMC of $NO_{2,3}$-N was only significant in the summer. Temperature and dissolved oxygen content may have seasonally altered pollutant removal capabilities in the basin. Concentration export of $NO_{2,3}$-N in the winter exceeded removal in the summer. Though

nitrogen removal was inconsistent, the study suggested soluble nitrogen removal is possible in dry ponds.

Natarajan and Davis (2016) studied a “failed” infiltration basin for pollutant removal. Clogging reduced infiltration rates and caused the basin to “transition” into a wetland, which can be compared to similar processes in a dry pond. Dry ponds and infiltration basins are also sized similarly and have similar drawdown requirements (NCDEQ, 2017c). The basin was reported to have an average mass RE of 82% TN and 82% TP (Natarajan and Davis 2016). Additionally, the infiltration basin removed an average of 86% $NO_{2,3}$-N, 63% NH_3-N, and 76% dissolved phosphorus. This study illustrates the potential of shifting one type of SCM into another and emphasizes the potential that exists for pollutant removal in SCMs converted into wetlands.

Potential exists to convert sections of dry ponds into wetlands (Ballestero et al., 2012). Wetlands employ more pollutant removal mechanisms than do dry ponds and therefore have potential to more greatly reduce pollutants present in stormwater, as shown in the following section.

1.11: Constructed Stormwater Wetlands

A (surface flow) constructed stormwater wetland (CSW) is a system built specifically for hydrologic and water quality control. In CSWs, water is stored in shallow pools that are suitable for wetland plants and animals (*Figures 1-7 and 1-8;* Schueler, 1992). They are designed to incorporate physical, chemical, and biological water quality treatment processes present in natural wetlands (Harrington et al., 2005). CSWs are designed with the intent to mimic the natural wetland environment and provide ecosystem services (Bolund and Hunhammar, 1999; Costanza et al., 1997; Moore and Hunt, 2012).

Like dry ponds, a CSW is composed of an inlet and an orifice designed to detain runoff for a designated period, usually between 2 – 5 days (NCDEQ, 2017c). CSWs have multiple zones that utilize different PRMs to treat runoff. Zone characterization herein is based upon regulations from the NC DEQ Stormwater Design Manual (2017c). Deep pools (Zone 1) are designated areas in a wetland with depths greater than 45 cm intended to reduce water velocity, dissipate energy, and promote sedimentation. The shallow water zone (Zone 2) is the area of the wetland where many biological processes take place, including denitrification. These two zones are maintained in a permanent pool of water. CSWs are generally designed in more impervious soils or include an impervious liner to maintain permanent pool elevation. Per NCDEQ standards, the temporary inundation zone (TIZ) remains dry during inter-event periods and can store up to 25 mm of runoff for water quality treatment. Maximum ponding depth in this zone is 38 mm. The peak attenuation zone provides storage space to allow for peak flow mitigation during storm events greater than 25 mm (*Table 1-4*). Appropriate vegetation is planted in these zones to provide pollutant removal and stabilize the soil in the CSW.

Table 1-4: A constructed stormwater wetland separated into zones by water depth, where each zone has unique functions (NCDEQ, 2017c).

Zone	**Depth**	**Wetland Area**	**Function**
	cm	*%*	
Forebay	60-100	10-15	Dissipate energy Reduce velocity Settle particles
Deep Pool	45+	5-15	Same as forebay
Shallow Water	0-25	35-45	Gross filtration Filtration Microbial processes
Temporary Inundation	0-40	30-45	Store up to 38 cm storms Water quality treatment volume Dessication UV radiation
Peak Attenuation	40+	-	Peak flow mitigation for larger storms

Figure 1-7: A constructed stormwater wetland implemented at an elementary school.

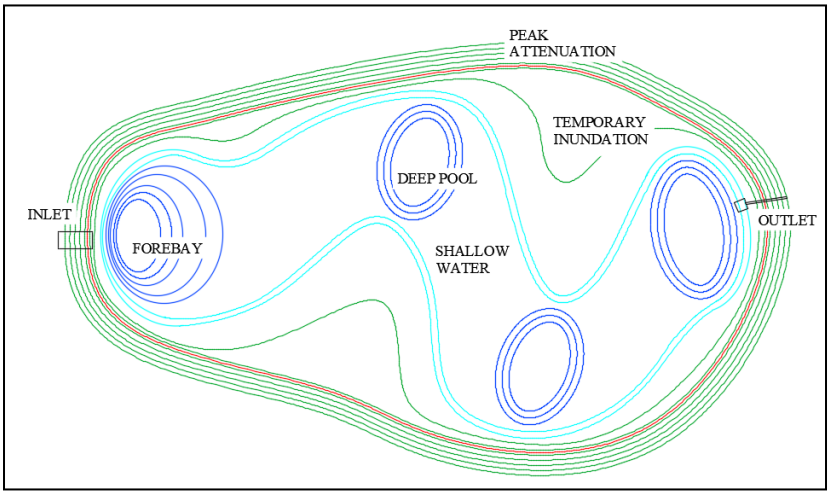

Figure 1-8: Typical layout of a stormwater wetland, including different zones for treatment.

1.12: Constructed Stormwater Wetland Hydrologic and Water Quality Treatment

CSWs, like most SCMs, are designed to provide both hydrologic and water quality treatment of runoff. This section analyzes their capabilities in both categories.

1.12.1: CSW Hydrologic Mitigation

Hydrologic treatment of four CSWs in North Carolina are compared in *Table 1-5*. The wetland in Riverbend, NC was studied at two separate intervals: (1) after initial construction in 2007-2008 (Lenhart and Hunt, 2011) and (2) after five years of maturation (Merriman and Hunt, 2014). Initial reported runoff volume reduction of the wetland was much higher (54%) than after

years of maturation (0%). It was reported that initial volume reduction was due to infiltration and ET, though ET was estimated to contribute to only 5% of the volume lost (Lenhart and Hunt, 2011). Merriman and Hunt (2014) suggested that after a period of maturation, infiltration was essentially eliminated due to the loss of storage volume from sediment accumulation and a change in water table dynamics from shorter antecedent dry periods (Bullock and Acreman 2003; Hensel and Miller 1991).

At the wetland in the Piedmont Region (Line et al., 2008), whose surface area was 2.2% of that of the drainage area, all outflow was produced from storms greater than 33 mm. Volume reduction 100% in 6 out of 11 storms. This reduction was dependent on rainfall characteristics and antecedent rainfall. Less volume reduction associated with the wetland in the Mountain Region of NC, largely due to minimal storage capacity. The wetland often drained during intra-event periods due to riser board leakages, seepage, and ET, increasing storm flow storage capacity. Peak flow reductions were similar for both CSWs.

Merriman et al. (2016) studied a CSW in the first two years after installation. Monthly ET rates ranged from 46 mm to 171 mm over a 14-month period. Volume reduction was largely attributed to ET due to large CSW surface area, dense vegetation, and long detention times. Underlying geology, most likely allowing for infiltration, also contributed to volume reduction. *Table 1-5* summarizes peak flow mitigation and volume reduction of these studies.

Table 1-5: Hydrologic treatment of constructed stormwater wetlands from the literature.

Author	**Location**	**Number of Storms**	**Peak Flow Mitigation[a]** *%*	**Volume Reduction[a]** *%*
Line et al (2008)	Peidmont, NC	11	72(99)	27(93)
Line et al (2008)	Mountains, NC	12	77(96)	9(35)
Lenhart and Hunt (2011)	River Bend, NC[b]	20	80	54
Merriman and Hunt (2014)	River Bend, NC[c]	41	64	0
Merriman et al. (2016)	Coastal NC	44	-	63

[a]*mean(median)*
[b]*newly constructed wetland*
[c]*wetland after 5 years of maturation*

1.12.2: CSW Water Quality Treatment

CSWs employ most PRMs detailed in Section 2.5, resulting in effective pollutant removal. Many authors have evaluated CSW water quality treatment (*Table 1-6*). Concentrations increased from inlet to outlet in River Bend, NC due to large volume reductions (Lenhart and Hunt, 2011). This volume reduction led to high total pollutant load removals. Large pollutant load reductions due to volume reductions were also seen at the wetland in the Piedmont Region (Line et al, 2008). In these studies, volume reduction is the main mechanism for pollutant removal. In cases where volume reduction was less pronounced, there were still substantial load reductions for most pollutants, indicating the effective treatment.

Table 1-6: Water quality treatment of constructed stormwater wetlands from the literature.

Author	Location	TSS RE Conc./Load %	TP RE Conc./Load %	TN RE Conc./Load %	$NO_{2,3}$-N RE Conc./Load %	TAN RE Conc./Load %
Hathaway et al. (2010)	Mooresville, NC	84/-	62/-	52/-	67/-	85/-
Johnson (2006)	Charlotte, NC	66/49	55/33	40/14*	32/10*	62/49
Johnson (2006)	Smithfield, NC	-/-	73/59	80/74	89/88	58/53
Line et al (2008)	Piedmont, NC	64/72	43/59	21/47	23/51	39/54
Line et al (2008)	Mountains, NC	83/88	52/70*	42/59	63/71*	57/69
Lenhart and Hunt (2011)	River Bend, NC	-30/49.2	0/47.2	-51/35.7	9/40.7	-53/41.6

**not statistically significant to $\alpha = 0.05$*

In a study by Kohler et al. (Kohler et al., 2004), in over half the storms sampled, the CSW reduced sediment loads by more than 50%. These rates were even higher for baseflow from a golf course treated in the CSW during dry periods.

Many studies identified CSW design features that contribute to pollutant removal. Like in dry ponds, sedimentation is closely tied to how water moves in and through the CSW, and especially to detention time (Strecker et al., 1992). Terzakis et al. (2008) tested wetland performance on four CSWs with both 12 and 24-hour detention times to measure effects of detention time. Treatment was similar in all CSWs, with reported REs of 49% for TN, 58% for $NO_{2,3}$-N, and 60% for TP (Terzakis et al., 2008). Strecker et al. (1992) analyzed 26 different CSWs for treatment capacity based on factors including vegetation type, watershed land use, wetland area/watershed area ratio (AR), runoff volume, and inlet type. Few meaningful direct relationships were found, partially due insufficient data that would be required to test interaction effects. Wadzuk et al. (2010) both baseflow and stormflow of a CSW and found that sediment

was removed at all flow conditions. The data show that treatment in CSWs is based on many interconnected design factors, where few stand apart as the most influential.

1.13: Hydraulic Residence Time

Hydraulic residence time (HRT) a main factor in pollutant reduction in stormwater wetlands. The largest contributor to HRT is storage capacity, the space available to treat a volume of water (Walker, 1998). However, storage capacity does not account for length to width (L/R) ratio or CSW surface area. Small L/R can result in short-circuiting or the lack of plug-flow conditions, which is essentially the assumption that water moves uniformly through a wetland. Without plug flow, this removal rate becomes invalid. Hydrologic factors, including precipitation, evapotranspiration, and infiltration, all of which affect flow through the system, impact wetland HRT (Watson and Hobson, 1989).

Tomlinson et al. (1993) as reviewed by Walker (1998), graphed the rate of decay of phosphorus and other pollutants in CSWs. Reduction greatly increased with an increase in detention time. However, rate of phosphorus removal substantially decreased after two days.

HRT can be increased for smaller storms (up to 2-year events) by increasing bottom topography in wetlands (Conn and Fiedler 2006). Hydraulic efficiency is the ratio of time to peak outflow pollution concentration to total detention period Hydraulic efficiency is increased as the length to width ratio of a wetland is increased (Perrson et al., 1999). Obstructions such as baffles or underwater berms (i.e. topography) can increase the wetland hydraulic efficiency (Persson et al., 1999). A modeled wetland created by Conn and Fielder (2006), showed up to a 113% increase in HRT through the addition of bottom topology. The implementation of more complex structures within a wetland (like a longer flow path or a mix between shallow water and deep

pools) can increase the wetland surface area, and hence pollutant removal, when space is a limiting factor (Schueler, 1992).

In a study by Gain (1996), the flowpath of a wet pond – wetland system was modified to double the distance between the inlet and outlet. There were no significant water quality changes in water leaving the pond and entering the wetland, but effluent concentrations of the CSW were reduced by 23 mg/L, 0.55 mg/L, and 0.08 mg/L for TSS, TN, and TP, respectively as compared to the inlet. However, there was also an increase in export of heavy metals and organic nitrogen after modification, which was attributed to resuspension of particles settled in the original CSW after flow was rerouted.

As storm events create variable inflow into stormwater wetlands, it can be challenging to design for a particular flow rate. Due to the stochastic nature of storms, HRT can be greatly decreased if there is not sufficient storage capacity from short antecedent dry periods (Wong and Somes 1995). NCDEQ (2017c) requires drawdown times between 2 – 5 days to account for the frequency of storm events in the state.

Storage capacity is one of the main parameters of wetland systems that affect HRT. However, studies have also addressed certain issues like short-circuiting or lack of proper mixing of stored water that can negatively impact this reduction. Overall, topography and routing that encourage full use of the wetland and mixing make a wetland more effective.

1.14: Effects of Wetland Sizing (Surface Area Ratio)

In addition to HRT, many articles discuss the importance of the ratio of CSW surface area to watershed surface area in pollutant removal. This equation can be defined as follows:

$$AR = \frac{A_{CSW}}{A_{WS}}$$

where

AR: Surface area ratio

A_{CSW}: Surface area of CSW

A_{WS}: Watershed surface area

AR is important because a larger CSW surface area increases the contact time of water with the soil interface, which is key in pollutant removal (Schueler, 1992). Maximum ponding depth in CSWs requires a larger footprint to store the design volume (Weiss et al., 2007). A high AR maximizes soil contact time and treatment through sedimentation, filtration, absorption, algal uptake, and microbial activity (Schueler, 1992).

Several studies have analyzed the effect of AR on water quality treatment. For adequate removal of pollutants, Schueler (1992) suggested a minimum AR of 0.02. By creating a model based on zero order kinetics for nutrient uptake rates, Tilley and Brown (1998) found that for every 1% increase in urban area of a watershed, an increase in CSW area of 0.1% was needed to treat runoff. Strecker et al. (1992) found that pollutant removal was higher in CSWs with larger ARs than those with smaller ARs.

Carleton et al. (2001) compared pollutant removal efficiency for 49 CSWs. Wetlands whose treatment are reported in *Table 1-7* were a) constructed, b) in urban watersheds, and c) had removal efficiencies based on load reductions. Two wetlands from Line et al. (2008) have been added to the table. Carleton et al. (2001) did not report a direct correlation between AR and removal efficiency but suggested that interaction effects limited direct comparisons.

Table 1-7: Pollutant load removal efficiency of urban stormwater wetlands by wetland area/watershed area ratio (AR).

Author	Location	Ratio AR	Long term load removal efficiency (%) TSS	TP	TN	$NO_{2,3}$-N	TAN
OWML (1990)[a]	Manassas, VA	0.0078	61.5	14.9	-	59.8	-0.5
Mejorin (1989)[a,b]	Fremont, CA	0.0183	64	48	-	15	10
Line et al. (2008)	Piedmont, NC	0.022	72	59	47	51	54
Carleton et al. (2000)[a]	Manassas, VA	0.0241	57.9	45.9	21.7	39.4	54.7
McCann and Olson (1994)[a]	Orlando, FL	0.0246	68.3	61.5	-11	-13.2	10.2
Athanas and Stevenson (1991)[a]	Centerville, MD	0.0375	65.1	39.4	22.8	54.9	55.8
Line et al. (2008)	Mountains, NC	0.0467	88	70[d]	59	71[d]	69
Rushton and Dye (1993)[a]	Tampa, FL	0.0508	55	65	-	65	39
Wotzka and Oberts (1988), Oberts and Osgood (1991) [a,c]	Roseville, MN	0.0658	83	41	35	35	-

[a]*taken from Carelton et al, 2001*
[b]*partially agricultural watershed*
[c]*treatment cycle spanned multiple storms*
[d]*not statistically significant*

Carleton et al. (2001) found that removal of TP in CSWs depended more on detention time than AR. No strong statistical linear relationships between AR and pollutant removal existed (Strecker et al., 1992). REs reported by Line et al. (2008) were much higher than those reported by others in *Table 1-7*, largely due to load reduction associated with volume reduction. In some CSWs, outflow exceeded inflow, due to groundwater inflow or runon from sources other than the inlet (Carleton et al., 2000; McCann and Olson, 1994). In an urban constructed stormwater wetland with a AR of 0.001, pollutant concentration removal efficiency was -4%, 12%, and 16% for TSS, TP, and TN, respectively (Birch et al., 2004), indicating that a very small AR has a negative effect on pollutant removal.

Bass (2000) monitored an in-stream stormwater wetland that was constructed in a North Carolina coastal plain, with a wetland to watershed area ratio of 0.004. Samples taken included TKN, NH_3-N, $NO_{2,3}$-N, TP, and OP. This area was composed of 2/3 agricultural and forested

land and 1/3 commercial and urban development. Inlet and outlet concentrations were not significantly different at baseflow. In the first year, Bass reported concentration reductions from pre-construction to post-construction of 60%, 30%, 9.5%, and 20% for $NO_{2,3}$-N, NH_3-N, TKN, and TN, respectively, during storm events. These removal rates remained constant or improved in subsequent years. However, both TP and O-PO_4^{3-} were exported at rates of 55% in the first year, which decreased in following years. This initial spike in phosphorus was attributed to phosphorus content in the soil that was stirred up during construction. Results of this study suggest that CSW pollutant removal can be effective in watersheds with a small AR.

Another study examined a CSW that was retrofitted from an impaired stormwater conveyance channel on the campus of North Carolina State University (Tucker 2007). Area constraints only allowed for storage and treatment of the first 5.2 mm of runoff, or 20% of the water quality volume. Tucker found that although runoff exceeded maximum storage capacity in a majority of events, there was evidence of significant treatment TSS, O-PO_4^{3-}, NH_3-N, $NO_{2,3}$-N, TP, and TKN. The outlet was modified halfway through the study to promote extended detention, which dramatically improved treatment. Mean pollutant load removal increased from 26% to 71% for TSS and from 22% to 60% for $NO_{2,3}$-N. This reiterates that HRT is a major factor influencing water quality treatment.

Hathaway and Hunt (2009) retrofitted an area with a wetland-in-series design in a 13.88 ha watershed that was 60% impervious. These CSWs were 0.19 ha, 0.073 ha, and 0.004 ha with ARs of 0.014, 0.0052, 0.00029, respectively. Water quality samples were collected at the inlet of the wetland system and at each of the three basin outlets. The first wetland effectively reduced pollutant concentrations from the inflow, even though it was considered undersized. Significant reductions were not found between inlet and outlet of the two subsequent CSWs. Treatment

provided by undersized wetlands may be sufficient, and additional CSW surface area provided by additional cells may not enhance pollutant removal.

Many studies have shown that AR can impact water quality treatment in CSW, though no direct correlation between the two has been found. While recommendations suggest that appropriate AR should range between 0.05 – 0.1 (Schueler, 1992), many studies discussed in this section found that CSWs with smaller ARs effectively removed pollutants.

1.15: Wetland Ecosystem Services

Ecosystem services are defined as benefits, both ecological and economic, that humans obtain from ecosystems (Costanza et al., 1997). Air and water pollution in urban areas impacts quality of life and human health. Ecosystems incorporated into urban environments provide services that directly and indirectly improve quality of life (Bolund and Hunhammer, 1999). Globally, wetlands contribute a disproportionately large amount of ecosystem services for the land area they occupy (Zedler and Kercher 2005). CSWs, which are constructed to mimic natural wetlands, can sequester carbon, support vegetative diversity, provide habitat, and serve as recreational and cultural amenities in urban centers (Moore and Hunt, 2012). These services provided by CSWs have been observed as early as the point of installation (Merriman et al., 2016).

1.16: Wetland Maintenance

SCM age and frequency of maintenance influence hydrologic and water quality treatment (Blecken et al., 2017). Maintenance issues including clogging, loss of storage space due to sedimentation, and improper management of vegetation reduce SCM treatment ability (Hunt et al. 2011). However, CSWs are slightly more resilient to aging and a lack of maintenance than other SCMs (Merriman and Hunt, 2014). Proper maintenance has been found to be key

component of CSW function. Clogged outlet structures (Hunt et al., 2011) and a buildup of sediment (Merriman and Hunt, 2014) can cause CSWs to fail over time.

Water quality treatment of a 19-year old wet pond/constructed wetland system that received minimal maintenance was similar to treatment of system when first constructed (Al-Rubaei et al., 2016). Annual load removal TN, TP, and TSS significantly increased in this system by the end of the 19-year period. Wetland maturation can enhance pollutant removal efficiency (Merriman and Hunt, 2014).

Wadzuk et al. (2010) monitored water quality in a CSW for two two-year periods four years apart. There was reportedly no maintenance of the wetland before or during the first monitoring period, but between monitoring phases, a control plan of harvest was implemented to eliminate the abundant *Phragmites australis*. The authors found no significant difference in the water quality treatment of the CSW from these two time frames, indicating that wetlands can function effectively long-term, even with minimal maintenance.

Harvesting plants in CSWs has been explored as a method to permanently remove nutrients. Lenhart et al. (2012) found that harvesting plants in a wetland increased N removal. Decomposing plant material in CSWs releases nutrients, which can be minimized through harvesting (Chimney and Pietro 2006). Invasive vegetation and sediment build up can be major issues in constructed wetlands, and there have been significant reported cases of pipe clogging, sediment and debris, and bank erosion (Erickson et al. 2010).

Wetlands, natural and constructed, are viewed by the public as ideal mosquito habitat. Hunt et al. (2006) monitored several SCMs including 18 CSWs for prevalence of mosquito larvae. They found that the larvae, only present at one-third of facilities, was associated with presence of cattails, willows, or absence of mosquito fish. Santana et al. (1994) found that

mosquitoes, which were found at 89% of monitored sites, were productive at retention structures where a mixture of vegetation, including woody vegetation and cattails, were present. Greenway et al. (2003) found that the quantity of mosquito larvae found in SCMs decreased as the biodiversity of macroinvertebrates and macrophytes increased. Designing CSWs with appropriate vegetation and biodiversity can minimize mosquito impacts.

1.17: Dry Pond/CSW Maintenance and Cost Comparison

Cost and commitment time are considered when installing an SCM. Improper maintenance can lead to diminished effectiveness, both for hydrologic and water quality control. This section compares maintenance needs and costs of dry ponds and CSWs.

Time required for maintenance is comparable for dry ponds and CSWs (*Table 1-8*). However, solving maintenance problems is frequently more complicated in wetlands than in dry ponds, often requiring consultation from a professional (Erickson et al., 2010).

Table 1-8: Annual commitment time of maintenance and inspection activities for selected SCMs.

Study	Dry Pond	CSW
	hrs	*hrs*
CALTRANS (2004)	72	-
Houle et al. (2013)	48-71	40-68**
Erickson et al. (2010)*	1	1.5

**median hours spent per visit with one or more visit per year*
***subsurface gravel wetland*

Weiss et al. (2007) found that construction costs were lower for CSWs than for dry ponds, especially when sized for small watersheds. However, the large surface area required by CSWs to meet design standards can be costly in places where land acquisition costs are expensive (Weiss et al. 2007; *Table 1-9*).

Table 1-9: Initial construction costs of SCMs per Water Quality Volume (WQV).

Study	Units	Dry Pond	CSW
Houle et al. (2013)[a]	*$*	40,700	67,800
Weiss et al. (2007)	*$/m^3*	10-150	9-80
USEPA (1999)	*$/m^3*	18-35	21-44[b]

[a]*normalized for ha of impervious cover treated*
[b]*wetland cost was calculated using the assumption that they were 25% greater than dry ponds due plant selection and sediment forebay requirements*

Maintenance costs are an important consideration when comparing SCMs. SCMs with complex design features often require more maintenance and thus greater investment. Many studies have shown that CSWs cost more to maintain as a percentage of construction cost (*Table 1-10*).

Table 1-10: Operational cost of SCMs as a percentage of construction cost.

Study	Dry Pond	CSW
USEPA (1999)	<1%	2%
Weiss et al. (2007)	1.8-2.7%	4.0-14.2%
Houle et al. (2013)	15%	8%
Erickson et al. (2010)[a]	$100-$1500	$200-$3200

[a]*Range of annual cost of sediment removal*

Weiss et al. (2007) compared total cost of SCMs, which included construction costs and operation and maintenance costs. CSWs were found to be the least expensive of all SCMs. Additionally, when comparing total costs, as WQV increased, the rate of increase of cost was more rapid for dry ponds than for CSWs (*Figure 1-9*).

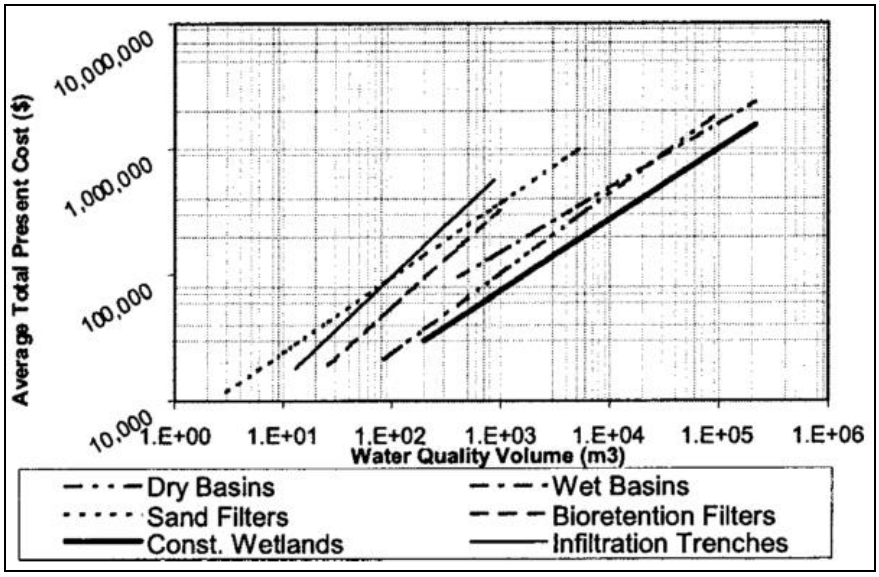

Figure 1-9: Average 2004 cost of SCMs per WQV as compiled by Wiess et al (2007).

Cost per treatment potential is another way to compare SCMs. Houle et al. (2013) found that, while capital cost of TSS removal was higher for CSWs than dry ponds, in all other categories, price per unit of treatment were much lower in CSWs than dry ponds (*Table 1-11*).

Table 1-11: SCM cost per kilogram of treatment potential (taken from Houle et al., 2013).

Pollutant	**Capital Cost**		**Operational cost ($/yr)**	
	Dry Pond	Wetland	Dry Pond	Wetland
TSS ($/kg)	75	102	11	8
TP ($/g)	NT	40	NT	3
DIN* ($/g)	6	3	0.93	0.28

**Dissolved inorganic nitrogen*
NT: no treatment

1.18: Viability of a Wetland Conversion

Selecting SCMs for pollutant removal is an essential part of stormwater management. Dry ponds fail to effectively treat water quality, with especially poor treatment of soluble pollutants. Several retrofits have been installed on various dry ponds with some success, though the widespread implementation of such retrofits has not been discussed in the literature.

Often, issues such as clogging, invasive vegetation, or a wet-bottom basin cause an increase in maintenance concerns in dry ponds. However, as Natarajan and Davis (2016)

reported, a practice that appears to be failing by not functioning as intended may have unexpected benefits. In dry ponds, features that are characteristic of a CSW can be added to introduce more pollutant removal mechanisms, which could increase load removal.

Outlet modification and wetland vegetation installations are simple changes that can be used to "convert" dry ponds to CSWs; however, several limitations may prevent pollutant treatment. Dry ponds lack a flow path to increase HRT, though HRT can also be increased by reducing the diameter of the outlet orifice. Dry pond size cannot be modified to accommodate for wetland design criteria. This under-sizing may reduce volume of runoff treated (Hunt and Doll, 2000). However, as several studies have suggested, even when undersized, wetlands can provide pollutant removal to runoff.

Another temporary issue is the establishment of wetlands in the old dry ponds. Water quality improvements may be gradual at first, as the shift to a wetland requires time and the establishment of such indicators as vegetation and hydric soils (Natarajan and Davis 2016). As a CSW matures, its pollutant removal capabilities are likely to increase (Merriman and Hunt, 2014). Merriman et al. (2016) found that fully matured wetlands did not treat nutrient as well as growing wetlands. Hydrologic services may increase if dry ponds are converted, as wetlands have high rates of evapotranspiration (Merriman et al., 2016). Though there are limitations to a successful conversion, evidence suggests a retrofit of this type would improve pollutant removal in dry ponds.

1.19: References

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CHAPTER 2: Water Quality Treatment of Three Dry Ponds in the Piedmont Region of North Carolina

2.1: Abstract

Dry detention basins, or dry ponds, are employed throughout the United States, particularly in regions with poorly infiltrating soils. These ponds have been historically implemented for flood control and peak flow mitigation but have not been shown to efficiently treat water quality. Despite their relatively poor performance record, industry familiarity and low costs of dry pond construction have incentivized designers to implement these practices. Although dry ponds have a reputation of dry ponds for poor treatment, few peer-reviewed journal articles exist that quantify dry pond pollutant removal, especially for soluble pollutants. Because dry ponds are simple, opportunity exists for structural enhancement to improve performance. In this study, three dry ponds located in the Piedmont region of North Carolina were studied to (1) characterize dry pond water quality treatment and (2) create a baseline from which to measure effects of a proposed dry pond retrofit on water quality. Two dry ponds in Morrisville, NC were monitored from January to September 2017, and a dry pond in Winston-Salem, NC was monitored from February 2017 to March 2018. Concentration removal efficiency, cumulative load removal, and effluent water quality were assessed. Removal was varied widely between ponds, where cumulative load "reductions" ranged from -22 to 24% for TSS, -32 to 17% for TP, and -25 to 25% for TN. Both concentration and load removal efficiencies were partially dependent on influent concentrations, where higher inflow concentrations resulted in greater removal. Effluent concentrations failed to meet ambient water quality standards for TSS, TP, and TN in 68%, 100%, and 100% of samples, respectively.

2.2: Introduction

Increased impervious surface areas from urbanization have disrupted the natural hydrologic cycle, shifting the water balance to a higher proportion of runoff leaving urban landscapes. Excessive runoff yields higher stream flows, which can lead to channel degradation and negatively impact stream habitats. Runoff transports increased pollutant loads from urban watersheds directly into water bodies. In recent years, stormwater management has focused on both hydrologic and water quality control to improve watershed health. Stormwater control measures (SCMs) have been implemented throughout urban landscapes to manage runoff flows and treat runoff pollutants. These SCMs are designed to include pollutant removal mechanisms, such as nutrient uptake, filtration, and sedimentation (Greenway, 2004).

Although SCMs are all implemented for stormwater control, each SCM type has unique advantages and reasons for implementation. For example, dry ponds, because of their simple design, are relatively inexpensive and easy to construct (Weiss et al., 2007). A dry pond is a large basin with an outlet structure that detains runoff and completely dewaters between storm events. They can also be installed in locations where standing water is to be avoided, such as in highway right-of-ways and densely populated areas. Maintenance for dry ponds is also often considered simpler and less expensive than maintenance for other SCMs (Erickson et al., 2010; USEPA, 1999; Weiss et al., 2007).

While dry ponds are valued for their ease and simplicity, their ability to provide water quality treatment has been questioned (Birch, Matthai, and Fazeli 2006; Carpenter et al. 2014; Stanley 1996). Dry ponds, originally implemented for flood control, were designed only to mitigate peak flow rates. Upon the advent of water quality control needs in SCMs, the extended detention dry pond was created and was designed for a minimum drawdown time rather than a

maximum peak flow rate (Virginia DEQ, 2013). Detention time required can range from 12 hours to 5 days, depending on the state – 24 to 36 hours (Virginia DEQ, 2013), 2 to 5 days (NCDEQa, 2017), and 12 to 24 hours in Maryland (CWP, 2009). The principal, and nearly exclusive, pollutant removal mechanism (PRM) in dry ponds is sedimentation, the process by which particles are settled out of the water column. Employing only one PRM limits the potential for water quality improvement. Though limited studies have been published, water quality treatment of both flood control and extended detention dry ponds has been found to be minimal (Stanley, 1996; Shammaa et al., 2002; Carpenter et al., 2014).

Though the disadvantages of dry ponds are clear, their advantages have led to their widespread implementation (Erickson et al., 2010; Weiss et al., 2007). However, with the increasing need to manage pollutant loads in watersheds, the suspected large pollutant loads discharged from dry ponds remains problematic. In this study, three dry ponds in North Carolina were studied to assess water quality treatment. The objective of this study was to quantify pollutant removal capability of dry ponds in the Piedmont of North Carolina for the following pollutants: total suspended solids (TSS), total phosphorus (TP), orthophosphate ($O\text{-}PO_4^{3-}$), total nitrogen (TN), total Kjeldahl nitrogen (TKN), ammoniacal nitrogen (TAN), organic nitrogen (ON), and nitrate-nitrite nitrogen ($NO_{2,3}$-N).

2.3: Methodology

2.3.1: Site Description

Three sites were selected to evaluate existing dry pond performance. Two of the dry ponds (MOV1 and MOV2) were located in Providence Place, a residential area in Morrisville, North Carolina *(Figure 2-1*). This neighborhood had coordinates 35°51’10.48” N and 78°50’46.90” W. MOV1’s watershed was approximately 4.1 ha, 35% of which was impervious

surfaces, including roof, sidewalk and driveway, and roadway. MOV2's watershed of 2.68 ha had similar land use percentages (*Table 2-1*). The majority of the soil in this region was considered urban land by the Natural Resource Conservation Service (NRCS, 2018). The majority of existing underlying soils not considered urban land were in hydrologic soil group D, which are highly impermeable soils (Appendix A).

Figure 2-1: Aerial view of Providence Place: MOV1 and MOV2 are located in the bottom center.

Table 2-1: Watershed characteristics of the studied dry ponds.

	MOV1	**MOV2**	**WS**
Latitude (N)	35°51'11.14"	35°51'10.49"	36°06'22.93"
Longitude (W)	78°50'49.95"	78°50'46.45"	80°18'28.58"
Watershed area (ha)	4.15	2.52	0.93
Roof (ha)	0.48	0.16	0
Sidewalk/Driveway (ha)	0.40	0.09	0.16
Roadway (ha)	0.57	0.38	0
Landscaped (ha)	0.48	1.22	0.76
% impervious	35%	28%	18%
Composite Curve Number	86	85	78

MOV1's surface area was 0.29 ha, draining a 4.15-ha watershed (*Figure 2-2A*). Water flowed into the pond through two culvert inlets – a 60-cm inlet on the south end of the pond (MOV1_IN1) and a 75-cm inlet on the northwest edge (MOV1_IN2, *Figure 2-3*). The pond had one outlet riser structure with an 11.5-cm orifice and a 5.8-m emergency broad-crested weir at a height of 75 cm. There was a consistent baseflow from MOV1_IN2 caused by groundwater infiltration and intrusion into the culvert upstream of the basin, which caused portions of the pond to be constantly wet, where wet areas of the dry pond took on wetland characteristics, including the growth of wetland plants *(Figure 2-4).*

Figure 2-2: Three dry ponds were monitored for the study: MOV1 (A), MOV2 (B), and WS (C).

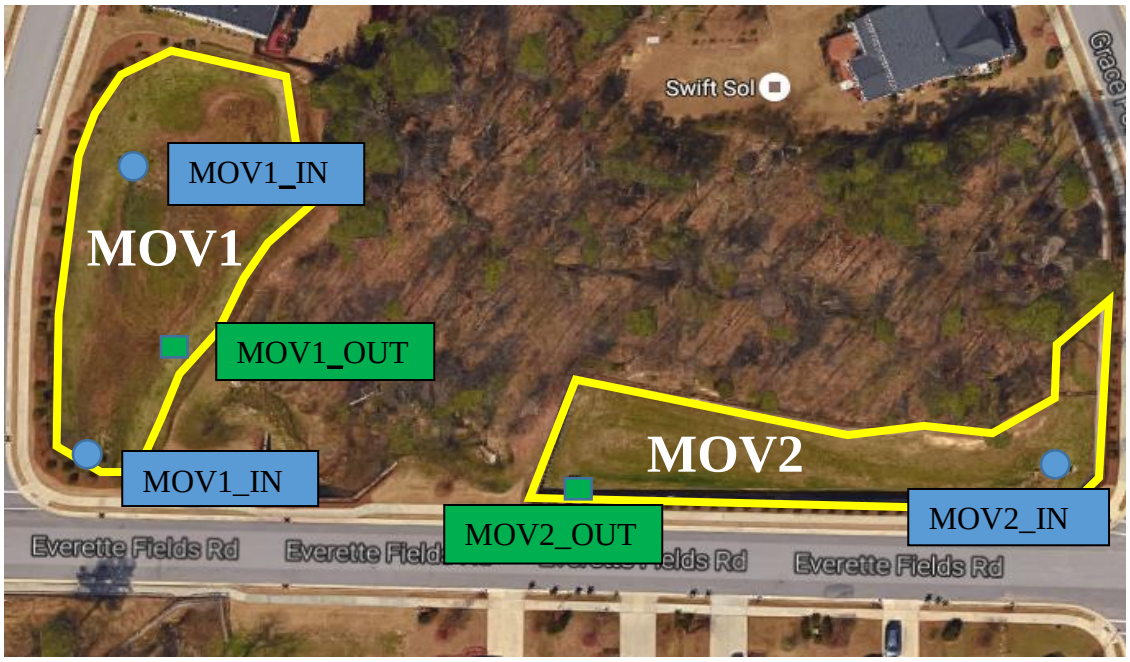

Figure 2-3: MOV1 and MOV2 are located within the same neighborhood block in Providence Place.

Figure 2-4: Wetland plants such as bulrush thrive in the constantly saturated basin of MOV1.

MOV2 (*Figure 2-2B and 2-3*) had a surface area of 0.21 ha with a watershed of 2.68 ha. A 90-cm concrete pipe entered the pond with an outlet riser structure orifice of 8.5 cm. A 4.9-m broad-crested weir at a height of 1.22 m conveyed large storms. The pond's geometry was long and thin, with length to with ratio of 4.5:1, and the Town of Morrisville assumed that water quality treatment in the pond was high by dry pond standards. This pond dried out between

storms, except for a spot of standing water in the inlet pipe from poor grading. No wetland vegetation is present in MOV2.

WS was located on the grounds of Sherwood Forest Elementary School in Winston-Salem, North Carolina (*Figure 2-5*). Latitude and longitude coordinates of the school were 36°06’22.50” N and 80°18’33.80” W. A 0.93-ha portion of the property drained to the 0.04 ha dry pond. The majority of this watershed was a playground area at the school, with 0.17 ha of impervious surfaces (paved areas only) and 0.76 ha of landscaped area (*Table 2-1*). The majority of the WS watershed’s soil were Udorthents in hydrologic soil group C (NRCS, 2018).

Figure 2-5: WS, a kidney-bean shaped dry pond, sits on the eastern edge of Sherwood Forest Elementary.

This dry pond had a 30-cm concrete pipe inlet and an outlet riser structure with a 16-cm orifice as well as two 1.5-m broad-crested weirs for larger storm events (detail in Section 3.2.2). The length to with ratio of the pond was 1:8, with inlet and outlet located on the short end of the pond (*Figure 2-6*), which is generally considered poor design (NCDEQa, 2017).

Figure 2-6: Inlet and outlet are located at the shortest part of WS, with a length to width ratio of 1:8.

2.3.2: Monitoring Design

For all project sites, an upstream/downstream monitoring design (Spooner et al., 1985) was used to evaluate water quality performance. The upstream sites were considered the control and the downstream location the treatment. Sampling stations were installed at each inlet and outlet location, with slightly different setups at each location (*Table 2-2*).

Table 2-2: Monitoring set up of all sampling stations in dry pond study.

Site	Monitoring Equipment	Trigger to Enable	Sample Pacing*	Collection
MOV1_IN1	ISCO 6712™ water quality sampler 2 HOBO™ tipping bucket rain gauges	rainfall exceeded 3.05 mm in 6 hours	rainfall-paced, every 0.76 mm of rain	1 10-L composite bottle
MOV1_IN2	ISCO 6712™ water quality sampler 1 HOBO™ tipping bucket rain gauge	rainfall exceeded 3.05 mm in 6 hours	rainfall-paced, every 0.76 mm of rain	1 10-L composite bottle
MOV1_OUT	ISCO 6712™ water quality sampler ISCO 730™ bubbler flow module	water level exceeded 4.6 cm from orifice invert	flow-paced, every 17.0 m^3 of runoff	24 1-L pi bottles
MOV2_IN	ISCO 6712™ water quality sampler ISCO 750™ area velocity flow module	water level reached 18.3 cm above culvert invert	flow-paced, every 5.66 m^3 of runoff	1 10-L composite bottle
MOV2_OUT	ISCO 6712™ water quality sampler ISCO 730™ bubbler flow module	None	flow-paced, every 5.66 m^3 of runoff	24 1-L pi bottles
WS_IN	ISCO 6712™ water quality sampler 1 HOBO™ tipping bucket rain gauge	rainfall exceeded 10.2 mm in 6 hours	rainfall-paced, every 0.76 mm of rain	24 1-L pi bottles
WS_OUT	ISCO 6712™ water quality sampler ISCO 730™ bubbler flow module 90° v-notch weir	None	flow-paced, every 0.71 m^3 of runoff	24 1-L pi bottles

**pacing was changed on some occasions to capture extreme events*

Monitoring of MOV1 and MOV2 took place between January 2017 and September 2017. Rainfall depth and intensity were measured using a manual rain gauge and a HOBO™ tipping bucket rain gauge located 1.5 m above the ground free from any obstructions to rainfall (*Figure 2-7A)*. One rain gauge was used to measure rainfall for both sites, as they were located within 64 m of each other. The two inlets of MOV1 were also equipped with ISCO 6712™ water quality samplers (*Figure 2-7B,D,E)* with tipping bucket rain gauge attachments for rainfall-paced sampling (*Figure 2-7A,D)*, where rainfall intensity and accumulation dictated when water quality samples were taken. At each of these stations, a composite sample was created based on rainfall intensity that collected a sample every 0.76 mm after an initial minimum rainfall depth of 3.05 mm. It was assumed that inflow rates in MOV1 were proportional to the rainfall intensity of the storm. MOV1_OUT was equipped with an ISCO 6712™ water quality sampler with an ISCO 730™ bubbler flow module (*Figure 2-7C*). Outflow rates were determined using the recorded level and stage-discharge equations (*Section 2.3.4.2*). Flow-proportional composite water quality samples were taken at this location.

At MOV2_IN, an ISCO 6712™ water quality sampler was installed with an ISCO 750™ area velocity flow module (AVM) inside the inlet culvert for inflow measurements (*Figure 2-8A,B*). The ISCO 6712™ at MOV2_OUT was equipped with an ISCO 730™ bubbler flow module (*Figure 2-8C*) and collected samples based on the orifice and broad-crested weir equations (*Section 2.3.4.2*). Both MOV2_IN and _OUT were programmed to take flow-proportional composite water quality samples during storm events. Samplers at all five sampling locations logged data at 2-minute intervals.

Figure 2-7: MOV1 (A)IN1 sampling station, with manual and tipping rain gauge, (B) IN2 inlet pipe, (C) outlet riser with HOBO water level logger, (D) IN2 sampling station with rainfall-paced sampling, and (E) IN2 inlet pipe.

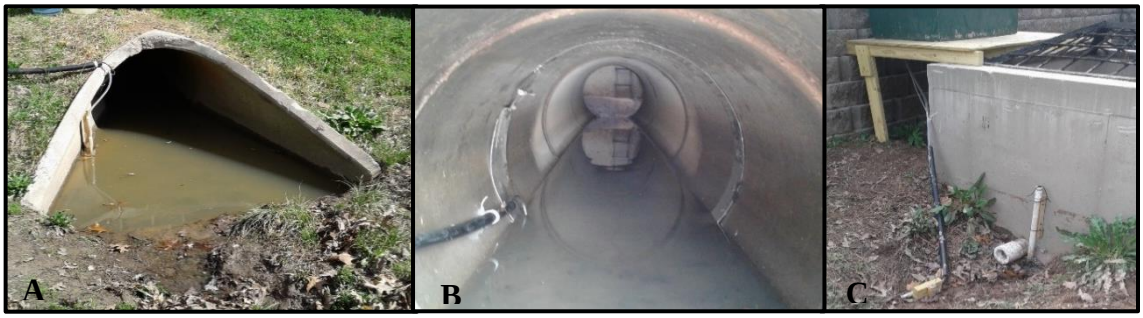

Figure 2-8: MOV2 (A) inlet pipe, (B) inlet area velocity meter for flow-paced sampling, and (C) outlet riser structure with HOBO water level logger.

Flow-proportional, composite samples were collected at each sampling station during storm events using ISCO 6712^{TM} automated samplers to represent the event mean concentration (EMC). At all inlets, samples were deposited into a single composite bottle during the storm. During sample collection, these bottles were agitated by hand to create a homogenous mixture which was then poured into sample bottles. Outlet samples were collected in 24 one-liter bottles that were then shaken and composited in a 24-L bottle. A sample from this composite bottle was taken as the representative sample from the storm. Samples were retrieved within 48 hours of a

storm event and transported on ice to the Center of Applied Aquatic Ecology (CAAE) in Raleigh, NC. Samples from all sites were analyzed for TSS, $NO_{2,3}$-N, TKN, TAN, TP, and O-PO_4^{3-}.

At WS, a manual rain gauge and a HOBO™ tipping bucket rain gauge were installed on site, approximately 1.5 m above the ground, clear from any obstructions (*Figure 2-9B*). The outlet (WS_OUT, *Figure 2-9*C) was equipped with an ISCO 6712™ automated sampler and an ISCO 730™ bubbler module. A contracted weir above a 90° v-notch weir was used to measure flow leaving the pond (*Figure 2-9D*). An ISCO 6712™ connected to a tipping bucket rain gauge was installed at WS_IN (*Figure 2-9*A) to take rainfall proportional samples as a surrogate for flow proportional sampling. Samples were collected with every 0.76 mm of rainfall after an initial rainfall depth of 10.2 mm. Flow proportional (or rainfall proportional) composite samples were collected at both the inlet and outlet of the pond using 24 one-liter bottles at each sampling station. These bottles were shaken and composited into one composite bottle before being transferred to the final sample bottle. These samples were retrieved within 48 hours of a storm event and transported to CAAE to be analyzed for TSS, $NO_{2,3}$-N, TKN, TAN, TP, and O-PO_4^{3-} (*Table 2-3*).

Table 2-3: Reported detection limits and analysis methods for water quality analytes monitored.

Parameter	PQL	Units	Method
TSS	dependent on volume filtered	mg/L	Std Method 2540D
TKN	280	µg/L	EPA Method 351.1
TKN	280	µg/L	EPA Method 351.1
TAN	17.5	µg/L	EPA Method 350-1
$NO_{2,3}$-N	11.2	µg/L	EPA Method 353.2
TP	10	µg/L	EPA Method 365.1
O-PO_4^{3-}	12	µg/L	EPA Method 365.1*

**samples filtered to 0.45 µm*
PQL: Practical quantitation limit, below which the accuracy of the test is no longer valid

Figure 2-9: WS (A) inlet pipe, (B) ISCO tipping rain gauge and sampling box at WS_IN, (C) outlet structure, and (D) installed weir in outlet structure with bubbler tube for level measurements.

It should be noted that ISCO 750™ area velocity flow modules (AVMs) were initially installed at MOV1_IN2 and WS_IN. However, these AVMs did not function correctly in measuring velocity or level and were replaced by tipping bucket rain gauges. Because of backwater present at MOV1_IN2, a weir could not be installed to properly measure flow through the culvert. The slope of the culvert at WS_IN was too steep to install a weir that would properly record flow information. Alternate methods of inflow calculations are discussed in Section 3.2.4.

2.3.3: Monitoring Challenges

The baseflow in MOV1 delivered a consistent load of sediment to the pond's outflow. During inter-event periods, this sediment appeared to build up on the sample intake of MOV1_OUT, which was permanently submerged. A sump was dug beneath the sampler, but it filled in periodically and had to be cleaned out. However, the sediment on the outflow sample intake may have affected outflow TSS concentrations (and possibly ON/TKN/TN

concentrations) in several storm events. TSS concentrations for several storm events were considered inaccurate (Appendix D).

2.3.4: Hydrologic Data Analysis

2.3.4.1 Precipitation

Any storm with precipitation depth greater than 2.5 mm and antecedent dry period no less than 6 hours was characterized as a discrete rainfall event (Driscoll et al., 1989). Rainfall depths from tipping bucket rain gauges were adjusted by a scaling factor using Equation 2.1. During intense rainfall, tipping bucket rain gauges may not adequately record total depth of rainfall, and these adjustments account for total depth missed.

$$P_a = \frac{D_a}{D_m} * P_m \qquad \text{(Eq. 2.1)}$$

where

P_a: corrected depth of rainfall per interval (mm)

Pm: measured depth of rainfall per interval by tipping gauge (mm)

Da: manual rain gauge depth measured sampling period (mm)

D_m: tipping rain gauge depth measured over sampling period (mm)

2.3.4.2 Water Balance

Outflow volumes were recorded using an ISCO 730™ bubbler module. Details for weir and orifice equations to calculate flow are detailed in this section. Exact inflow volumes from MOV1 and WS were not recorded, but rather calculated using a water balance, presented herein.

As stated previously, an AVM was installed at MOV2_IN recorded both level and velocity to calculate flow rates. However, with variable storm flows and often high turbidity in influent, validity of some of these data are questionable. MOV2 inflow volumes were recorded to be between -11 and 100% “greater” than outflow volumes. Based on minimal evaporative and

infiltration losses, it is unlikely that these inflow rates were that much higher than outflow rates. Thus, inflow volumes were calculated using the water balance presented for as MOV1.

A water balance was created to assess volume of flow entering and leaving the dry ponds. Possible inputs include precipitation, inflow, and groundwater inflow. Possible outputs include evapotranspiration, infiltration, and runoff. The water balance is as follows:

$$V_{in} = V_{out}, \text{ or} \qquad \text{(Eq. 2.2)}$$

$$P + R_{in} + G_{in} = ET + G_{out} + R_{out} \qquad \text{(Eq. 2.3)}$$

where

R_{in}: inflow

P: precipitation

G_{in}: groundwater inflow

ET: evapotranspiration

R_{out}: outflow

G_{out}: infiltration

Outflow was calculated directly at all sites, and inflow was calculated using Equation 2.3, as described in the following section. Direct inflow from the banks near the basins that did not enter through the inlets were not included in influent water quality measurements. Each factor of the equation will be discussed.

Morrisville (MOV1 and MOV2):

<u>Groundwater inflow</u>: Per visual observations, it was assumed that the groundwater seepage at this site was negligible. Baseflow from the inlet was considered as direct inflow.

Infiltration: As mentioned in Section 3.2.1, the majority of the soils in both MOV1 and MOV2 watersheds are D soils, which characteristically have low infiltration rates. However, infiltration

tests were performed to account for water loss from infiltration. A modified Philip-Dunne (MPD) infiltrometer from Upstream Technologies™ was used to calculate infiltration rates in the ponds. Both infiltration rates of MOV1 and MOV2 were found to be negligible. Details of these tests can be found in *Appendix A*.

Evapotranspiration: In Morrisville, ET rates were calculated by using a reference crop evapotranspiration from the online North Carolina Climate Office server (NCCO, 2018). The selected site, KRDU, was located at Raleigh-Durham International Airport, 5.9 km from the Morrisville ponds. Data were reported as daily ET. For the five longest storms in MOV1 and MOV2, ET as a percentage of total outflow volume was calculated. This was based on stage-storage information. In all cases, expected ET was found to be 0.31% or less of total volume. ET was considered negligible and was not included in the water balance. Details of this analysis are in *Appendix A*.

Precipitation: Volume of precipitation can be calculated by multiplying the depth of rainfall during a storm by the area on which it fell. In this case, the area of the dry pond is the only area of the watershed receiving rainfall that has not first traveled through the culverts as inflow.

Based on this information, the water balance can be reduced to:

$$P + R_{in} = R_{out} \qquad \text{(Eq. 2.4)}$$

where inflow was calculated from measured precipitation and known outflow volumes.

Outflow: Each monitored dry pond has an outlet structure through which effluent water flows. Outlet structures in both MOV1 and MOV2 were designed with an orifice and a broad-crested weir for larger flows. These outlets were all equipped with an ISCO bubbler module, which read the water level near the outlet every two minutes during storm events. This level was then used

to calculate flowrate, and subsequently volume, by utilizing the orifice equation (Eqs. 3-5 and 3-6) and the weir equation (Eq. 2.7).

When the level in the pond did not exceed orifice diameter, the following equation was used (Malcom, 1989):

$$Q = 4.464 C_d D^{2.5} \qquad \text{(Eq. 2.5)}$$

where

Q: discharge (cfs)

C_d: discharge coefficient, 0.6

D: orifice diameter (ft)

When the pond level exceeded the orifice diameter, the orifice equation (Malcom, 1989) was used:

$$Q = C_d A \sqrt{2gh} \qquad \text{(Eq. 2.6)}$$

where

C_d: discharge coefficient, 0.6

A: orifice area (m^2)

g: gravitational acceleration (m/s^2)

h: depth of orifice center from water surface (m)

A broad-crested weir was a component of both riser structures (*Figure 2-7C and 2-8C*). The equation is as follows:

$$Q = C_d L h^{3/2} \qquad \text{(Eq. 2.7)}$$

where

C_d: 1.49 for a broad-crested weir

L: length of weir (m)

h: height above the weir (m)

By subtracting precipitation volume from calculated discharge volume, total inflow volumes can be calculated (Eq. 2.3). In MOV1, inflow was divided between two inlets, with watersheds of unequal sizes. Both subwatersheds were characterized by percent impervious cover. Using the Simple Method (Schueler, 1987), the proportion of the total inflow volume contributed by each subwatershed was calculated.

$$R_V = 0.05 + 0.9*I_A \quad \text{(Eq. 2.8)}$$

R_V: runoff coefficient

I_A: impervious fraction

and

$$Q_v = 10*R_D*R_V*A \quad \text{(Eq. 2.9)}$$

Q_v: inflow volume (m^3)

R_D: rainfall depth (mm)

A: drainage area (ha)

The entire volume (Q) for the watershed of MOV1 was

$$Q_v = Q_{1,1} + Q_{1,2} \quad \text{(Eq. 2.10)}$$

where

$Q_{1,1}$: volume of inflow from MOV1_IN1

$Q_{1,2}$: volume of inflow from MOV1_IN2

The proportion of inflow from MOV1_IN1 was

$$\frac{Q_{1,1}}{Q_{1,1}+Q_{1,2}} \quad \text{(Eq. 2.11)}$$

or

$$\frac{10*R_D*R_{V1,1}*A_{1,1}}{(10*R_D*R_{V1,1}*A_{1,1})+(10*R_D*R_{V1,2}*A_{1,2})} \quad \text{(Eq. 2.12)}$$

Reducing this formula leaves

$$\frac{R_{V1,1}*A_{1,1}}{(R_{V1,1}*A_{1,1})+(R_{V1,2}*A_{1,2})} \quad \text{(Eq. 2.13)}$$

where proportion of inflow is a function of the runoff coefficient and watershed area. These values are reported in *Table 2-4.*

Table 2-4: Characteristics of subwatersheds that contribute inflow through IN1 and IN2 at MOV1 using the Simple Method.

Rational Method for MOV1		
	MOV1_IN1	**MOV1_IN2**
I_A	0.34	0.38
A (ha)	1.27	2.68
R_V	0.35	0.39
R_VA (ha)	0.45	1.04
% of inflow	30%	70%

Winston-Salem:

The WS watershed was only 18% impervious and produced little outflow for storms less than 13 mm. From observations and measured infiltration testing using the Upstream Technologies™ MPD Infiltrometer, infiltration could not be considered negligible (*Appendix A*). Therefore, total inflow volume in WS was calculated using a different methodology than that used for MOV1 and MOV2.

$$Vin = ET + G_{out} + R_{out} \quad \text{(Eq. 2.14)}$$

To estimate total volume loss due to infiltration and ET, data collected from after the monitoring period were utilized. On March 23, 2018, the 16.5 cm orifice at the base of the pond was capped, allowing for up to 150 mm of ponding within the basin. After storm events, it was observed that this pool of water would draw down within a few days. A HOBO water level logger installed at the base of the basin was utilized to determine duration of basin drawdown.

Using these data and a stage-storage relationship developed for WS, average hourly drawdown rate was calculated in cubic meters/hour. This rate was estimated using five storm events (*Table 2-5*). Though the number of events used to estimate drawdown rate was relatively small, standard error was much less than the estimated mean, rendering the mean an acceptable estimation for drawdown rate. Drawdown rate in WS was assumed to include both infiltration and ET. This rate may vary slightly on a seasonal basis.

Table 2-5: Drawdown rate of WS among five monitored events.

Date	**Starting Elevation** *mm*	**Duration** *h:m*	**Drawdown Rate** *mm/hr*
3/25/2018	153	36:46	2.46
4/7/2018	97	20:42	4.45
4/16/2018	123	25:58	3.64
4/25/2018	155	32:24	2.36
4/26/2018	177	49:54	2.32
		Mean	3.05
		Std. Error	0.96

Runoff was calculated using a 90° v-notch weir installed inside the riser structure of WS and an ISCO 760™ bubbler module. The equation for a 90° v-notch weir is (Malcom, 1989):

$$Q = C_d h^{5/2} \qquad \text{(Eq. 2.15)}$$

where

C_d: 1469 (constant)

h: height of water above the weir (m)

2.3.5: Water Quality Data Analysis

Influent and effluent water quality of each dry pond was monitored. There are several methods for assessing how an SCM treats water quality. Reduction in pollutant event mean

concentration (EMC) is the most direct way to compare water quality of influent and effluent of the dry pond. EMC reduction is found by:

$$RE_c = \frac{C_i - C_o}{C_i} * 100 \qquad \text{(Eq. 2.16)}$$

where

C_i: influent concentration (mg/L)

C_o: effluent concentration (mg/L)

RE_c: removal efficiency (%)

However, this equation does not account for total volume of water of influent or effluent, i.e. water associated with rainfall, infiltration, or ET. The only volume change considered herein is due to precipitation falling directly on the dry pond, per the water balance described in Section 3.2.4. The second method for evaluating water quality performance, pollutant load reduction, incorporates volumes to evaluate system performance. These volumes are multiplied by influent and effluent EMCs in the equations:

$$EL_{in_i} = C_{in_i} * V_{in_i} \qquad \text{(Eq. 2.17)}$$

$$EL_{out_i} = C_{out_i} * V_{out_i} \qquad \text{(Eq. 2.18)}$$

where

EL: event load (g)

V: volume (m^3)

In WS, per the method for calculating V_{in}, measured influent concentration was attributed to the entire inflow volume. In MOV1 and MOV2, any difference in inflow and outflow volumes was attributed to precipitation. Assuming that concentrations of pollutants in rainfall were equal to inflow concentrations would likely be an overestimate of total inflow loads. Typically, measured concentrations of TSS, TP, Ortho-P, and ON in rainfall are much lower (Anderson and

Downing, 2006; Wu et al., 1998) than the influent concentrations recorded herein. However, dry and wet deposition of nitrogen contributes to TAN and $NO_{2,3}$-N loads (Line et al., 2002; Wu et al., 1998). At the Morrisville sites, rainfall was a large percentage of incoming water volume, ranging from 8 – 19% of total volume at MOV1 and 11 – 43% at MOV2. Therefore, comparing influent and effluent nitrogen loads in these ponds without considering loads contributed from direct deposition would underestimate pollutant removal performance in the pond.

In both MOV1 and MOV2, a portion of outflow was directly attributed to precipitation. The literature has shown that direct deposition of nitrogen contribute substantially to nitrogen load (Line et al., 2002; Wu et al., 1998). To quantify the portion of nitrogen load that was attributed to direct deposition, precipitation sampling stations were set up at the Morrisville sites after the initial monitoring period. The incorporation of rainwater nitrogen deposition concentrations more accurately estimates load.

Bulk deposition was measured along with storm event water quality beginning on 5/14/2018. Oil pans were installed at ground level at both MOV1 and MOV2 to collect bulk deposition of nitrogen. Due to potential contamination issues from surface pollutants, the oil pans were raised 0.3 m off the ground (5/23/20118) and covered with a nylon wire mesh (6/12/2018). Within 24 hours of the event's completion, samples from each pond were composited into one acidified bottle and transported on ice to CAAE to be analyzed within 48 hours. Oil pans were cleaned in the field with deionized water before the next event. Median concentrations of nitrogen forms associated with deposition were combined over all storm events and used as a representative concentration. Total influent loads were calculated as follows:

$$EL_{in} = C_P AP + C_{in} V_{in} \qquad \text{(Eq. 2.19)}$$

where

EL_{in}: total influent pollutant load (g)

C_p: concentration from direct rainfall (mg/L)

A: area of the dry pond (ha)

P: rainfall depth (mm)

C_{in}: concentration from the dry pond inlet (mg/L)

V_{in}: runoff volume (m^3)

Annual event loading can also be determined by summing total influent and effluent loads over the monitoring periods. These loads are then normalized for total storm flow from hydrologic events throughout the monitoring period and also normalized for 30-year normal rainfall (Eq. 2.20, NOAA, 2018a):

$$Annual\ Event\ Load\ Export = \frac{\sum EL_O * \frac{\sum V_{out,total}}{\sum V_{out,WQ}}}{1000 * DA} * \frac{P_{annual}}{P_{observed}} \qquad \text{(Eq. 2.20)}$$

where

EL_O: Event load out (g)

DA: Drainage area (ha)

$\sum V_{out,total}$: Sum of outflow volumes from storm events for 8-month period (m^3)

$\sum V_{out,WQ}$: Sum of outflow volumes from events monitored for water quality for 8-month period (m^3)

P_{annual}: Normal 12-month precipitation for local area (mm)

$P_{observed}$: Observed rainfall during 8-month monitoring period (mm)

In MOV1, a constant baseflow flowed through the pond year-round. This made it challenging to separate stormflow and baseflow. For consistency, stormwater runoff volume was included in calculations when the level at the outlet orifice exceeded 0.046 m. This elevation was

chosen because it was observed that baseflow never exceeded this height. Outflow samples were collected when water levels exceeded 0.046 m to avoid erroneous sampling.

3.2.6 Statistical Analysis

Paired inflow/outflow water quality data for each pond were tested for normality and log-normality using Shapiro-Wilk, Kolmogorov-Smirnov, and Anderson-Darling normality tests. Tests that were normally or log-normally distributed were analyzed using the Student's t-test. Tests that were not normally or log-normally distributed were analyzed using non-parametric statistics. For non-parametric tests, data sets were first tested for symmetry using the m-out-of-n bootstrap symmetry test (Miao et al.,2006). The Wilcoxon signed rank test was used for symmetrical non-parametric datasets, and non-symmetrical datasets were compared using the paired samples sign test. Analysis was performed using R 3.4.1 statistical software, and $\alpha = 0.05$ was used to test statistical significance.

2.4: Results and Discussion

2.4.1: Morrisville Hydrology

Hydrologic monitoring of MOV1 and MOV2 in their existing condition took place for a 7.5-month period between 1/26/2017 and 9/12/2017. Rainfall during this period was 118% of the 30-year normal precipitation during this period in Raleigh, NC, as reported by the NOAA National Centers for Environmental Information (NOAA, 2018a). Seasonal rainfall was separated based on the solstices and equinoxes, and duration of the sample period is as follows:

- Winter: 1/27/17 – 3/19/17 (60% of total period)
- Spring: 3/20/17 – 6/20/17 (100% of total period)
- Summer: 6/21/17 – 9/13/17 (89% of total period)

- Fall: no data collected

During this monitoring period, 42 discrete events occurred (*Table 2-6*), with a maximum 5-minute peak intensity of 123.7 mm (*Table 2-7*), which is similar to a storm with a one-year return interval of 120 mm/hr (NOAA, 2018b). A 60+ hour storm occurred in April 2017, but it was not adequately captured due to clogging of rain gauges. The second largest storm totaled 47.8 mm in 29 hours, which is smaller than the one-year, 24-hour, which is 71.9 mm (NOAA, 2018b).

Table 2-6: Seasonal rainfall in Morrisville for monitoring period compared to 30-year normal rainfall.

Season	**Rainfall Observed** *mm*	**% of Period**	**Adjusted normal rainfall*** *mm*	**% of Normal Rainfall**
Winter	70.1	60%	149.9	47%
Spring	515.1	100%	261.6	197%
Summer	233.2	89%	283.2	82%
Fall	-	-	-	-
Total	818.4	63%	694.8	118%

**Adjusted normal rainfall is the percentage of the seasonal normal rainfall adjusted to fit the sampling period*

Peak outflows, runoff volumes, and 5-minute peak rainfall intensities were monitored for both MOV1 and MOV2 (*Table 2-7* and *Table* 2-8). Total event outflow from MOV1 reached 13357 cubic meters, which was more than three times than that of MOV2. This difference is likely due to the larger watershed of MOV1 and the contribution of baseflow during storm events. The highest peak outflow from MOV1 was 260.3 L/s in April 2017. Although tipping rain gauges clogged during this event, manual rain gauges collected 178 mm of rainfall during the storm (4/23 – 4/26). This depth of rainfall is nearly equivalent to a 72-hour, 25-year event in Raleigh, NC (NOAA, 2018b). A runoff volume of 4630.4 m^3 was also attributed to this event, a third of the total runoff during the monitoring period. During the second largest storm, outflow

volume was 1032.1 m^3 and peak outflow reached 124.2 L/s, much higher than the median. By including baseflow, outflow from MOV1 occurred in every storm event. Runoff for storms between 9/6/17 and 9/12/17 were not recorded due to equipment failure.

Table 2-7: Rainfall event characteristics and hydrologic mitigation provided by MOV1.

Parameter	**Rainfall Depth** *mm*	**Storm Duration** *hr:min*	**5-min Peak Intensity** *mm/hr*	**Peak Outflow MOV1** *L/s*	**Runoff Volume MOV1** *m^3*
Min	2.54	1:06	3.05	1.8	7.6
Median	9.14	8:34	25.9	13.8	126.3
Mean	13.97	10:18	36.6	22.7	333.9
Max	47.75*	72:02	123.7	260.3	4630.4
Sum	724.39	-	-	-	13357

**Second largest storm. A large amount of rain fell over the course of three days in April 2017, but clogging of the tipping bucket rain gauge prevented characterization of the storm(s) during this time*

In MOV2, peak outflows reached 206.3 L/s and runoff volume was 1873.4 m^3 for the 3-day storm in April (*Table 2-8*). Runoff volume from the second largest storm was 414.6 m^3, with a peak runoff of 11.9 L/s. This peak outflow is much lower than the peak outflow for the same storm in MOV1, because the latter overtopped the riser structure. In MOV2, water stored remained below the top of the outlet structure for the duration of the storm and allowed the basin to better mitigate flows. There were many storms that produced no outflow in MOV2. Outflow from several storms between 7/23/17 and 8/8/17 was not properly recorded due to equipment failure.

Table 2-8: Rainfall event characteristics and hydrologic mitigation provided by MOV2.

Parameter	**Rainfall Depth** *mm*	**Storm Duration** *hr:min*	**5-min Peak Intensity** *mm/hr*	**Peak Outflow MOV2** *L/s*	**Runoff Volume MOV2**** *m^3*
Min	2.54	1:00	3.05	0	0
Median	9.14	5:58	25.91	4.6	10.1
Mean	13.97	8:28	36.58	6.8	96.6
Max	47.75*	70:44	123.70	106.3	1873.4
Sum	724.39	-	-	-	4059.0**

**Second largest storm. A large amount of rain fell over the course of three days in April 2017, but clogging of the tipping bucket rain gauge prevented characterization of the storm(s) during this time*
***Outflow volume for storms between 7/23/17 and 8/8/17 may be underpredicted due to faulty equipment*

2.4.2: Morrisville Water Quality Analysis

Although 42 events were monitored for hydrology, water quality samples were collected for fewer events. Water quality samples were taken for 14 storms at MOV1. However, during the storm event on 5/22/2017, a bubbler was used in lieu of a rain gauge for sampling at MOV1_IN2 in an effort to take capture flow-placed samples. It is believed that flow rates were greatly overestimated during the latter portion of the storm when water was ponded in the basin. It is likely that samples were taken from ponded water after inflow ceased. This issue could potentially dilute the composite water quality samples and render them inaccurate, so the samples from MOV1 for this storm were not included in the analysis.

Water quality samples were collected for 10 storm events at MOV2. However, during the storm event on 7/23/2017, the bubbler module on MOV2_OUT was not functioning and likely did not properly calculate total flow volume, therefore possibly under-collecting samples. This is based on evidence that similarly-sized storms or smaller storms with similar 5-min peak intensities had much larger calculated outflow volumes. This error would create concentration data that were not representative of the entire storm. For this reason, this storm was discarded

from the analysis. Eight of the nine events sampled from MOV2 were paired with samples from the same storm event from MOV1 (*Table 2-9*).

Table 2-9: Total number of events monitored for hydrology and water quality in MOV1 and MOV2.

Location	Total	Winter	Spring	Summer	Fall	Rainfall Median *mm*	Rainfall Sum *mm*
Hydrology	42	4	20	18	-	9.1	568.5
Water Quality							
MOV1/MOV2 (paired)	13/9 (8)	2/1 (1)	5/4 (3)	6/4 (4)	-	21.6 (24.4)	287.2 (224.6)

The dry ponds were analyzed using four metrics (Lenhart and Hunt, 2011):

- Concentration reduction efficiency
- Cumulative load reduction
- Comparison to ambient water quality conditions
- Comparison to dry pond effluent concentration used by NCDEQ (2017b)

Ambient TSS conditions are compared to a target concentration of 25 mg/L suggested by Barrett et al. (2004). Ambient water quality conditions for nutrients were taken from median concentrations in the piedmont ecoregion reported by McNett et al. (2010) in streams with a "good" rating for macroinvertebrates and used previously as threshold values by Koryto et al. (2017) and Brown and Hunt (2011).

Dry pond water quality treatment was compared to NCDEQ regulatory standards (2017b). For influent concentrations between 35 and 100 mg/L, NCDEQ states that SCMs must reduce TSS to no more than 25 mg/L to be considered a primary practice, the best appraisal (2017b). Based on previous studies, dry ponds are not given this classification. An NCDEQ SCM crediting document (2017b) establishes effluent TP and TN concentrations for various

SCM types, per monitoring studies centric to North Carolina. Each pond was analyzed individually and ponds were also directly compared.

2.4.2.1 *MOV1*

Although impervious percentage of the two inlet watersheds of MOV1 was similar, land use distribution was not equal. Concentrations of MOV1_IN2 were generally higher than those of MOV1_IN1. Based on the Simple Method, the IN2 watershed contributed 70% of the flow. Pollutant concentration boxplots at each sampling station are presented in *Figure 2-10.*

As mentioned, sediment buildup on the MOV1_OUT sample intake was an issue in several storms. In response to this error, the two storms (4/3/2017 and 7/23/2017) with the highest outflow TSS concentrations were eliminated, as they were considered outliers using Cook's Distance. Metrics were reanalyzed in the following section. Both ON and TP concentrations are composed in part by particulate matter. To assess sample contamination based on inaccurately high TSS concentrations, a linear regression was performed for TP, ON, TKN, and TN with the log of TSS as a predictor variable and the log of each aforementioned pollutant as response variables. ON and TKN were found to be significantly related to TSS ($\alpha = 0.05$). There was no significant relationship between TSS and TN or TSS and TP. For analysis, TSS, ON, and TKN concentrations were not included for these two storms.

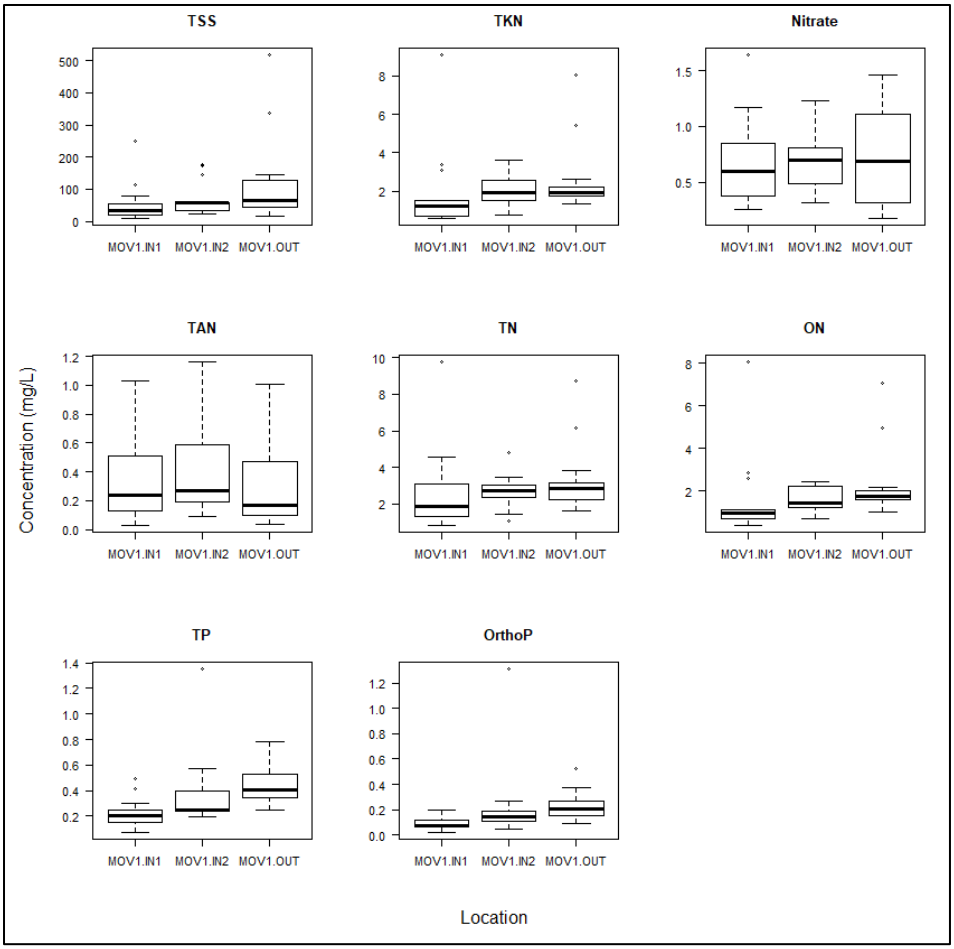

Figure 2-10: EMCs for all locations in MOV1 during the monitoring period.

The combined inlet EMC was compared to outlet EMC to test for a statistical difference (*Table 2-10*). A significant difference was found only for TN, with a median concentration removal efficiency (RE_c) of -24.5%. This indicates that nitrogen was exported from the pond rather than removed. No significant differences were found for any other forms of nitrogen, suggesting that the type of nitrogen being exported was variable.

In MOV1, which remained constantly wet due to baseflow, there was a large buildup of organic sediment near the center of the pond. It is possible that this organic matter was stirred up in the pond during storm events. Additionally, the primary outlet was at the base of the pond, so sediment dislodged by flow from the basin bottom had no opportunity to resettle.

For most parameters, median RE_c was negative, with the exception of TAN, though not significant to $\alpha = 0.05$ (*Table 2-10*). Overall, RE_c suggests that a large percentage of pollutants were exported from MOV1. While dry ponds are not credited with high removal rates (Stanley, 1996), they are not expected to export sediments and nutrients either.

Table 2-10: Median influent and effluent EMCs of MOV1 (mg/L) and concentration removal efficiencies (RE_c).

	Location				**Significance**		
Pollutant	Rain	MOV1_IN1 (n=13)	MOV1_IN2 (n=13)	MOV1_OUT (n=13)	Inflow to Outflow (n=13) p-value**	Test	Median RE_c
TSS*	-	34	42	54	0.592	Paired t-test	-7.6%
TP	-	0.20	0.25	0.41	0.202	Paired t-test	-45%
$O\text{-}PO_4^{3}$-	-	0.08	0.14	0.21	0.168	Wilcoxon Signed Rank	-43%
TN	0.37	1.85	2.75	2.85	**0.0398**	Wilcoxon Signed Rank	**-25%**
TAN	0.16	0.24	0.27	0.17	0.254	Paired t-test	26%
TKN*	0.24	1.19	1.73	1.88	0.0830	Wilcoxon Signed Rank	-23%
ON*	0.035	0.96	1.45	1.68	0.102	Wilcoxon Signed Rank	-42%
$NO_{2,3}$-N	0.13	0.60	0.70	0.69	0.792	Paired t-test	-7.0%

**n = 11*
***bolded values are significant*

In MOV1 and MOV2, nitrogen loads from direct deposition were included in the influent load estimations. Deposition rates were taken over four storm events due to issues with contamination and a lack of storms, and the median concentration was taken as a representative precipitation concentration. Total rainfall volume was considered depth of rainfall over the horizontal area of the basin.

Analyses using cumulative load reduction account for volume differences between the inflow and outflow that are not incorporated when comparing concentrations. Additionally,

cumulative loading accounts for variability among storms. Cumulative inflow and outflow loads were compared for paired statistical differences (*Table 2-11*). Event load reductions (ELR) were only positive for Ortho-P and TAN. Event loading changes were significant for TP (-11%), TN (-26%), and ON (-46%), indicating that there was an annual export of these pollutants. Particles and other pollutants built up in the pond could be resuspended during low flows and discharged from the pond. No deep pool, which would promote stilling and limit particle resuspension, or settling mechanism existed near the outlet to prevent discharge of pollutants.

Table 2-11: Cumulative influent and effluent loads and annual event loading of pollutants in MOV1.

Pollutant	**Location**	**MOV1 Cumulative Load**	**Annual Loading**	**Event Load Reduction**	**Test**	**Significance**
		g	*kg/ha/yr*	*%*		*p-value***
TSS*	Inflow	217000	199	-22%	Student's t-test	0.467
	Outflow	265000	243			
TP	Inflow	1700	1.4	**-11%**	Log Student's t-test	**0.0161**
	Outflow	1880	1.59			
$O\text{-}PO_4^{3-}$	Inflow	1120	0.9	10%	Wilcoxon Signed Rank	0.0942
	Outflow	1010	0.85			
TN	Inflow	10500	8.9	**-25%**	Student's t-test	**0.0152**
	Outflow	13100	11.1			
TAN	Inflow	1680	1.4	27%	Wilcoxon Signed Rank	0.216
	Outflow	1230	1.0			
TKN*	Inflow	6880	6.3	-22%	Student's t-test	0.0857
	Outflow	8400	7.7			
ON*	Inflow	5320.0	4.9	**-37%**	Student's t-test	**0.0378**
	Outflow	7270.0	6.7			
$NO_{2,3}$-N	Inflow	2950	2.5	-11%	Student's t-test	0.258
	Outflow	3270	2.8			

**n = 11*
***bold values are significant to α = 0.05*

TSS loads in MOV1 were initially reduced, but an uptick in sediment export in May and June 2017 greatly increased TSS exports (*Figure 2-11*). Nitrogen export loads of nitrogen consistently exceeded import loads. Import and export loads of TP were relatively similar.

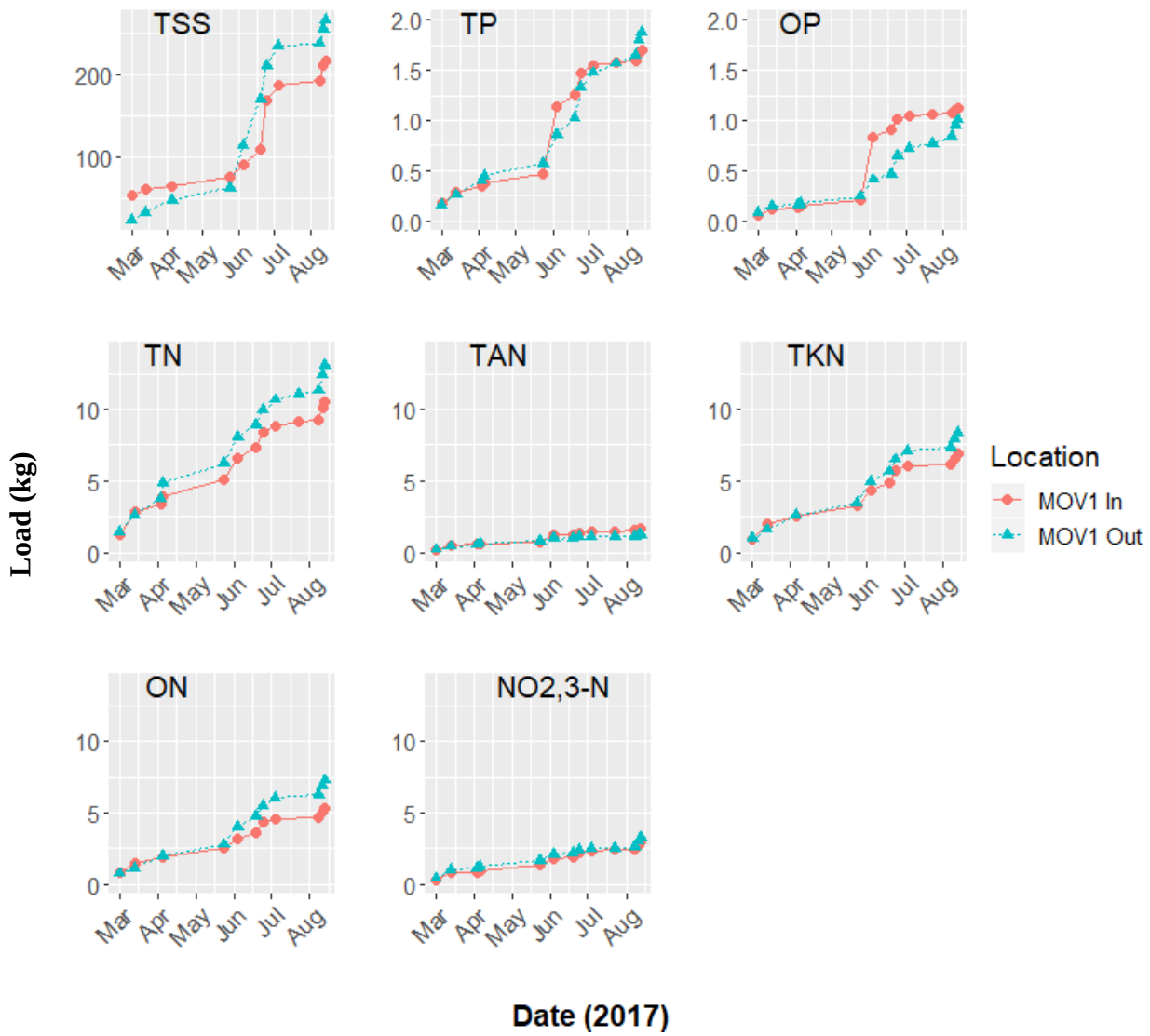

Figure 2-11: Cumulative loads of TSS, TN, and TP in MOV1 for an 8-month monitoring period.

Although water quality performance of an SCM can be measured by RE_c and ELR, it is also important to consider how effluent conditions compare to ambient water quality conditions. Elevated pollutant concentrations discharged into streams can create conditions where concentrations are higher than what is appropriate for aquatic life (Barbour et al., 1999). In this section, effluent concentrations of TSS, TN, and TP are compared to ambient water quality conditions in *Figure 2-12*, *2-13*, and *2-14*, respectively.

Effluent concentration credits from the NC Stormwater Control Measure Credit Removal document for TN and TP were also incorporated in the comparison. These concentrations can be used to directly compare dry pond treatment herein. These values are indicated by the green dashed line in *Figure 2-13* and *2-14*. Nearly all influent and effluent samples were less than 0.66 mg/L [NCDEQ (2017b) TP concentration ascribed to dry ponds]; this result is likely due to relatively "clean" influent water quality. For all parameters, the threshold for ambient conditions was exceeded in the majority of sample events (*Table 2-12*).

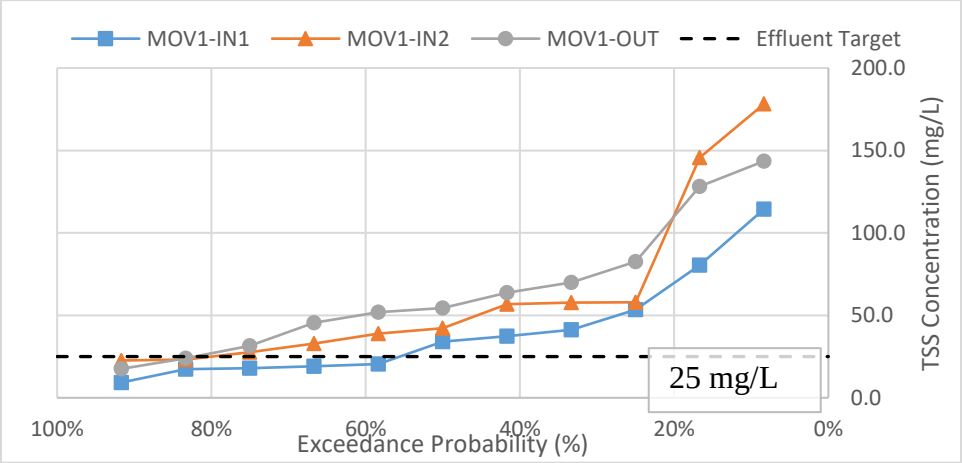

Figure 2-12: TSS exceedance probabilities at MOV1 with respect to stream ambient water quality threshold (Barett et al, 2004).

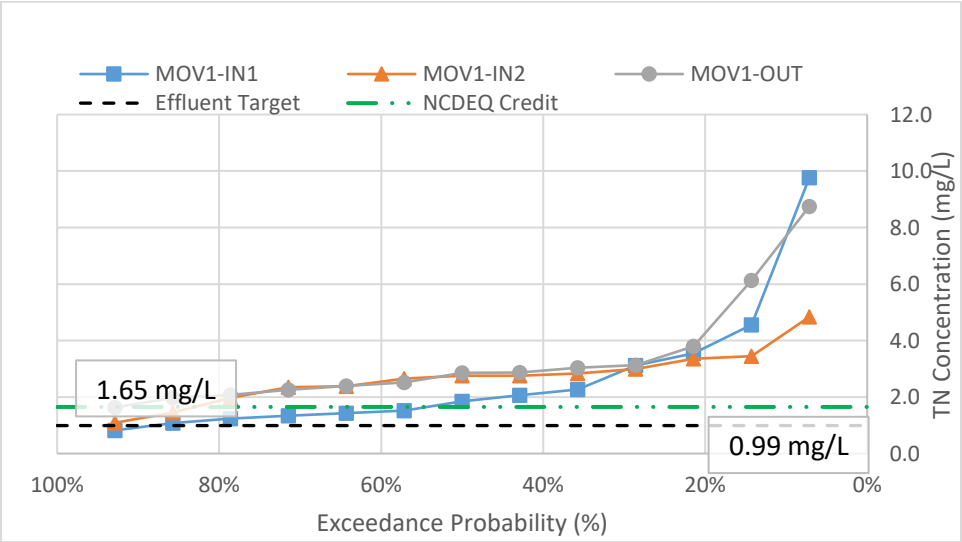

*Figure 2-13:*MOV1 exceedance probabilities for TN with respect to stream ambient water quality thresholds (McNett et al., 2010) and NCDEQ (2017b) ascribed credit.

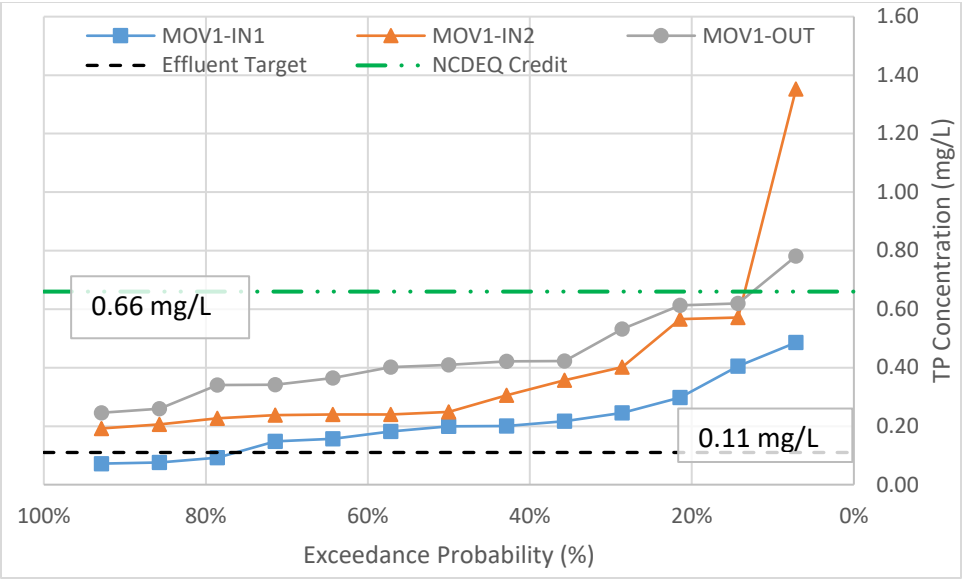

Figure 2-14: MOV1 exceedance probabilities for TP with respect to stream ambient water quality thresholds (McNett et al., 2010) and NCDEQ (2017b) ascribed credit.

Table 2-12: Exceedance probabilities of EMCs at all sampling stations at MOV1 with respect to ambient water quality thresholds proposed by Barrett et al. (2004) for TSS and by McNett et al. (2010) for other analytes.

Parameter	Target *mg/L*	Expected Probability of EMC Exceeding Target		
		IN1	IN2	OUT
TSS	25	50%	79%	86%
TP	0.11	79%	100%	100%
TN	0.99	93%	100%	100%
TAN	0.04	93%	100%	100%
TKN	0.4	100%	100%	100%
$NO_{2,3}$-N	0.59	57%	71%	57%

Based on all four metrics, pollutant removal performance of MOV1 was poor. This SCM showed little ability to reduce pollutants and often exporting them. Because this dry pond did not provide any volume reduction, load exports reflected those of EMCs. Effluent concentrations from this pond exceeded all water quality thresholds associated with sensitive aquatic life.

2.4.2.2 MOV2

Unlike MOV1, MOV2 had no baseflow, resulting in a more typical dry pond hydrologic regime. Concentrations of nitrogen species sampled for MOV2 were highly variable. In the

storm sampled from 3/1/2017, EMCs of all forms of N were found to be factors higher than in all subsequent storm events. It is possible that these high concentrations were due to a loading of pollen (Line et al., 2002). Also, organic matter in the form of dead grass and other material was present in the discharge of MOV2 (*Figure 2-15*). These inordinately high concentrations from the 3/1 event were not observed in MOV1. Box plots of EMCs for all pollutants measured in MOV2 are displayed as *Figure 2-16.*

Figure 2-15: Buildup of organic matter like pollen and grass clippings may have contributed to high N concentrations on 3/1/17.

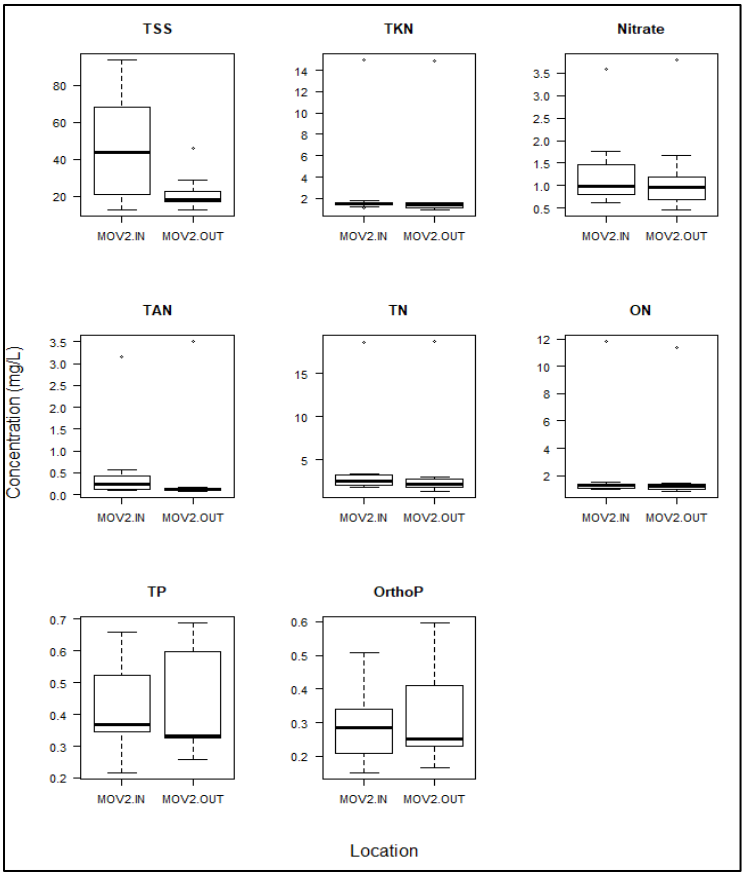

Figure 2-16: Boxplots of influent and effluent concentrations in MOV2 showed reductions in TSS and TAN.

RE_c of both TSS and TN were significant (*Table 2-13*). TSS removal was consistent with RE_c reported for other dry ponds in the literature (*Table 2-23;* Birch et al., 2006; Carpenter et al., 2014; Stanley, 1996; U.S. EPA, 1983). The median effluent TSS EMC was less than the 25 mg/L threshold suggested by Barrett et al. (2004). MOV2 was a long, gently sloping dry pond with a high length to width ratio. It is likely that plug flow conditions existed in the pond (Carleton et al., 2001), increasing hydraulic efficiency and settling potential of particulates. The proportional volume storage and length to width ratio in MOV2 were much higher than those in MOV1, increasing treatment time.

There was no statistically significant difference of soluble pollutants of any species, including TAN, again, similar to the literature (Birch et al., 2006; Carpenter et al., 2014; Stanley, 1996). It is likely that this lack of treatment was due to the minimal number of pollutant removal mechanisms present in dry ponds (Stanley, 1996). MOV2 was ineffective at treating soluble pollutants in runoff.

Table 2-13: Median influent and effluent EMCs of MOV2 and concentration removal efficiencies (RE_c).

	Location			**Significance**		
Pollutant	Rain	MOV2_IN (n=9) *mg/L*	MOV2_OUT (n=9) *mg/L*	Inflow to Outflow (n=9) p-value*	Test	Median RE_c
TSS	-	44	18	**0.01173**	Student's t-test	**52%**
TP	-	0.42	0.44	0.5218	Student's t-test	3.7%
O-PO_4^{3}-	-	0.29	0.25	0.1081	Student's t-test	-11%
TN	0.37	2.52	2.17	**0.04733**	Student's t-test	**10%**
TAN	0.16	0.25	0.11	0.2944	Student's t-test	37%
TKN	0.24	1.49	1.37	0.05254	Student's t-test	11%
ON	0.035	1.27	1.23	0.3248	Student's t-test	3.6%
$NO_{2,3}$-N	0.13	0.99	0.96	0.2256	Student's t-test	6.9%

**bold values are significant*

Effluent cumulative pollutant loads were higher than those of inflow, except for (*Table 2-14*). Likely due to the TSS load associated with direct rainfall on the pond, median TSS load reduction was less pronounced than TSS RE_c. Additionally, the reduction was no longer significant. However, ELR was approximately 40%, which indicates the pond was able to trap TSS. Significant pollutant loads of TP and Ortho-P were both exported, with ELRs of -32% and -43%, respectively. Particularly later in the monitoring period, total inflow TSS loads were much greater than those of outflow. Several storms in June and July with depths exceeding 24 mm and 5-minute peak intensities exceeding 86 mm/hr contributed proportionally large TSS loads. It is

likely that many particles from these intense storms were larger and were more easily settled out in the pond, greatly reducing effluent loads. This was not true for either TN or TP (*Figure 2-17*).

Table 2-14: Cumulative influent and effluent loads and annual event loading of pollutants at MOV2.

Pollutant	**Location**	**MOV2 Cumulative Load**	**Annual Loading**	**Event Load Reduction**	**Test**	**Significance**
		g	*kg/ha/yr*	*%*		*p-value**
TSS	Inflow	43748	80.2	40%	Student's	0.0997
	Outflow	26294	48.2		t-test	
TP	Inflow	386	0.7	**-32%**	Student's	**0.0101**
	Outflow	509	0.9		t-test	
$O\text{-}PO_4^{3-}$	Inflow	267	0.5	**-43%**	Student's	**0.0133**
	Outflow	383	0.7		t-test	
TN	Inflow	4180	7.7	-14%	Wilcoxon	0.164
	Outflow	4770	8.7		Signed Rank	
TAN	Inflow	604	1.11	-4%	Wilcoxon	0.129
	Outflow	626	1.15		Signed Rank	
TKN	Inflow	2990	5.5	-13%	Wilcoxon	0.359
	Outflow	3390	6.2		Signed Rank	
ON	Inflow	2360	4.3	-17%	Wilcoxon	0.0742
	Outflow	2760	5.1		Signed Rank	
$NO_{2,3}\text{-}N$	Inflow	1170	2.1	-18%	Student's	0.200
	Outflow	1380	2.5		t-test	

**bold values are significant*

TSS exceedance probability plot (*Figure 2-18)* shows that TSS was greatly reduced in this practice and met the water quality threshold of 25 mg/L in 7 of 9 events. When comparing influent and effluent TN concentrations to ambient water quality, MOV2 did not meet thresholds; it also performed poorly relative to concentrations assigned by NCDEQ (2017b) to dry ponds (Figure 2-19). Inflow and outflow concentrations of TP uniformly exceeded ambient water quality conditions; however, TP outflow concentrations in MOV2 were much lower than those of other dry ponds in North Carolina. This was likely due to low influent concentrations (*Figure 2-20*).

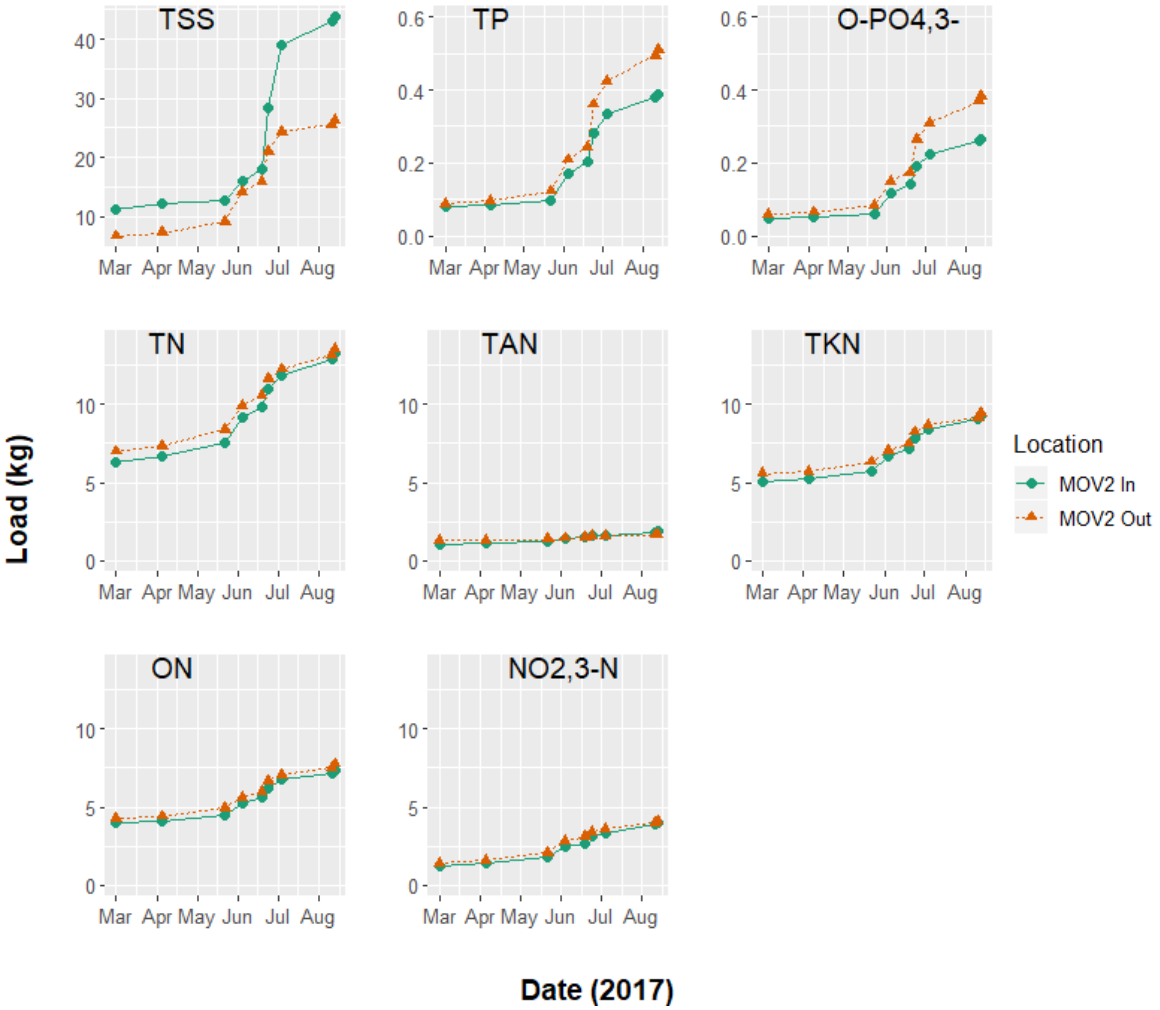

Figure 2-17: Influent and effluent cumulative loads for all pollutants in MOV2.

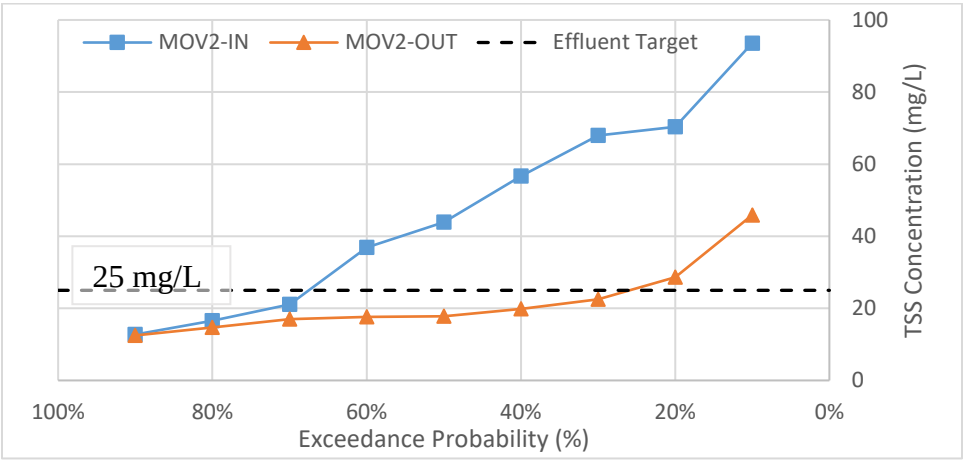

Figure 2-18: TSS exceedance probabilities at MOV2 with respect to stream ambient water quality threshold (Barett et al, 2004).

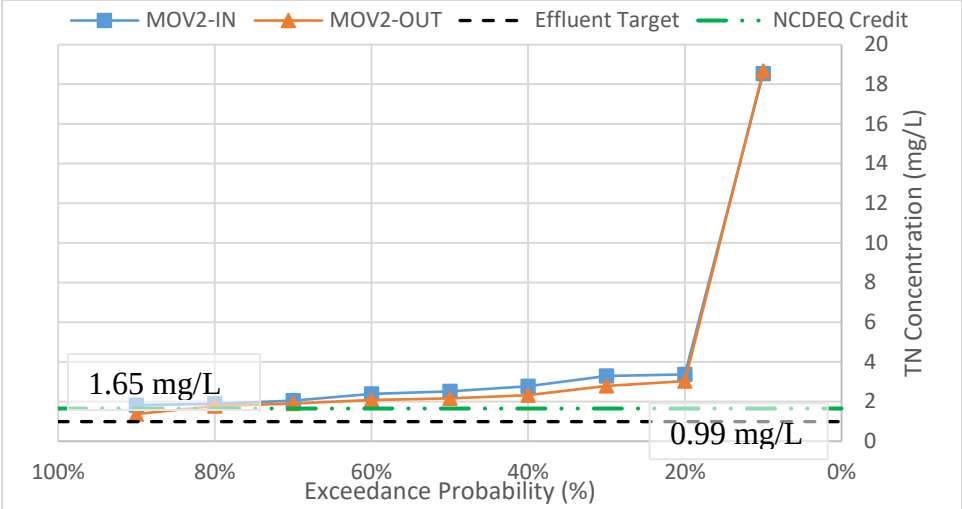

Figure 2-19: MOV2 exceedance probabilities for TN with respect to stream ambient water quality thresholds (McNett et al., 2010) and NCDEQ (2017b) ascribed credit.

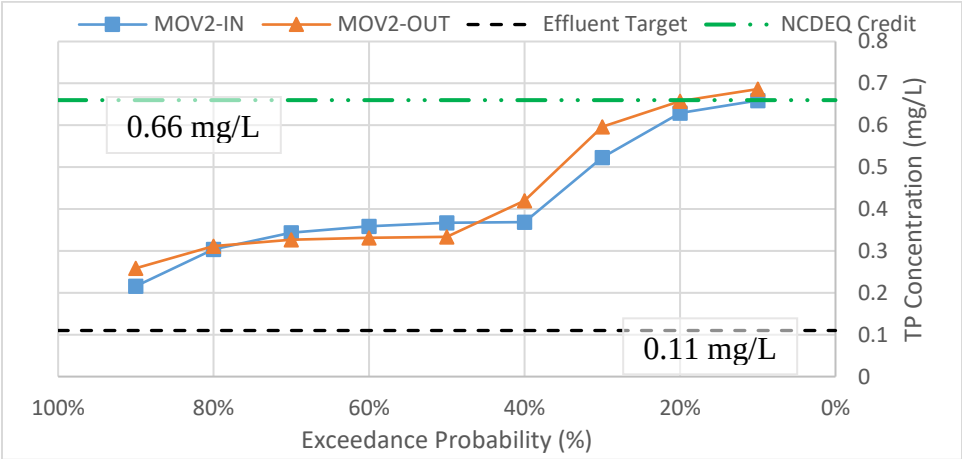

Figure 2-20: MOV2 exceedance probabilities for TP with respect to stream ambient water quality thresholds (McNett et al., 2010) and NCDEQ (2017b) ascribed credit.

The frequency at which MOV2 influent and effluent EMCs failed to meet target concentrations for all pollutants is reported in *Table 2-15*. While influent TSS concentrations exceeded the water quality threshold 67% of the time, exceedance probability at the outlet was reduced to 22%. This is an indication that MOV2 effectively reduced TSS a majority of the time, but also that 33% of influent TSS concentrations met standards before treatment. Other results indicate that this dry pond was ineffective at treating nutrients.

Table 2-15: Exceedance probabilities of EMCs at all sampling stations at MOV2 with respect to ambient water quality thresholds proposed by Barrett et al. (2004) for TSS and by McNett et al. (2010) for other analytes.

Parameter	Target *mg/L*	Expected Probability of EMC Exceeding Target	
		IN	OUT
TSS	25	67%	22%
TP	0.11	100%	100%
TN	0.99	100%	100%
TAN	0.04	100%	100%
TKN	0.4	100%	100%
$NO_{2,3}$-N	0.59	100%	89%

2.4.3: WS Hydrology

WS was monitored from February 2017 to February 2018. Fifty storms were monitored during this period, though outflow was only produced in 31 events due to high initial abstraction in the watershed. In general, no outflow was produced in storms less than 11.4 mm. In 19 of the 22 storms with no outflow, rainfall ranged between 2.4 and 11 mm. Seasonal collection of rainfall was as follows:

- Winter: 2/4/17 – 3/19/17 and 1/10/18 – 3/19/18 (approximately 126% of a season)
- Spring: 3/20/17 – 6/6/17 and 3/20/18 – 3/23/18 (89% of total period)
- Summer: 7/25/17 – 9/21/17 (62% of total period)
- Fall: 9/21/17 – 10/24/17 (36% of the total period)

In three seasons, there was less rainfall in the monitored period than the 30-year normal rainfall (*Table 2-16*). However, more than double the normal rainfall fell in the spring.

Table 2-16: Rainfall at WS during the monitoring period compared to 30-year normal rainfall at Smith Reynolds Airport in Winston-Salem, NC (NCCO,2018).

Season	**Rainfall** *mm*	**% of Period**	**Adjusted normal rainfall** *mm*	**% of Normal Rainfall**
Winter	259.8	126%	328.27	79%
Spring	597.2	89%	263.18	227%
Summer	164.1	62%	222.88	74%
Fall	63.2	36%	98.89	64%
Total	1084.3	63%	913.23	119%

Table 2-17 is a summary of the hydrologic mitigation provided by WS. There was little to no outflow 44% of storms. Clogging of a rain gage from a storm between 4/24/2017 and 4/26/2017 prevented accurate rainfall intensity and depth of that storm. From the manual rain gage reading, this storm was approximately 150 – 160 mm, a 10- 25- annual return interval for a 2-day event. Outflow volumes were not properly recorded for at least two storm events because the bubbler tube unattached.

Table 2-17: Rainfall event characteristics and hydrologic mitigation provided by WS.

Parameter	**Rainfall Depth** *mm*	**Storm Duration** *hrs*	**5-min Peak Intensity** *mm/hr*	**Peak Outflow** *L/s*	**Runoff Volume** m^3
Min	2.4	0:02	4.3	0.0	0.0
Median	11.8	11:21	16.4	0.0	0.20
Mean	15.7	15:02	28.7	4.61	25.2
Max	123.8	55:30	192.4*	54.8	487.8
Sum	1026.1	-	-	-	1410.9

**Rainfall intensity from 4/24/17 storm was not adequately captured*

Characteristics of storms monitored for hydrology and water quality are detailed in *Table 2-18.* Rain gauge failure led to loss of monitoring data between 6/7/2017 and 7/24/2017. A hard freeze on 1/9/2018 caused a reset on monitoring equipment at both sampling locations that led to a loss of data between 10/25/2017 and 1/10/2018.

Table 2-18: Number of events monitored by season for hydrology and water quality in WS.

Location	Total	Winter	Spring	Summer	Fall	Rainfall Median *mm*	Rainfall Sum *mm*
Hydrology	56	18*	23	11*	4*	11.8	1026.1
WQ WS	10	4	4	1	1	28.4	278.1

**loss of storms due to equipment malfunction*

2.4.4: WS Water Quality Analysis

In general, WS appeared effective in reducing pollutants from stormwater. These results have been analyzed using the four previously used metrics. From visual inspection of influent and effluent pollutant concentrations in WS, it appears that all pollutants were reduced in the outflow (*Figure 2-21*). In most cases, apart from TAN, the range of effluent concentrations was also narrower than that of influent concentrations. WS was consistent in its removal of pollutants, except when influent concentrations were too low to be properly treated in the pond.

For nearly all tested parameters, RE_c was significant, with removal percentages ranging between 22 and 49%. Only $O\text{-}PO_4^{3-}$ was not reduced to significant levels. TSS removal rates (median RE_c 49%) were consistent with those found in previous studies and similar to that of MOV2 (Birch et al., 2006; Carpenter et al., 2014; US EPA, 1983). Removal rates of all other pollutants were much greater than those in MOV1 and MOV2, excluding TAN. RE_c for both dissolved forms of nitrogen were substantial and significant (*Table 2-19).*

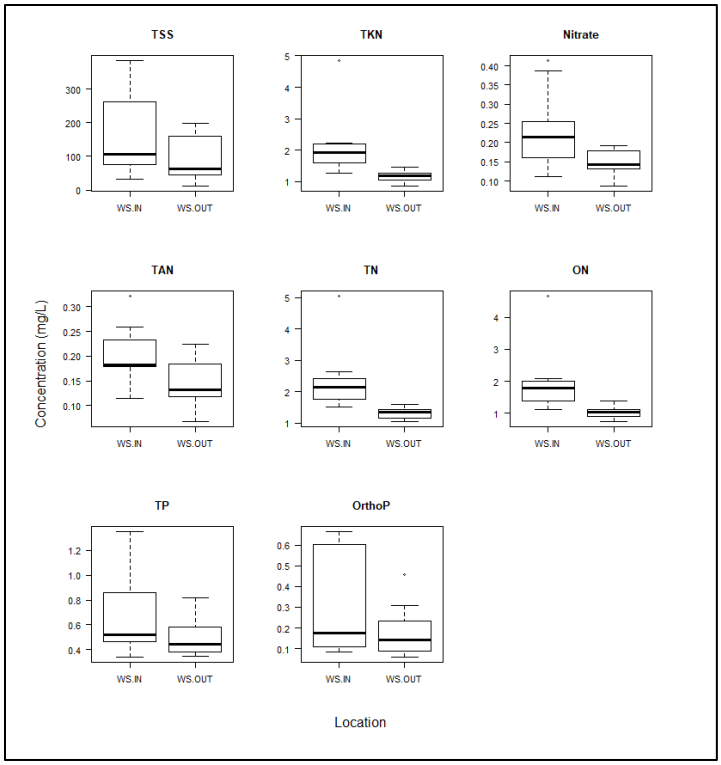

Figure 2-21: Box plots of influent and effluent pollutant concentrations in WS.

Table 2-19: Median influent and effluent EMCs of WS and median concentration removal efficiencies (RE_c).

	Location		**Significance**		
Pollutant	WS_IN (n=10)	WS_OUT (n=10)	Inflow to Outflow (n=10) p-value	Test	Median RE
TSS	105	53	**0.0022**	Paired t-test	**49%**
TP	0.52	0.43	**0.0148**	Paired t-test	**22%**
O-PO_4^{3}-	0.15	0.12	0.189	Paired t-test	31%
TN	2.09	1.25	**0.00195**	Wilcoxon Sign	**41%**
TAN	0.18	0.13	**0.0161**	Paired t-test	**29%**
TKN (n=9)	1.87	1.06	**0.00391**	Wilcoxon Sign	**38%**
ON	1.67	0.98	**0.00195**	Wilcoxon Sign	**42%**
$NO_{2,3}$-N	0.21	0.14	**0.00195**	Wilcoxon Sign	**34%**

**bold values are significant*

Unlike MOV1 and MOV2, soil conditions of WS allowed for some infiltration of runoff, resulting in lower effluent volumes than influent volumes. This generally contributed to overall

load reduction of pollutants. In most cases, ELR was higher than median RE_c (*Table 2-20*), suggesting that a representative median performance estimate, particularly one based on concentrations, does not wholly describe basin performance. ELR accounts differences in load reductions among storms and weights storms proportionally by volume. Volume reductions from storm events contributed to total load reduction.

Based on load removal, WS treated TSS less similarly to what was found for other dry ponds (Stanley, 1996; Birch et al., 2006; Carpenter et al., 2014). The opposite was true for TP and TN, where ELR was 55% and 25%, respectively. Rate of pollutant load removal of TSS, TN, and TP was relatively consistent across storms (*Figure 2-22*).

Table 2-20: Influent and effluent cumulative loads and annual event loading of pollutants in WS.

Pollutant	**Location**	**WS Cumulative Load**	**Annual Loading**	**Event Load Reduction**	**Test**	**Significance**
		g	*kg/ha/yr*	*%*		*p-value**
TSS	Inflow	55400	206	**43%**	Student's t-test	**0.004387**
	Outflow	31500	118			
TP	Inflow	396	1.47	**61%**	Paired Samples	**0.00195**
	Outflow	153	0.57		Sign Test	
$O\text{-}PO_4^{3-}$	Inflow	89.6	0.33	22%	Wilcoxon	0.0644
	Outflow	69.0	0.26		Signed Rank	
TN	Inflow	776	2.89	**49%**	Wilcoxon	**0.00195**
	Outflow	394	1.48		Signed Rank	
TAN	Inflow	70.8	0.26	**39%**	Wilcoxon	**0.00586**
	Outflow	42.9	0.16		Signed Rank	
TKN	Inflow	670	2.67	**52%**	Student's t-test	**0.0313**
(n = 9)	Outflow	328	1.29			
ON	Inflow	639	2.38	**51%**	Wilcoxon	**0.00195**
	Outflow	308	1.16		Signed Rank	
$NO_{2,3}$-N	Inflow	65.6	0.24	**35%**	Student's t-test	**0.00450**
	Outflow	42.4	0.16			

**bold values are significant*

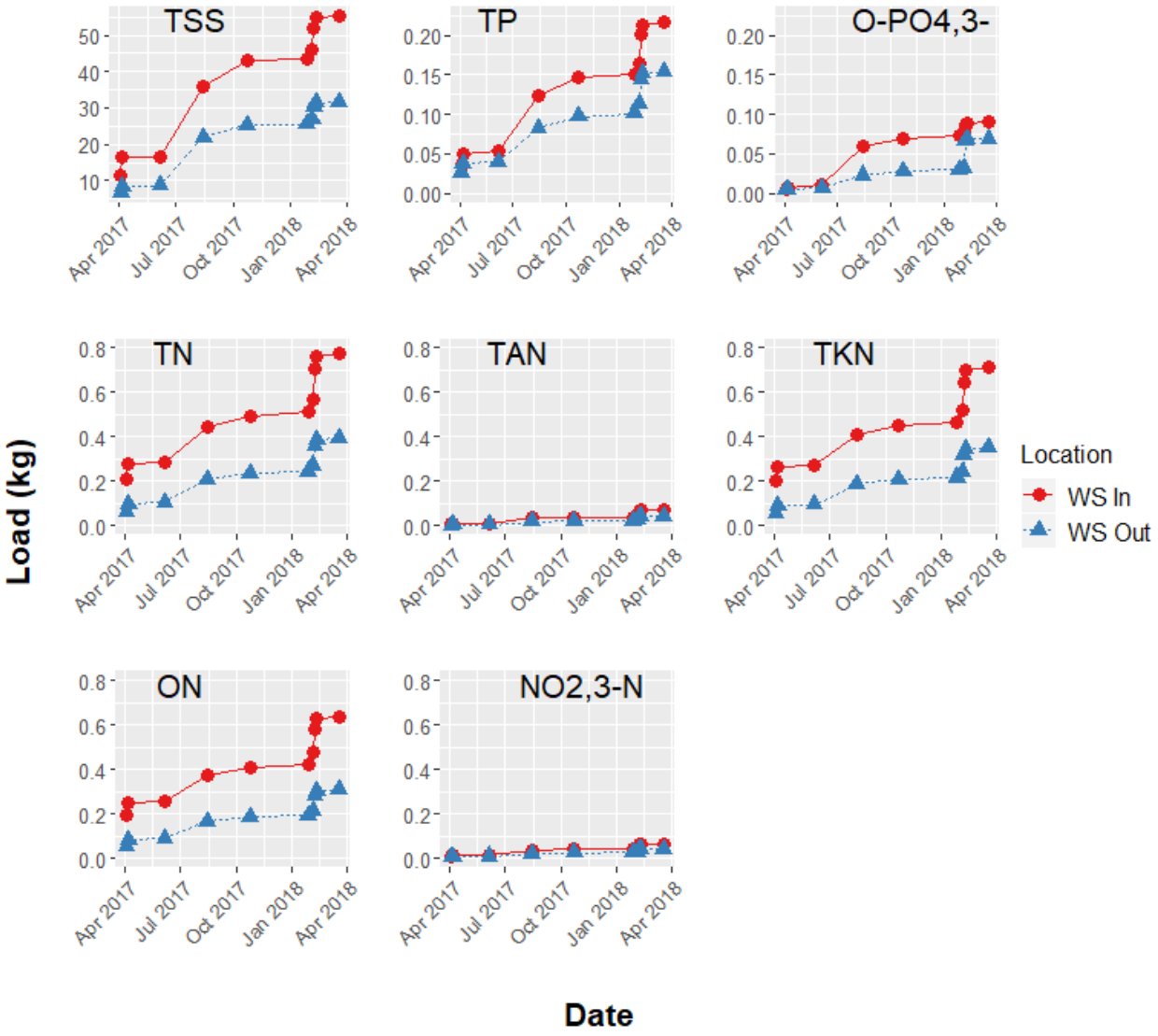

Figure 2-22: Influent and effluent cumulative load plots for all pollutants in WS.

Although TSS concentration and load reductions were large and significant in WS, effluent concentrations in general did not meet water quality thresholds (*Figure 2-23).* Nearly all effluent concentrations for TSS, TN, and TP exceeded levels for "good" water quality conditions in the piedmont of NC. However, in nearly all sampled events, both TN and TP concentrations were reduced to a lower level than is assigned to dry ponds by NCDEQ (2017b; *Figure 2-24 and 2-25*).

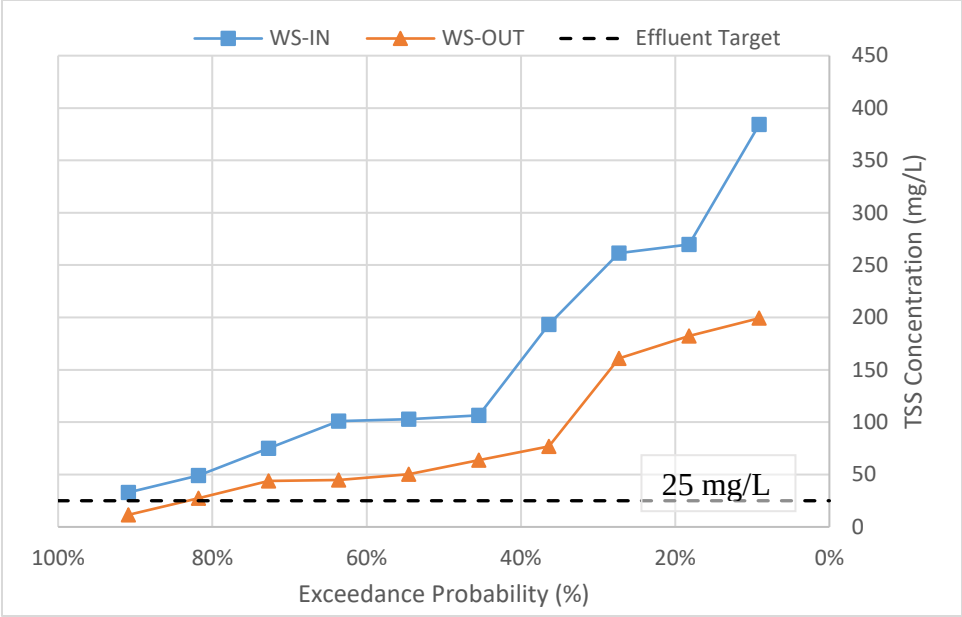

Figure 2-23: WS exceedance probabilities for TSS with respect to stream ambient water quality thresholds (Barrett et al., 2004).

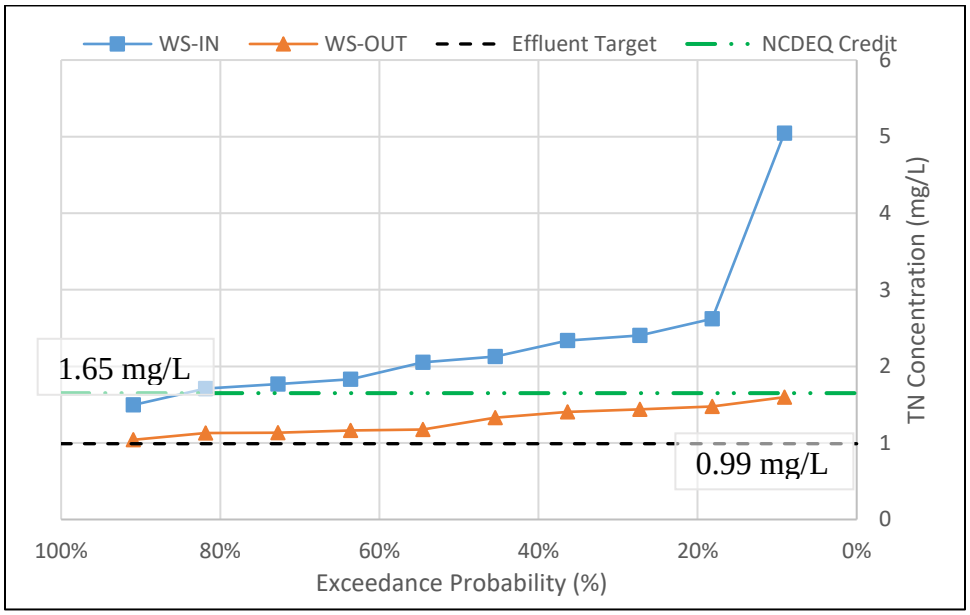

Figure 2-24: WS exceedance probabilities for TN with respect to stream ambient water quality thresholds (McNett et al., 2010) and NCDEQ (2017b) ascribed credit.

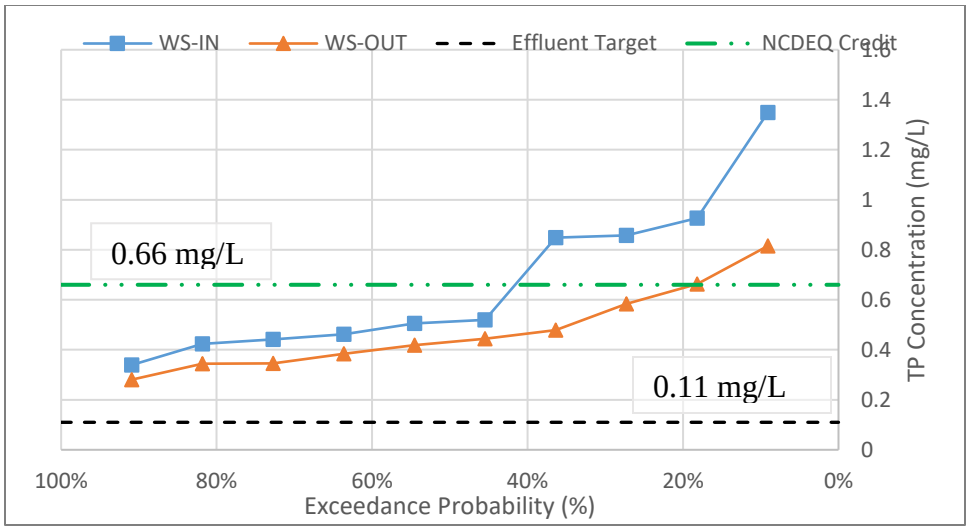

Figure 2-25: WS exceedance probabilities for TP with respect to stream ambient water quality thresholds (McNett et al., 2010) and NCDEQ (2017b) ascribed credit.

A summary of exceedance probabilities based on the standard from Barrett et al. (2004) and McNett et al. (2010) is displayed in *Table 2-21*. Only $NO_{2,3}$-N concentrations met standards, but they were below acceptable levels for both influent and effluent. It is possible that pollutant removal mechanisms other than sedimentation were present in this pond. Vegetation, which was often overgrown, may have provided some nutrient uptake and filtration of particles. Riprap present near the inlet may have provided additional filtration and energy dissipation to aid in sedimentation.

Table 2-21: Exceedance probabilities of EMCs at all sampling stations in WS with respect to ambient water quality thresholds proposed by Barrett et al. (2004) for TSS and by McNett et al. (2010) for other analytes.

Parameter	Target *mg/L*	Expected Probability of EMC Exceeding Target	
		IN	OUT
TSS	25	100%	90%
TP	0.11	100%	100%
TN	0.99	100%	100%
TAN	0.04	100%	100%
TKN	0.4	100%	100%
$NO_{2,3}$-N	0.59	0%	0%

2.4.5: Comparison of Studied Dry Ponds

Performance among dry ponds was highly variable. Sometimes pollutants were trapped; others were exported. Neither length to width ratio nor basin configuration predicted performance. In part, RE_c was dependent on influent concentration, which is consistent with the literature (Lenhart and Hunt 2011; Strecker et al. 2001).

Expected effluent concentrations in dry ponds (NCDEQ, 2017b) for TN and TP are 1.65 mg/L and 0.66 mg/L, respectively, and influent concentrations lower than these values might be considered irreducible by dry ponds. When influent concentrations are near irreducible levels, concentration removal efficiencies have been shown to be a poor representation of SCM performance (Lenhart and Hunt, 2011). Influent concentrations of TSS and TN were higher than dry pond irreducible levels (NCDEQ, 2017b), which should have allowed for treatment (*Figure 2-26 and 2-28*). However, it appeared that only MOV2 and WS were capable of removing TSS, and TN was only reduced in WS. In most storm events among all ponds, *influent* TP concentrations were lower than expected dry pond effluent concentrations (NCDEQ, 2017b), which limited treatment potential in these dry ponds (*Figure 2-27*).

When using RE_c as the metric of comparison, WS outperformed the other ponds (*Table 2-22*). Though few articles report for soluble pollutant treatment by dry ponds, those that do have generally lower removal efficiencies than that of WS, especially for $NO_{2,3}$-N (Rosenzweig et al., 2011; Stanley, 1996; US EPA, 1983). Median RE_c of TAN among ponds was most similar, though neither MOV1 nor MOV2 showed significant improvement. However, influent concentrations of TAN were much lower in WS than those of MOV1 and MOV2. On a load removal basis, WS yielded the best results. In part, load removal of WS can be attributed to

volume reduction; the Morrisville ponds did not appreciably reduce volumes (*Table 2-22*). In all cases of significant change, both MOV1 and MOV2 exported loads rather than reduce them.

Table 2-22: Median concentration removal efficiency/event load reduction at each dry pond (%).

Basin	TSS	TP	PO_4^{3-}	TN	TAN	TKN	ON	$NO_{2,-3}$-N
MOV1	-8/-22	-45/**-11**	-43/10	**-25/-25**	26/27	-23/-22	-42/**-37**	-7/-11
MOV2	**52**/40	4/**-32**	-11/**-43**	**10**/-14	37/-4	11/-13	4/-17	7/-18
WS	**49/43**	**22/61**	31/**22**	**41/49**	**29/39**	**38/52**	**42/51**	**34/35**

**Bold values are significant*

RE_cs and ELRs alone do not sufficiently characterize dry pond performance. Often, WS influent concentrations were much higher than those of MOV1 and MOV2, allowing more opportunity for removal before concentrations reached irreducible levels. This was the case for TSS *(Figure 2-26*) and TP (*Figure 2-27*). However, influent concentrations of TN at WS were similar to or lower than those of MOV1 and MOV2 (*Figure 2-28*). Still, RE_c of TN, $NO_{2,3}$-N, TKN, and ON in WS were significant and greater than those of the other ponds.

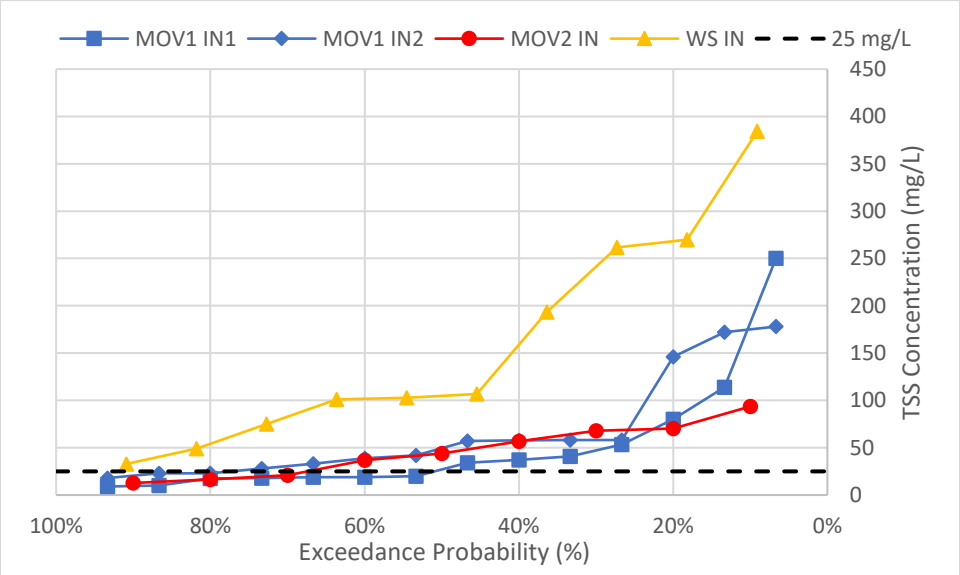

Figure 2-26: Exceedance probabilities of influent TSS concentrations at both MOV1 inlets, MOV2, and WS with respect to water quality threshold proposed by Barrett et al. (2004).

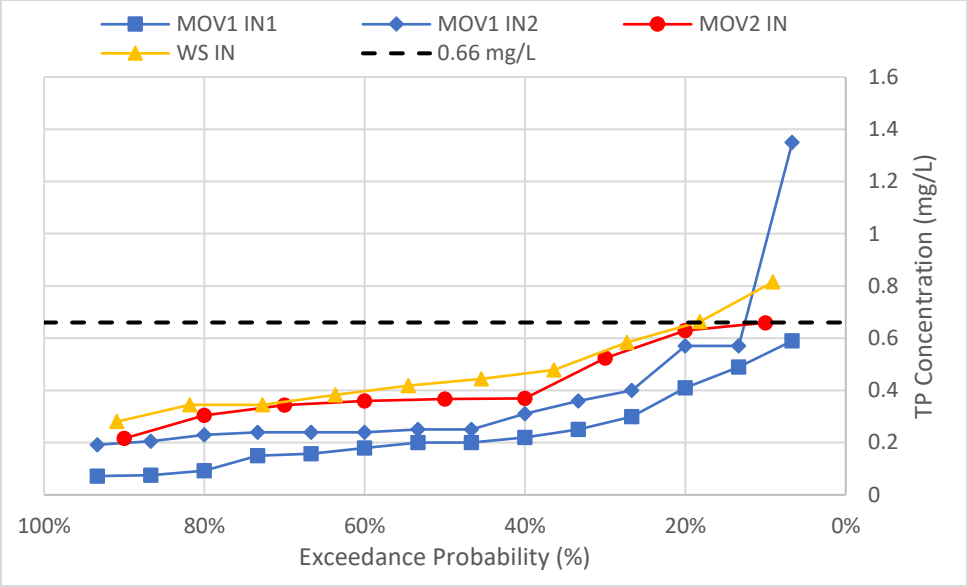

Figure 2-27: Exceedance probabilities of influent TP concentrations at both MOV1 inlets, MOV2, and WS with respect to NCDEQ ascribed credit for dry ponds (2017b).

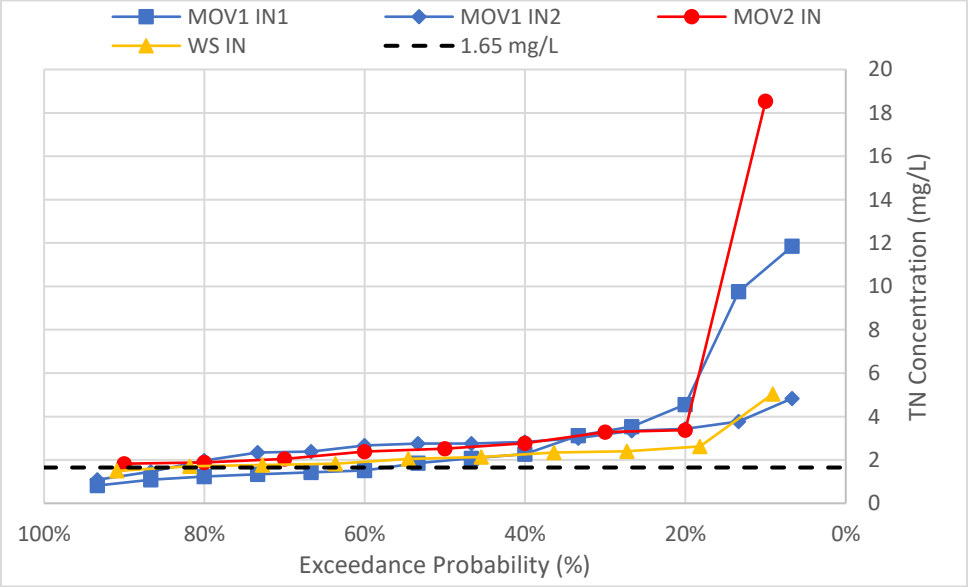

Figure 2-28: Exceedance probabilities of influent TN concentrations at both MOV1 inlets, MOV2, and WS with respect to NCDEQ ascribed credit for dry ponds (2017b).

Assessing performance using ambient water quality thresholds provides a modestly different outcome. MOV2 appeared to be the most successful dry pond, as TSS concentrations were less than 25 mg/L in 78% of events. TSS effluent concentrations were less than 25 mg/L for only 18% and 10% of cases at MOV1 and WS, respectively. Effluent concentrations of TN and TP always exceeded thresholds at all ponds. Effluent concentrations of $NO_{2,3}$-N in WS met water

quality thresholds (McNett et al., 2010) most often, though influent concentrations were also below these levels. Despite low $NO_{2,3}$-N concentrations at WS, both RE_c and ELR of $NO_{2,3}$-N in WS were much higher than in the Morrisville ponds, indicating a high ability to reduce $NO_{2,3}$-N. It is possible that other pollutant removal mechanisms were present in WS that aided in treatment. Vegetation that was often overgrown may have provided filtration or an uptake of nutrients in the pond.

TP influent concentrations at all ponds were generally lower than the NCDEQ-assigned (2017b) effluent concentrations for dry ponds of 0.66 mg/L. However, effluent TP concentrations were never lower than the 0.11 mg/L water quality threshold proposed by McNett et al. (2010). Reported exports of TP in MOV1 were likely due in part to influent concentrations that were below dry pond treatment threshold (NCDEQ, 2017b), limiting removal (Lenhart and Hunt, 2011). Particle-bound TP may also have been exported with sediment that was discharged from the basin. This apparent export was likely due to TSS and TP that were exported during large storm events. It is likely that there was deposition of these pollutants in the basin during smaller storm events and baseflow periods.

Pollutant effluent EMCs of the studied dry ponds are compared to treatment of other SCMs in *Table 2-23.* Effluent TSS concentrations were similar to those of other dry ponds but much higher than those of other SCMs. Concentrations of inorganic nitrogen species in the studied ponds were much higher than those of any other SCMs and most comparable to other dry ponds. Additionally, the majority of the nitrogen exported from the basins was in the form of organic nitrogen, for which TKN was used as a comparison due to a lack of reporting of ON concentrations in other studies. These effluent concentrations were high in the studied dry ponds, though comparable to other dry ponds.

Table 2-23: Effluent water quality of dry ponds studied herein compared to various SCMs from the literature.

SCM Type	Reference	Name/Location	Mean Effluent EMC (mg/L)						
			TSS	TP	PO_4^{3-}	TN	TKN	$NO_{2,3}$-N	TAN
Dry Pond	herein	MOV1	80	0.41	0.22	3.38	2.66	0.71	0.32
Dry Pond	herein	MOV2	22	0.44	0.33	4.01	2.77	1.24	0.48
Dry Pond	herein	WS	85	0.48	0.18	1.29	1.13	0.14	0.14
Dry Pond	Birch et al. (2006)	Sydney, Australia	93	0.23	-	2.91	1.13	1.82	-
Dry Pond	Stanley (1996)	Greenville, NC	32	-	0.10	-	-	0.36	0.12
Dry Pond	Caltrans (2004)	California[a]	39	0.32	0.14	2.83	1.85	0.98	-
Dry Pond	Carpenter (2014)	Quebec City, Canada	75	-	-	-	-	-	0.19
CSW	Lenhart & Hunt (2011)	River Bend, NC	41	0.23	0.09	1.11	0.94	0.17	0.08
CSW*	Line et al. (2008)	The Piedmont Region, NC	18	0.99	0.13	1.00	0.87	0.13	0.14
CSW*	Line et al. (2008)	Asheville, NC	31	0.12	0.01	0.94	0.79	0.15	0.08
CSW	Hathaway & Hunt (2010)	Mooresville, NC	9	0.10	-	0.72	0.67	0.07	0.03
Wet Pond	Winston et al. (2013)	DOT/Durham, NC	30	0.17	0.12	1.05	0.97	0.08	0.11
Wet Pond	Winston et al. (2013)	Museum/Durham, NC	24	0.11	0.07	0.41	0.35	0.06	0.05
Wet Pond	Wu et al. (1996)	Waterford/Charlotte, NC	44	0.11	-	-	0.73	-	-
Wet Pond	Wu et al. (1996)	Runaway Bay/Charlotte, NC	22	0.08	-	-	0.63	-	-
Wet Pond	Wu et al. (1996)	Lakeside/Charlotte, NC	7	0.08	-	-	0.59	-	-
Bioretention	Hunt et al. (2008)	Charlotte, NC	20	0.13	-	1.14	0.70	0.43	0.10
Bioretention	Brown & Hunt (2011)	Sandy/Rocky Mount, NC	17	0.23	0.10	1.31	0.82	0.49	0.07
Bioretention	Brown & Hunt (2011)	SLC/Rocky Mount, NC	16.9	0.09	0.01	0.43	0.31	0.12	0.06

**median effluent EMC*
[a]Compilation of six basins

In general, these results are surprising, considering dry pond design attributes. WS had a very low length to width ratio, proportionally little storage volume, and an outlet structure with a large orifice that did not detain flows for long periods. With these attributes, pollutant removal opportunities were expected to be minimal. In some cases, higher inflow concentrations in WS made percent removal more likely. Volume loss from infiltration in WS among storms ranged from 0.4 m^3 to 7.3 m^3, providing additional load reduction not present in MOV1 or MOV2. Additional filtration and sedimentation may have occurred aided by the riprap and vegetation present at the pond's inlet.

Additionally, this study analyzed the performance of a “soggy bottom” dry pond (MOV1). Several studies have quantified water quality changes due to “failed” SCMs that have taken on wetland features, including dry ponds (Rosenzweig et al., 2011) and infiltration basins (Natarajan and Davis, 2016), with positive results. However, MOV1 was different; it was the worst-performing dry pond examined herein. Evidence indicates that this basin was a source of nitrogen. Perhaps organic matter accumulated in the basin inter-event was resuspended by storm flows and exported from the pond. In dry ponds, no permanent pool is present to maintain stability of the pond bottom; a permanent pool, would have limited resuspension. Sediment was observed to accumulate near the outlet, but no deep pool was present at the outlet to prevent resuspension and export. Perhaps assessing $NO_{2,3}$-N reduction is the most relevant measure of wetland-like performance (Lee et al., 2009). Concentration “removal” efficiencies ranged between -65 and 76%, thus were highly variable. While mean concentration removal was essentially zero, cumulative loads were *added* by 13% over the entire period. Neither result was significant. This study took place between January and September of 2017, almost the entirety of the growing season. Periods of dormancy, where plants can decompose, export $NO_{2,3}$-N from SCMs (Rosenzweig et al., 2011). It is possible that $NO_{2,3}$-N export rates would increase during winter months.

Based on these data, potential exists for performance enhancement of these basins through retrofitting by implementing features that enhance pollutant removal mechanisms in other SCMs. For example, MOV1 already has “hints of” wetland characteristics; perhaps making this dry pond more wetland-like would enhance pollutant removal. CSWs provide much better water quality treatment than dry ponds (*Table 2-23*), and such a conversion would not greatly decrease storage volume. NCDEQ (2017b) assigns stormwater wetlands with effluent

concentrations of 1.12 mg/L for TN and 0.18 mg/L for TP, both of which are must lower than expected concentrations of dry ponds.

2.5: Summary and Conclusions

This study assessed water quality treatment in three dry ponds in the Piedmont Region of North Carolina. The major findings of the assessment are as follows:

- There were significant increases in effluent concentrations and loads over inflow at MOV1. At MOV2, RE_cs were significant for TSS and TN, but effluent loads of TP and O-PO_4^{3-} significantly increased over influent loads. WS was the most successful dry pond, with large and significant RE_cs for TSS, TP, TN, TAN, TKN, ON, and $NO_{2,3}$-N, and significant ELRs for all pollutants sampled.
- "Ideal" design features did not effectively predict treatment. Length to width ratio, detention time, and storage volume were proportionally lower in WS than in the Morrisville ponds, but treatment provided by WS was the best. There may have been additional pollutant removal mechanisms provided by riprap and vegetation in WS that improved treatment.
- The presence of wetland vegetation and shallow pools of standing water in MOV1 did not improve treatment. Without a permanent pool near the outlet to settle particles, loose sediment and organic matter accumulated in the basin was exported from the pond during storm events.
- Largely, influent concentrations of TSS, TP, and TN were indicators for concentration and load removals, especially in WS and MOV2.
- Dry pond effluent concentrations herein were similar to or higher than those of other dry ponds. These concentrations were much higher than those of other types of SCMs.

Structures that provide pollutant removal in other SCMs may provide a template to retrofit dry ponds for treatment enhancement.

- Dry pond water quality treatment is poor and does not match that of other SCMs. Retrofitting dry ponds to function more like other SCMs may improve dry pond effluent water quality.

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CHAPTER 3: A Dry Pond to Constructed Stormwater Wetland Conversion to Enhance Water Quality

3.1: Abstract

A dry pond is a stormwater control measure (SCM) known for poor runoff water quality treatment because its design does not provide ample mechanisms for pollutant removal. Altering, or retrofitting, dry ponds to incorporate more pollutant removal mechanisms can improve treatment. In this study, two dry ponds (MOV1 and WS) were converted to constructed stormwater wetlands (CSWs). Underlying soils in the MOV1 watershed [4.15 ha, 35% directly connected impervious area (DCIA)] were in hydrologic soil group D, and those of the WS watershed (0.93 ha, 0% DCIA) were in group C. Two retrofits were installed: (1) outlet structure modification to create a permanent pool and increase detention time and (2) wetland vegetation installation. The water quality storage capacity was 65% of the design volume, the volume required to treat the first 25 mm of runoff, in MOV1 and 5% in WS. Water quality treatment was monitored in dry ponds in their original and retrofitted states, and effluent concentrations were compared. The MOV1 retrofit increased effluent concentration reductions from original dry pond removal rates by 58%, 65%, 69%, 64%, 82%, 71%, and 80% for total suspended solids (TSS), total phosphorus (TP), orthophosphate (O-PO_4^{3-}), total nitrogen (TN), total Kjeldahl nitrogen (TKN), total ammoniacal nitrogen (TAN), and organic nitrogen (ON), respectively. Mass exports were reduced from those of pre-retrofit by 89%, 60%, 57%, 71%, 75%, 69%, and 75% for TSS, TP O-PO_4^{3-}, TN, TKN, TAN, and ON, respectively. While nitrate-nitrite nitrogen ($NO_{2,3}$-N) was not significantly reduced during storm events, 99% removal rates during baseflow periods suggested denitrification potential. Few storms were monitored post-retrofit in WS; there were no significant changes detected in effluent concentrations or loads between monitoring periods.

This study shows that incorporating wetland features can improve water quality treatment in dry ponds, but these enhancements are limited by detention time and storage capacity.

3.2: Introduction

Urban stormwater runoff has become a large contributor of pollution in water bodies throughout the US, increasing sediment and nutrient loads that affect water quality of drinking water supply and recreational waters. Increased sediment loads can lead to altered stream dynamics and habitat degradation. High nutrient concentrations can lead to eutrophication, which can have significant ecological and economic effects.

National regulations have been enforced to reduce nonpoint source pollution, which includes urban stormwater runoff. In sensitive watersheds that serve as drinking water supply or critical habitat, pollutant limits are even more stringent. Pollutant load limits in urban settings require developers building new constructions to minimize their environmental impacts. Currently, developers in North Carolina can reduce pollutant loads in new developments either by paying offset fees or by implementing stormwater control measures (SCMs). Fees are typically paid to a third party that creates offsite mitigation at a hefty cost to the developer. Offsite mitigation is often far from the project site and may not directly benefit the watershed. SCM implementation is used on-site to mitigate flooding, control post-development hydrology, and treat runoff from urban developments. This option of management provides more direct watershed benefits.

There are many types of SCMs, and each is unique in its design, functions, and costs. With stricter regulations, there is more pressure on developers to implement SCMs with high treatment capabilities. In the near future, some existing SCMs may not be sufficient to meet pollutant reduction requirements. Dry ponds, for example, are a commonly implemented SCM,

used historically for peak flow mitigation and flood control with little water quality treatment. Dry ponds are often preferentially implemented due to low costs and ease of implementation. Because of this, dry ponds are nearly ubiquitous, including throughout the Mountains and Piedmont Ecoregions of North Carolina. However, their water quality treatment ability falls far below that of other SCMs.

Because of their pervasive presence and underperformance, dry ponds are a large contributor of stormwater pollutant loads. Replacing every dry pond with more effective SCMs would be cost-prohibitive and time consuming; however, features within dry ponds may be altered or improved to enhance pollutant treatment.

Retrofitting, or making alterations within an SCM to make it work better, may be an effective way to improve dry pond performance. Several studies have tested various dry pond retrofits, like orifice reduction on the outlet to increase detention time (Carpenter et al., 2014) batch-type treatment (Middleton and Barrett, 2008), and the effects of natural alterations made to these ponds (Rosenzweig et al., 2011). Though results throughout these studies vary, some factors remain consistent. Longer detention times can enhance pollutant removal, particularly if particle resuspension is minimized. Introducing wetland plants into SCMs has also been found to be effective in removing pollutants (Natarajan and Davis, 2016).

The effectiveness of these retrofits has been largely based on the exposure of runoff to pollutant removal mechanisms. The sole mechanism for pollutant removal in dry ponds is sedimentation, which is limited by short detention times and resuspension of particles near the outlet (Shammaa et al., 2002). Sedimentation also fails to reduce concentrations of dissolved pollutants. As previous retrofits have shown, introducing other mechanisms of pollutant removal by emulating other SCMs may be a viable option to improve dry pond performance.

Amongst SCMs, the components of dry ponds are most similar to those in wet ponds and constructed stormwater wetlands (CSWs). Both wet ponds and CSWs treat pollutants more effectively than dry ponds, making them ideal templates for dry pond enhancement. However, the deep permanent pools required to create a wet pond environment would drastically reduce storage availability in dry ponds. Alternatively, CSWs are designed with a much shallower permanent pool that would minimally alter dry pond storage capacity. Available storage volume affects both treatment capacity and flood control, making a wetland conversion a more viable option.

Many features of CSWs contribute to pollutant removal, including wetland vegetation, a small orifice to extend detention times, a permanent pool of water, and a flow path to prevent short circuiting. Many of these features can be utilized in existing dry ponds to make them more effective. Alterations must be chosen carefully to maximize benefits without requiring huge financial investments. Additionally, certain aspects of dry pond design may limit treatment in converted ponds. CSW design guidelines limit allowable ponding depth for the water quality treatment volume that would lead to higher loading ratios in converted dry ponds and limit function. CSW features must be selected carefully to create the most effective dry pond retrofit design.

The objectives of this study were to (1) create a simple and cost-effective retrofit design to convert dry ponds to constructed stormwater wetlands, (2) quantify pollutant removal changes associated with the retrofit, and (3) assign monetary values to a retrofit of this type.

3.3: Methods

Three sites were used to quantify the impact of a wetland retrofit on dry pond water quality. Two separate studies took place, located in Morrisville and in Winston-Salem. Each

study was based on calibration and treatment periods, where both initial conditions and retrofit conditions were monitored.

3.3.1: Morrisville Study Design

3.3.1.1: Site Description

Two dry ponds (MOV1 and MOV2) monitored in this study were located in Providence Place, a residential area in Morrisville, North Carolina (*Figure 3-1*). These dry ponds were monitored in a paired watershed study design, with a control and a retrofit pond. This neighborhood had coordinates 35°51’10.48” N and 78°50’46.90” W. MOV1’s watershed was approximately 4.1 ha, 35% of which was impervious surfaces, including roof, sidewalk and driveway, and roadway. MOV2’s watershed of 2.68 ha had similar land use percentages (*Table 3-1*). The majority of underlying soils in this region were considered urban land by the Natural Resource Conservation Service (NRCS, 2018). Existing underlying soils were largely in hydrologic soil group D, which are highly impermeable (*Appendix A*).

Table 3-1: Watershed characteristics of the studied dry ponds.

	MOV1	**MOV2**	**WS**
Latitude (N)	35°51’11.14”	35°51’10.49”	36°06’22.93”
Longitude (W)	78°50’49.95”	78°50’46.45”	80°18’28.58”
Watershed area (ha)	4.15	2.52	0.93
Roof (ha)	0.48	0.16	0
Sidewalk/Driveway (ha)	0.40	0.09	0.16
Roadway (ha)	0.57	0.38	0
Landscaped (ha)	0.48	1.22	0.76
% impervious	35%	28%	18%
Composite Curve Number	86	85	78

Figure 3-1*: Watershed area for dry ponds MOV1 and MOV2 in Providence Place.*

MOV1's surface area was 0.29 ha, draining a 4.15-ha watershed (*Figure 3-2*). Water flowed into the pond through two culvert inlets – a 60-cm inlet on the south end of the pond (MOV1_IN1) and a 75-cm inlet on the northwest edge (MOV1_IN2, *Figure 3-3*). The pond had one outlet riser structure with an 11.5-cm orifice and a 5.8-m emergency broad-crested weir at a height of 75 cm. There was a consistent baseflow from MOV1_IN2 from groundwater infiltration and intrusion into the culvert before entering the basin, which caused portions of the pond to be constantly wet, where wet areas of the dry pond took on wetland characteristics, including the growth of wetland vegetation.

MOV2 (*Figure 3-2 and 3-3*) had a surface area of 0.21 ha with a watershed of 2.68 ha. A 90-cm concrete pipe entered the pond with an outlet riser structure orifice of 8.5 cm. A 4.9-m broad-crested weir at a height of 1.22 m conveyed large storms. The pond's geometry was long and thin, with length to with ratio of 4.5:1, and the Town of Morrisville assumed that water quality treatment in the pond is high by dry pond standards. This pond dried out between storms, except for a spot of standing water in the inlet pipe from poor grading. No wetland vegetation was present in MOV2 during the study.

Figure 3-2: Dry ponds MOV1 (left) and MOV2 (right).

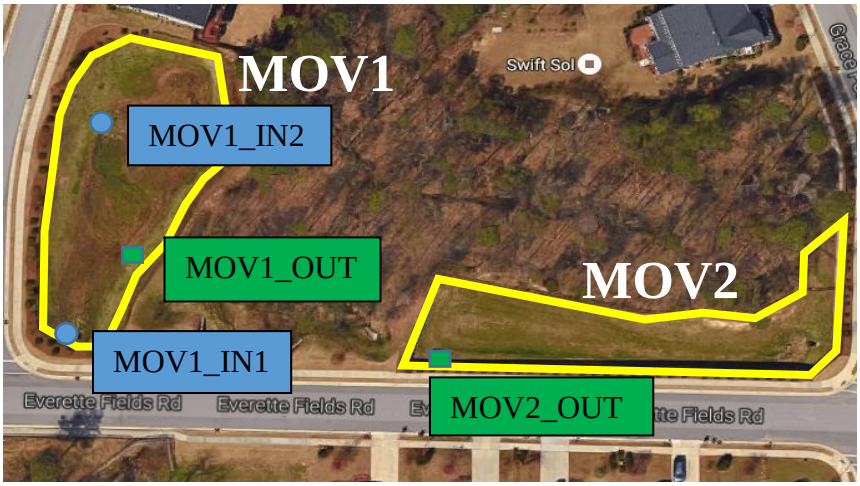

Figure 3-3: MOV1 and MOV2 are located within the same neighborhood block in Providence Place.

3.3.1.2: MOV1 Monitoring Set up

Both a paired watershed study design (Clausen and Spooner, 1993) and an upstream/downstream design (Spooner et al., 1985) were utilized for this monitoring study. The paired watershed approach requires two watersheds to be analyzed side-by-side, with two phases: calibration and treatment. A linear relationship between the two watersheds is created during the calibration phase, and changes from the retrofit can be detected when this linear relationship is altered. One watershed (in this case a dry pond) remains as the control for the

duration of the study while the other is modified after the calibration phase. The upstream/downstream approach utilizes the same methodology, except that the upstream (in this case a dry pond inlet) station is considered the control, and changes made during the study occur between the upstream and downstream stations. The complete model that combines the upstream/downstream and paired watershed designs can be found in Section 4.3.5. Monitoring took place between January 2017 and July 2018. Sampling stations were installed at each of the inlet and outlet locations in MOV1 and at the outlet of MOV2 (*Table 3-2*).

Table 3-2: Monitoring set up of sampling stations at MOV1 and MOV2 in Morrisville.

Site	Monitoring Equipment	Trigger to Enable	Sample Pacing*	Collection
MOV1_IN1	ISCO 6712™ water quality sampler 2 HOBO™ tipping bucket rain gauges	rainfall exceeded 3.05 mm in 6 hours	rainfall-paced, every 0.76 mm of rain	1 10-L composite bottle
MOV1_IN2	ISCO 6712™ water quality sampler 1 HOBO™ tipping bucket rain gauge	rainfall exceeded 3.05 mm in 6 hours	rainfall-paced, every 0.76 mm of rain	1 10-L composite bottle
MOV1_OUT**	ISCO 6712™ water quality sampler ISCO 730™ bubbler flow module	water level exceeded 4.6 cm (1.5 cm) from orifice invert	flow-paced, every 17.0 m^3 (11.m^3) of runoff	24 1-L bottles
MOV2_IN	ISCO 6712™ water quality sampler ISCO 750™ area velocity flow module	water level reached 18.3 cm above culvert invert	flow-paced, every 5.66 m^3 of runoff	1 10-L composite bottle
MOV2_OUT	ISCO 6712™ water quality sampler ISCO 730™ bubbler flow module	None	flow-paced, every 5.66 m^3 of runoff	24 1-L bottles

**sample was changed on some occasions to capture extreme events*
***pre- and post-retrofit enable and pacing were not equivalent. Numbers in parenthesis pertain to post-retrofit*

Rainfall depth and intensity were measured using a manual rain gauge and a HOBO™ tipping bucket rain gauge located in MOV1 1.5 m above the ground free from any obstructions to rainfall (*Figure 3-4A)*. This rain gauge was considered valid for precipitation depth of both ponds. The two inlets in MOV1 were equipped with ISCO 6712™ water quality samplers (*Figure 3-4B, 3-4E)* with tipping bucket rain gauge attachments for rainfall-paced sampling (*Figure 3-4A, 3-4D)*, where rainfall intensity and accumulation dictated when water quality samples were taken. At each of these stations, a composite sample was created based on rainfall intensity that collected a sample every 0.76 mm after an initial minimum rainfall depth of 3.05 mm. It was assumed that inflow rates in MOV1 were proportional to the rainfall intensity of the storm. Both MOV1_OUT and MOV2_OUT were equipped with ISCO 6712™ water quality samplers with ISCO 730™ bubbler flow modules (*Figure 3-4C, 3-5*). In each pond, the sample intake was located in the same position during pre- and post-retrofit monitoring, which meant the sample intake of MOV1_OUT was near the base of the water column. Outflow rates were determined using the recorded level and stage-discharge equations provided (Section 3.3.3.2).

Figure 3-4: MOV1 (A)IN1 sampling station, with manual and tipping rain gauge, (B) IN2 inlet pipe, (C) outlet riser post-retrofit, (D) IN2 sampling station with rainfall-paced sampling, and (E) IN2 inlet pipe.

Figure 3-5: *MOV2 outlet riser structure with HOBO water level logger*.

Flow-proportional, composite samples were taken at each sampling station during storm events using ISCO 6712™ automated samplers to represent the event mean concentration (EMC). Samples at each inlet location were collected in one 10-L composite bottle during storm events. During sample collection, these bottles were agitated by hand to create a homogenous mixture, which was then poured into sample bottles. Outlet samples were collected in 24 1-L bottles during storm events that were shaken and composited in a 24-L bottle. A sub-sample from this composite bottle was then collected for analysis as the representative sample from the storm. Samples were collected within 48 hours of a storm event and transported on ice to the Center of Applied Aquatic Ecology (CAAE) in Raleigh, NC. Samples from all sites were analyzed for total suspended solids (TSS), nitrate-nitrite nitrogen ($NO_{2,3}$-N), total Kjeldahl nitrogen (TKN), total ammoniacal nitrogen (TAN), total phosphorus (TP), and orthophosphate (O-PO_4^{3-}, *Table 3-3*).

Table 3-3: Reported detection limits and analysis methods for pollutants monitored.

Parameter	PQL	Units	Method
TSS	dependent on volume filtered	mg/L	Std Method 2540D
TKN	280	µg/L	EPA Method 351.1
TKN	280	µg/L	EPA Method 351.1
TAN	17.5	µg/L	EPA Method 350-1
$NO_{2,3}$-N	11.2	µg/L	EPA Method 353.2
TP	10	µg/L	EPA Method 365.1
O-PO_4^{3-}	12	µg/L	EPA Method 365.1*

**samples were filtered to 0.45 µm*
PQL: Practical quantitation limit, below which the accuracy of the test is no longer valid

3.3.1.3 MOV1 Retrofit

After an initial 7-month calibration monitoring period, MOV1 was retrofitted to include wetland features. MOV2 remained unchanged to serve as the control. MOV1 was modified in the following ways:

1. The outlet orifice was raised 230 mm using a 100 mm PVC upturned street tee with a threaded cap to create a permanent pool of water covering 60% of the basin (*Figure 3-6B* and *3-7*).
2. A cap was placed on the lower end of upturned elbow, and a 50-mm orifice was drilled in the cap to create 48 hours of detention for the water quality volume (WQV) (*Figure 3-6D* and *3-7*). The cost of all materials required for the PVC installation was approximately $30.
3. Three 300 mm orifices were drilled into the concrete riser structure with inverts 380 mm above the invert of the installed upturned elbow to allow for ponding of water quality events and maintain pre-retrofit peak attenuation rates for the 10-year storm event. (*Figure 3-6A*). Total cost was $900.

4. Approximately 1500 plugs of wetland plants were installed in zones to promote pollutant removal and stabilize the ecosystem. (*Figure 3-6B*). Total cost of plants was $1000.

The outlet orifice was raised by installing a 10-cm PVC upturned elbow on the inside of the outlet structure of MOV1. This elbow included a PVC cleanout cap to facilitate maintenance. The PVC pipe on the wetland side was covered with a 10-cm PVC cap with a 5-cm orifice drilled in it for drawdown control (*Figure 3-6D* and *3-7*). The elbow was secured in place with hydraulic cement.

When the orifice was reduced and elevated, the wetland's ability to mitigate peak flow was somewhat compromised because of increased detention time and decreased storage. To maintain original peak flow attenuation requirements, three 30 cm orifices were drilled with inverts 38 cm above the invert of the water quality orifice. In North Carolina, wetlands are required to have maximum ponding depths of 30 cm for the WQV (NCDEQ, 2017a). Penhall Company was contracted to drill these orifices (*Figure 3-8A*).

Wetland vegetation was selected in accordance with the North Carolina Stormwater Design Manual (NCDEQ, 2017a). Most vegetation was bought from Mellow Marsh Farm in Chatham County, NC, located 45 km WSW of the site. *Pontederia cordata* (pickerelweed) was harvested and transplanted from Smithfield-Selma High School in Johnston County, NC, approximately 59 km southeast of the site. Planting and orifice modification took place in September 2017 (*Figure 3-8B*). A second round of vegetation was installed by BMP East in October 2017.

All retrofits totaled less than $2000. The newly converted wetland was 65% of ideal for a CSW per NCDEQ (2017a). Area loading ratio, the ratio of the watershed area to the CSW area, was 40:1. After initial conversion, the MOV1 wetland was left to stabilize for one month.

Monitoring of the retrofit began on 10/24/2017. Both MOV1 and MOV2 were monitored through July 2018, and post-retrofit performance (*Figure 3-9*) was compared to pre-retrofit performance, discussed in detail in Chapter 2.

Figure 3-6:(A) Original orifice, (B) installed upturned street tee, (C) upturned elbow design before installation, and (D) installed PVC orifice within wetland outlet structure for MOV1.

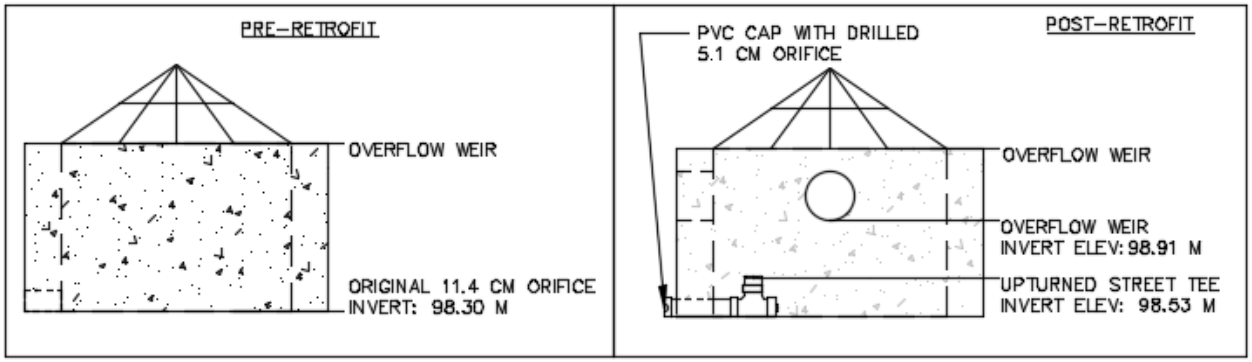

Figure 3-7: MOV1 outlet structure schematic for pre- and post-retrofit conditions. The upturned street tee set the permanent pool elevation in the converted pond.

Figure 3-8: (A) A core drill was used to create peak attenuation orifices in the outlet structure, (B) wetland vegetation, provided by Mellow Marsh Farm, was planted on 0.6 m centers.

Figure 3-9: The newly converted "wetland" as of May 2018.

3.3.1.4: Baseflow Sampling

Baseflow samples were collected in MOV1 during the treatment period to analyze water quality treatment during the inter-event period. Baseflow samples were collected with a minimum antecedent dry period of three days to allow for complete drawdown of the wetland. Samples were taken from MOV1_IN2 and MOV1_OUT. At each sampling station, a 1-L sample bottle was filled directly from the ISCO 6712™ sampler. Before sampling, the ISCO sample tubes were first rinsed to minimize collection of any sediment built up on the sample intake. At each site, 1000 mL was collected in a single sample. The samples were distributed into separate bottles for nitrogen and O-PO_4^{3-} sampling. All samples were transported on ice to the Center for Applied Aquatic Ecology (CAAE).

3.3.2: Winston-Salem Study Design

3.3.2.1: Site Description

The dry pond in Winston-Salem, North Carolina (WS) was monitored at Sherwood Forest Elementary School (*Figure 3-10*). Latitude and longitude coordinates of the school were 36°06’22.50” N and 80°18’33.80” W. A 0.93-ha portion of the property drained to the 0.04 ha dry pond. The majority of this watershed was a playground area at the school, with 0.17 ha of impervious surfaces (paved areas only) and 0.76 ha of landscaped area (*Table 3-1*).The majority of the WS watershed’s soil was Udorthents in hydrologic soil group C (NRCS, 2018).

Figure 3-10: WS is a kidney-shaped dry pond that sits on the eastern edge of Sherwood Forest Elementary.

This dry pond had a 30-cm concrete pipe inlet and an outlet riser structure with a 16-cm orifice as well as two 1.5-m broad-crested weirs for larger storm events (*Figure 3-11*). The length to with ratio of the pond was 1:8, with inlet and outlet located on the short end of the pond (*Figure 3-11)*, which is generally considered poor design (NCDEQa, 2017).

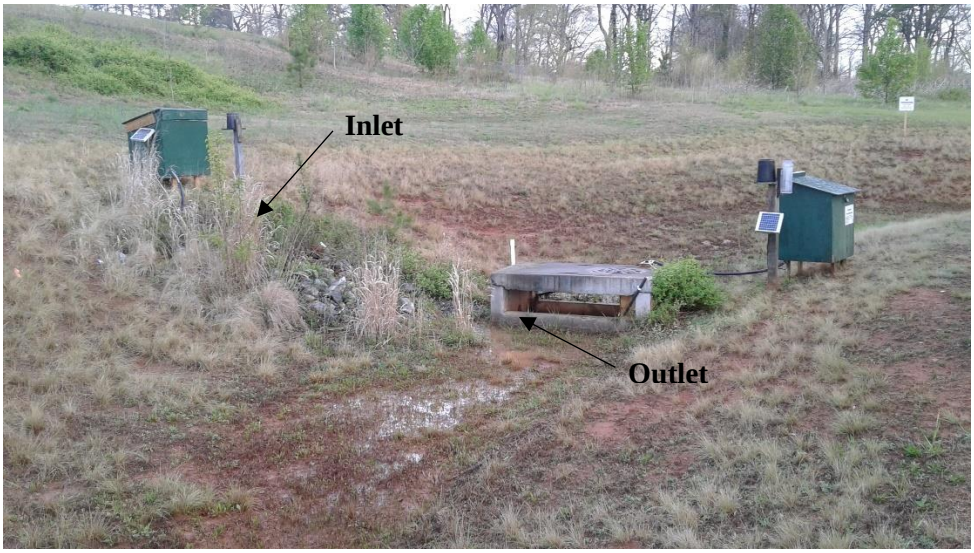

Figure 3-11: Inlet and outlet are located at the shortest part of WS, with a length to width ratio of 1:8.

3.3.2.2: WS Monitoring

WS was monitored utilizing the upstream/downstream design (Spooner et al. 1985, *Table 3-4*). The upstream (inlet) station (WS_IN, *Figure 3-12A*) was used as the control station, and the downstream (outlet) data set (*Figure 3-12*C) was compared to the control. During calibration, a linear relationship was created between the two stations. After the calibration period, changes were made at the downstream station, which altered the relationship between the two stations. Differences in relationships between the two periods were then compared.

Table 3-4: Monitoring set up of sampling locations at WS.

Site	Monitoring Equipment	Trigger to Enable	Sample Pacing*	Collection
WS_IN	ISCO 6712™ water quality sampler 1 HOBO™ tipping bucket rain gauge	rainfall exceeded 10.2 mm in 6 hours	rainfall-paced, every 0.76 mm of rain	24 1-L bottles
WS_OUT	ISCO 6712™ water quality sampler ISCO 730™ bubbler flow module 90° v-notch weir	None	flow-paced, every 0.71 m^3 of runoff	24 1-L bottles

Monitoring occurred between February 2017 and July 2018. Two sampling locations were installed at WS. A manual rain gauge and a HOBO™ tipping bucket rain gauge were located on site, approximately 1.5 m above the ground, clear from any obstructions (*Figure 3-12B*). The outlet (WS_OUT) was equipped with an ISCO 6712™ automated sampler and an ISCO 730™ bubbler module. A 90° v-notch weir was used to measure outflow (*Figure 3-12D*). An ISCO 6712™ connected to a tipping bucket rain gauge was installed at WS_IN and was used to take rainfall proportional samples as a surrogate for flow proportional sampling. Flow proportional (or rainfall proportional) composite samples were collected at both the inlet and outlet of the pond. There were 24 1-L bottles at each sampling station that were shaken and composited into one 24-L bottle before being transferred to the final sample bottle. These samples were retrieved within 48 hours of a storm event and transported on ice to CAAE to be analyzed for TSS, $NO_{2,3}$-N, TKN, TAN, TP, and O-PO_4^{3-} (*Table 3-3*).

Figure 3-12: WS (A) inlet pipe, (B) ISCO tipping bucket rain gauge and sampling box at WS_IN, (C) outlet structure with HOBO water level logger and (D) installed weir in outlet structure with bubbler tube for level measurements.

3.3.2.3: WS Retrofit

After calibration monitoring, WS was converted to a CSW on 3/23/2018, modified as follows:

1. The 16-cm orifice was blocked using hydraulic cement (*Figure 3-13*), costing $20.
2. Approximately 500 wetland plants were installed at surface elevations up to 23 cm above the overflow weirs (a level that was set as the temporary inundation zone as detention above the overflow weirs was limited; *Figure 3-14*), costing $450.

Wetland plants were purchased from Mellow Marsh Farms located 93 km ESE of the site and planted on the same day. *Pontederia cordata* (pickerelweed) was transplanted from Smithfield-Selma High School (*Figure 3-14*) located 190 km ESE of the site. Total retrofit costs were less than $500. Plants were installed on 3/23/2018, but a snow storm and freezing weather conditions on 3/24/2018 may have inhibited survivability of many of the plant plugs. Some of the pickerel weed planted appeared to survive. However, existing (pre-retrofit) vegetation persisted throughout the summer.

Additionally, ponded water from storm events was observed to infiltrate or evaporate within 48 hours. The riser structure only allowed for up to 15 cm of ponded water in the basin, after which detention of water was minimal.

Figure 3-13: The principal orifice in WS (left) was blocked to allow for a ponding zone (right).

Figure 3-14: Several species of wetland plants were installed according to elevation in WS.

3.3.3: Hydrologic Data Analysis

3.3.3.1: Precipitation

Any storm with precipitation depth greater than 2.5 mm and antecedent dry period no less than 6 hours was characterized as a discrete rainfall event (Driscoll et al., 1989). Rainfall depths from tipping bucket rain gauges were adjusted by a scaling factor using the following equation:

$$P_a = \frac{D_a}{D_m} * P_m \qquad \text{(Eq. 3.1)}$$

where

P_a: corrected depth of rainfall per interval (mm)

P_m: measured depth of rainfall per interval (mm)

D_a: manual rain gauge depth measured over sampling period (mm)

D_m: tipping rain gauge depth measured over sampling period (mm)

3.3.3.2: Water Balance

Outflow volumes were calculated and recorded using an ISCO 730™ bubbler module. Details for weir and orifice equations to calculate flow are detailed in this section. Exact inflow volumes from MOV1 and WS were not recorded. However, a water balance was created to calculate inflow and outflow volumes of the dry ponds (Eq. 3.2). Possible sources of input include precipitation, inflow, and groundwater inflow (Eq. 3.3). Possible outputs include evapotranspiration, infiltration, and runoff. The water balance is as follows:

$$V_{in} = V_{out}, \text{ or} \qquad \text{(Eq. 3.2)}$$

$$P + R_{in} + G_{in} = ET + G_{out} + R_{out} \qquad \text{(Eq. 3.3)}$$

where

R_{in}: inflow

P: precipitation

G_{in}: groundwater inflow

ET: evapotranspiration

Rout: outflow

G_{out}: infiltration (groundwater out)

Outflow was calculated directly at all sites, and inflow was calculated using Equation 3.3. Direct inflow from the banks near the basins that did not enter through the inlets were not included in influent water quality measurements. In the following section, each factor of the equation will be discussed.

Morrisville (MOV1 and MOV2):

Groundwater inflow: Per visual observations, it was assumed that the groundwater seepage into the basin at this site was negligible. Baseflow in MOV1, which was concentrated to one inlet, was considered inflow.

Infiltration: As mentioned in Section 4.3.1, the majority of soils in the MOV1 watershed are D soils, which are associated with low infiltration rates. However, infiltration tests were performed to account for water loss from infiltration. A modified Philip-Dunne (MPD) infiltrometer from Upstream Technologies™ was used to calculate infiltration rates in the ponds. Infiltration rates of MOV1 were found to be negligible. Details of these tests can be found in *Appendix A*.

Evapotranspiration: In Morrisville, ET rates were calculated using a reference crop evapotranspiration from the online North Carolina Climate Office server (NCCO, 2018), using the KRDU site located at Raleigh-Durham International Airport, located 5.9 km from the site. Data was reported as daily ET. For the five storms with the longest duration in MOV1 in the calibration phase, ET as a percentage of total outflow volume was calculated. This was based on stage-storage information. In all cases, expected ET was found to be 0.31% or less of total volume. ET was considered negligible and was not included in the water balance. Details of this analysis are in *Appendix A*.

Precipitation: Volume of precipitation can be calculated by multiplying the depth of rainfall during a storm by the area on which it fell. In this case, the area of the dry pond was the only area of the watershed receiving rainfall that had not been routed through the culverts as inflow.

Based on this information, the water balance was reduced to:

$$P + R_{in} = R_{out} \qquad \text{(Eq. 3.4)}$$

where inflow was calculated from measured precipitation and known outflow volumes.

Outflow: Each monitored dry pond has an outlet structure through which effluent flows. Outlet structures in both MOV1 and MOV2 were designed with an orifice and a broad-crested weir to pass larger flows. These outlets were all equipped with an ISCO bubbler module, which read the water level near the outlet every two minutes during storm events. This level was then used to calculate flowrate, and subsequently volume, by utilizing the orifice equation (Eq. 3.5 and 3.6) and the weir equation (Eq. 3.7).

When an orifice was not completely submerged, the following equation was used (Malcom 1989):

$$Q = 4.464 C_d D^{2.5} \qquad \text{(Eq 3.5)}$$

where

Q: discharge (cfs)

C_d: discharge coefficient, 0.6

D: orifice diameter (ft)

When the orifice was submerged, the orifice equation may be used (Malcom 1989):

$$Q = C_d A \sqrt{2gh} \qquad \text{(Eq 3.6)}$$

where

Q: discharge (m^3/s)

C_d: discharge coefficient, 0.6

A: orifice area (m^2)

g: gravitational acceleration (m/s^2)

h: depth of orifice center from water surface (m)

A broad-crested weir was a component of both riser structures (*Figure 3-4C, 3-5, and 3-7*). The equation is as follows:

$$Q = C_d L h^{3/2} \quad \text{(Eq 3.7)}$$

where

C_d: 1.49 for a broad-crested weir

L: length of weir (m)

h: height above the weir (m)

By subtracting precipitation volume from calculated outflow volume, total inflow volumes could be calculated (Eq. 3.4). In MOV1, inflow was divided between two inlets, with watersheds of unequal sizes. Both subwatersheds were characterized by percent impervious cover. Using the Simple Method (Schueler, 1987), the proportion of the total inflow volume contributed by each subwatershed was calculated (Eq 3.8 – 3.14).

$$R_V = 0.05 + 0.9*I_A \quad \text{(Eq. 3.8)}$$

R_V: runoff coefficient

I_A: impervious fraction

and

$$Q_v = 10*R_D*R_V*A \quad \text{(Eq. 3.9)}$$

Q_v: inflow volume (m^3)

R_D: rainfall depth (mm)

A: drainage area (ha)

The entire inflow volume (Q) from the watershed of MOV1 shall be

$$Q_v = Q_{1,1} + Q_{1,2} \quad \text{(Eq. 3.10)}$$

where

$Q_{1,1}$: volume of inflow from the watershed of MOV1_IN1

Q1,2: volume of inflow from the watershed of MOV1_IN2

The proportion of inflow from MOV1_IN1 was

$$\frac{Q_{1,1}}{Q_{1,1}+Q_{1,2}} \quad \text{(Eq. 3.11)}$$

or

$$\frac{10*R_D*R_{V1,1}*A_{1,1}}{(10*R_D*R_{V1,1}*A_{1,1})+(10*R_D*R_{V1,2}*A_{1,2})} \quad \text{(Eq. 3.12)}$$

Reducing this formula leaves

$$\frac{R_{V1,1}*A_{1,1}}{(R_{V1,1}*A_{1,1})+(R_{V1,2}*A_{1,2})} \quad \text{(Eq. 3.13)}$$

where proportion of inflow is a function of the runoff coefficient and watershed area. These values are reported in *Table 3-5*.

Table 3-5: Characteristics of subwatersheds that contribute inflow through IN1 and IN2 at MOV1 using the Simple Method.

Rational Method for MOV1		
	MOV1_IN1	**MOV1_IN2**
I_A	0.34	0.38
A (ha)	1.27	2.68
R_V	0.35	0.39
R_VA (ha)	0.45	1.04
% of inflow	30%	70%

3.3.3.4: Winston-Salem:

The WS watershed was only 18% impervious and produced little outflow for storms less than 13 mm. From field observations and measured infiltration rates using the Upstream Technologies™ MPD Infiltrometer, infiltration could not be considered negligible (*Appendix A*). Therefore, total inflow volumes in WS were calculated using a different methodology than that used for MOV sites. (Eq. 3.15).

$$Vin = ET + G_{out} + R_{out} \quad \text{(Eq. 3.15)}$$

To estimate total volume loss due to infiltration and ET, data was collected from a HOBO water level logger installed near the former outlet orifice, which recorded temperatures and absolute pressures at the pond outlet. A second HOBO logger was placed in one of the sample boxes to record atmospheric pressure; depth of water above the sensor (water levels) were determined by the difference in pressure between the two loggers. During the treatment phase, up to 150 mm of water, the height of the overflow weirs, was ponded within the basin during storm events before outflow commenced. Drawdown rates from 150 mm of ponding were estimated for the first five storms during the treatment phase (*Table 3-6*). Average drawdown rate was applied to all storms for which there was outflow during the calibration phase to calculate losses from ET and infiltration (ET&I). Drawdown rate in WS was assumed to include both infiltration and ET. It should be noted that this rate may have varied slightly on a seasonal basis.

Volumes were estimated for individual storms based on data recorded from the HOBO water level loggers and a stage-storage survey of the pond. For each storm, drawdown was estimated by calculating the duration of drawdown of a known volume (up to berm crest) after inflow had ceased (water level no longer increased). This drawdown rate was applied to the entirety of the event to calculate total volume lost to ET&I.

Table 3-6: Drawdown rate of WS for storms used to calculate volume reduction pre-retrofit.

Date	**Starting Elevation** *mm*	**Duration** *h:m*	**Drawdown Rate** *mm/hr*
3/25/2018	153	36:46	2.46
4/7/2018	97	20:42	4.45
4/16/2018	123	25:58	3.64
4/25/2018	155	32:24	2.36
4/26/2018	177	49:54	2.32
		Mean	3.05
		Std. Error	0.96

Outflow was calculated using a 90° v-notch weir installed inside the riser structure of WS and an ISCO 760™ bubbler module (Eq. 3.16). The equation for a 90° v-notch weir is (Malcom 1989):

$$Q = C_d h^{5/2} \qquad \text{(Eq. 3.17)}$$

where

C_d: 1469 (constant)

h: height of water above the weir (m)

Total inflow volumes were estimated by adding calculated ET&I to total measured outflow volumes.

3.3.4 Water Quality Data Analysis

Concentration and load estimations were made utilizing collected concentration and volume data. Concentrations from MOV1_IN1 and MOV1_IN2 were compiled using methodology from *Table 3-5* in Section 3.3.3. Representative EMCs for MOV_IN were calculated as follows:

$$C_{1,in} = 0.3 * C_{1,1} + 0.7 * C_{1,2} \qquad \text{(Eq. 3.18)}$$

where

$C_{1,in}$ = composite MOV_IN EMC (mg/L)

Concentration removal efficiencies were estimated during each phase of monitoring with the following equation:

$$RE_c = \frac{C_i - C_o}{C_i} * 100 \qquad \text{(Eq. 3.19)}$$

where

C_i: influent concentration (mg/L)

C_o: effluent concentration (mg/L)

RE_c: removal efficiency (%)

RE_c does not account for changes in water volumes between sampling stations due to rainfall, ET, or infiltration. Pollutant removal is also analyzed by total mass removed. To determine these rates, influent and effluent loads must be calculated (Eq. 3.20 – 3.21).

$$EL_{in_i} = C_{in_i} * V_{in_i} \quad \text{(Eq. 3.20)}$$

$$EL_{out_i} = C_{out_i} * V_{out_i} \quad \text{(Eq. 3.21)}$$

where

EL: event load (g)

V: volume (m^3)

In WS, based on the method for V_{in}, measured influent concentration was attributed to the entire inflow volume. In MOV1, any difference in inflow and outflow volumes was attributed to precipitation. Assuming concentrations of pollutants in rainfall were equal to inflow concentrations would likely be an overestimate of total inflow loads. Typically, measured concentrations of TSS, TP, O-PO_4^{3-}, and organic nitrogen (ON) in rainfall are much lower than the influent concentrations recorded in this study (Anderson and Downing, 2006; Wu et al., 1998), and were considered negligible. However, dry and wet deposition of nitrogen can be a large contributor to loads of TAN and $NO_{2,3}$-N (Line et al., 2002; Wu et al., 1998). At the Morrisville sites, rainfall was a large percentage of incoming water volume, ranging from 8 – 19% of total volume in MOV1 during the calibration phase. Therefore, comparing influent and effluent nitrogen loads in these ponds without considering loads contributed from direct deposition would underestimate pollutant removal performance in the pond. To quantify the portion of nitrogen loads that were attributed to direct deposition, precipitation sampling stations

were set up at the Morrisville sites after the initial monitoring period. The addition of nitrogen concentrations in rainwater from deposition creates a more accurate inflow load estimation.

During the study, bulk deposition was measured along with water quality from storm events beginning on 5/14/2018. Oil pans were installed in each MOV1 and MOV2 at ground level to collect bulk deposition of nitrogen (*Figure 3-15*). Due to contamination issues from surface pollutants like grass and insects, the oil pans were raised 0.3 m off the ground (5/23/20118) and covered with a nylon wire mesh to prevent contamination from large particles (6/12/2018). Within 24 hours of the end of storm events, direct deposition samples from each pond were composited into one acidified bottle and transported on ice to CAAE to be analyzed within 48 hours. Oil pans were cleaned in the field with deionized water and replaced for the next event.

Figure 3-15: Oil pans were used to collect water quality samples direct rainfall on the basins in Morrisville.

Median concentration of nitrogen forms in rainwater was combined over all storm events and used as a representative concentration. Total influent loads for MOV1 and MOV2 were combined as follows:

$$EL_{in} = C_P AP + C_{in} V_{in} \quad \text{(Eq. 3.22)}$$

where

ELin: total influent pollutant load (g)

C_p: concentration from direct rainfall (mg/L)

A: area of the dry pond (ha)

P: precipitation (mm)

Cin: concentration from the dry pond inlet (mg/L)

Vin: inflow volume (m^3)

Annual event loading can also be determined by summing total influent and effluent loads over the monitoring periods. These loads are then normalized for total storm flow from hydrologic events throughout the monitoring period and also normalized for 30-year normal rainfall (Eq. 3.23, NOAA, 2018).

$$Annual\ Event\ Load\ Export = \frac{\sum EL_o * \frac{\sum V_{out,total}}{\sum V_{out,WQ}}}{1000 * DA} * \frac{P_{annual}}{P_{observed}} \quad \text{(Eq. 3.23)}$$

where

EL_O: Event load out (g)

DA: Drainage area (ha)

$\sum V_{out,total}$: Sum of outflow volumes from storm events for 8-month period (m^3)

$\sum V_{out,WQ}$: Sum of outflow volumes from events monitored for water quality for 8-month period (m^3)

P_{annual}: Normal 12-month precipitation for local area (mm)

$P_{observed}$: Observed rainfall during 8-month monitoring period (mm)

In MOV1, there was a constant baseflow through the pond year-round. This made it challenging to separate stormflow and baseflow. For consistency, stormwater runoff volume was included in calculations when the level at the outlet orifice exceeded 0.046 m elevation pre-retrofit and 0.015 m post-retrofit. These levels were chosen because it was observed that baseflow never exceeded these heights. Outflow samples were taken when water levels exceeded these heights to avoid erroneous sampling.

3.3.5: Statistical Analysis

Effects of the MOV1 retrofit were determined using both the paired watershed design and the upstream/downstream design (Clausen and Spooner, 1993; Spooner et al., 1985; Spooner and Harcum, 2014). A multiple linear regression was run between control and treatment sampling stations to determine if significant correlations existed between variables (predictor variables are MOV1_IN and MOV2_OUT and MOV1_OUT as the response) (Eq. 4.24). If so, variables with significant relationships were used as covariates (*Appendix H*). In MOV1, the general model for this regression equation is as follows:

$$\hat{y} = \beta_o + \beta_1 x_1 + \beta_2 x_2 + \beta_{1,2} x_1 x_2 \qquad \text{(Eq. 3.24)}$$

Where

$\hat{y}$: expected concentration/load of MOV1_OUT

x1: MOV1 upstream concentration/load

x2: MOV2_OUT concentration/load

βo: intercept

β1: partial regression coefficient of MOV1 upstream

β2: partial regression coefficient of MOV2

β1,2: interaction between MOV1 upstream and MOV2

The upstream/downstream design required a test of the simple linear regression of WS_IN and WS_OUT (Eq. 4.25, Spooner et al., 1985).

$$\hat{y}_{ws} = \beta_o + \beta_1 x_1 \qquad \text{(Eq. 3.25)}$$

where

$\hat{y}_{ws}$: expected concentration/load of WS_OUT

x_1: WS_IN concentration/load

βo: intercept

β1: regression coefficient of WS_IN

An analysis of covariance (ANCOVA) was run on determined models to detect differences between the calibration and treatment periods. Full models were run initially with interaction effects between the treatment period and covariates. Models were analyzed for significantly different intercepts and slopes between periods. Those models for which interaction coefficients were not significant were reduced and retested. Significance level was considered as $\alpha = 0.05$. Residuals were visually inspected for normality, constant variance, and autocorrelation (*Appendix H*). These tests were utilized for both concentration and load comparison in MOV1 and WS.

Least squares means (LSM) were also calculated using R software. LSMs were used over means to correct for unbalanced data (different sample sizes between monitoring periods). Percent reductions in pollutant export between periods was calculated using the following equation (Eq. 4.26):

$$\%\, Reduction = \left(1 - \frac{\bar{Y}_T}{\bar{Y}_C}\right) * 100 \qquad \text{(Eq. 3.26)}$$

where

$\bar{Y}_T$: response variable LSM during Treatment phase

$\bar{Y}_C$: response variable LSM during Calibration phase

When no variables were appropriate covariates for the effluent water quality, ANCOVA was not performed. Instead, Student's t-tests were used to detect differences in effluent water quality between treatment periods.

3.4: Results and Discussion

3.4.1: Hydrology

The study can be divided into three periods – calibration, installation, and treatment (*Table 3-7*). In Morrisville, the treatment period began one month after modification allow for site stabilization in MOV1. Because plants were installed in September, they remained dormant until the spring growing season. Ideally, each monitoring period would have continued for a year, but both time and funding limited study duration. However, there were sufficient storms sampled in both periods to establish relationships needed for the ANCOVA model. Due to time constraints, treatment period monitoring began immediately after retrofit installation at WS.

Table 3-7: Monitoring period phases of the dry pond retrofit study.

	MOV	**WS**
Calibration Monitoring	1/27/17 - 9/11/17	2/5/17 - 3/22/18
Retrofit Installation	9/21/17 - 10/5/17	3/23/2018
Treatment Monitoring	10/23/17 - 8/1/18	3/24/18 – 4/30/2019

In Morrisville, there were eight paired water quality events during the calibration phase and sixteen during the treatment phase. Precipitation during the monitoring periods is summarized in *Table 3-8*. The distribution of rainfall depth for water quality events was similar for both periods in MOV1 and WS. Storms within a different depth ranges were analyzed for this

assessment. Only four storms were monitored for water quality in WS in the treatment period, which limits analysis. Monitoring is ongoing at this site. The results reported herein are preliminary and may not reflect the final analysis of this retrofit.

Table 3-8: Precipitation characterization during both phases of monitoring at MOV1 and WS, where n = number of storms monitored (all depths in mm).

Period		n	Range	<12.7	12.7<x<25.4	>25.4	Mean	Median	Total
MOV1									
Calibration	Hydrology	39	2.4 - 47.8*	25 (64%)	8 (21%)	6 (15%)	14.0*	8.6*	546.1
	WQ	8	7.9 - 47.8	1 (13%)	5 (63%)	2 (25%)	23.1	22.9	184.9
Treatment	Hydrology	45	2.4 - 93.5	20 (44%)	15 (33%)	10 (22%)	18.8	14.0	844.0
	WQ	16	7.4 - 37.6	2 (25%)	9 (56%)	5 (31%)	21.1	20.6	335.8
WS									
Calibration	Hydrology	56	2.4 - 123.7*	29 (52%)	17 (30%)	10 (18%)	15.7*	11.7*	1032.3
	WQ	10	13.7 – 47.5	0	5 (50%)	5 (50%)	27.8	26.0	278.1
Treatment	Hydrology	25	2.5 - 64.0	15 (60%)	6 (24%)	4 (16%)	13.9	9.4	362.5
	WQ	4	1.18 - 2.52	0	0	4 (100%)	40.6	35.6	162.3

**Largest storms at both sites in April 2017 were excluded due to equipment failure*

Peak rainfall intensity can affect pollutant concentrations and runoff flow rates in SCMs (Deletic, 1998). Five-minute mean and median peak intensities were comparable between phases at both monitoring sites (*Table 3-9*).

Table 3-9: 5-min peak rainfall intensity during both phases of monitoring at MOV1 and WS (all rates in mm/hr).

Dry Pond	Period	n	Range	Mean	Median
MOV1	Calibration	39	3.1 - 123.8	37.2	25.9
	Treatment	45	3.7 - 289.7	32.0	16.7
	Change			-14%	-36%
WS	Calibration	53	4.3 - 192.4	29.5	16.7
	Treatment	25	3.2 - 217.3	30.4	19.4
	Change*			3%	16%

**Negative sign = reduction*

After the retrofit, drawdown times in MOV1 increased due to reduced orifice diameter. As a result, baseflow became a larger proportion of total outflow volumes in MOV1; consequently, outflow volumes increased in the treatment period (*Table 3-10*). Therefore, total effluent volumes between monitoring periods in MOV1 were not directly compared.

Capping the principal orifice in WS created 4.7 m^3 of storage during storm events; all captured water infiltrated within 48 hours. Of storms that produced outflow during the calibration period, 21% produced less than 4.7 m^3. Median runoff volume of storms with outflow was 20.5 m^3, which is four times greater than available storage (*Table 3-10*). This suggests that total outflow may have been slightly reduced from the retrofit, though there is not sufficient treatment period data to verify this statistically.

Table 3-10: Inflow and outflow volume characterization during storm events at MOV1 and WS for both phases of monitoring (all volumes in m^3).

Period		Range	Mean	Median	Total
MOV1					
Calibration	Inflow	3 - 4337	304	113	12166
	Outflow	8 - 4630	334	126	13357
Treatment	Inflow	0 - 3234	372	234	16730
	Outflow	5 - 3387	403	263	18117
WS					
Calibration	Inflow	0 - 493	26.4	0.74	1481
	Outflow	0 - 488	25.3	0.15	1418
Treatment	Inflow	0 - 84	9.2	0	239
	Outflow	0 - 74	7.2	0	187

In Morrisville, the peak attenuation orifices (*Figure 3-8*) were designed to match the original outlet's peak discharge rates for a 10-yr, 24-hr storm (*Appendix F*). The elevation of these orifices and the reduced size of the water quality orifice reduced storage capacity for water quality treatment and peak flow attenuation, and caused increased peak outflows for medium-sized storm events (*Table 3-11*). Because MOV1 was only sized to capture 65% of the water quality (25-mm) event, a portion of runoff volume bypassed treatment in medium storms.

In WS, the principal orifice was covered in the retrofit, which could have led to increased peak flows and violate principal design requirements, though it has not yet. Based on analysis

using Hydraflow from AutoCAD Civil 3D, 10-year, 24-hour peak flows increased by 56% due to the retrofit (*Appendix F*).

Table 3-11: Calibration and treatment phase peak flow characteristics for retrofitted ponds at MOV1 and WS(reported in L/s).

Dry Pond	Period	n	Range	Mean	Median
MOV1	Calibration	40	1 - 260	22.7	13.8
	Treatment	45	1 - 295	26.6	2.81
	Change*			17%	-80%
WS	Calibration	54	0 - 54.8	4.8	0
	Treatment	25	0 -17.4	2.45	0
	Change			-49%	0%

**Negative sign = reduction*

Design regulations of CSWs limit ponding depth to a maximum of 0.38 m (NCDEQ, 2017a). Hence, CSWs have a larger footprint than dry ponds designed for the same WQV. These designed-as dry ponds were retrofitted to meet CSW design criteria to the extent possible. In MOV1, the ponding depth limit allowed for storage of only 65% of runoff produced by a 25 mm storm event (WQV). In WS, the available ponding storage was only 5% of the WQV. Both dry ponds became undersized by CSW standards upon conversion (*Table 3-12*).

Table 3-12: Watershed surface area to SCM surface area ratio (loading ratio) and percentage of the WQV that can be treated (storage capacity) at the converted MOV1 and WS basins.

	MOV1	WS
Loading Ratio	40:1	113:1
Storage Capacity	65%	5%

3.4.2: Water Quality Analysis

3.4.2.1: MOV

During sampling, there were issues with a buildup of sediment on the MOV1_OUT sample intake, leading to an overestimation of TSS concentrations in some events. To minimize this issue, storm events with TSS concentrations that exceeded Cook's distance for outliers were

excluded from analysis (Kim and Storer, 1996). Using this method, TSS concentrations from two storm events during the treatment phase were omitted from analysis.

Median EMCs for all sampling locations at the MOV site were compared between monitoring periods (*Table 3-13*). Direct EMC comparison shows reduction of TSS, TKN, TN, ON, TP, and O-PO_4^{3-} from the calibration to the treatment phase. Mean REc is also reported for all analytes. However, this simple comparison does not account for variability among storms, seasons, or pollutant concentrations.

Table 3-13: Median event mean concentrations at all MOV sampling locations and concentration removal efficiencies (RE_c) during both monitoring phases.

Period	**Location**	**TSS**	**TKN**	**$NO_{2,3}$-N**	**TAN**	**TN**	**ON**	**TP**	**O-PO_4^{3-}**
		mg/L	*mg/L*	*mg/L*	*mg/L*	*mg/L*	*mg/L*	*mg/L*	*mg/L*
Calibration n = 8	MOV2_OUT	18	1.26	0.85	0.11	2.12	1.15	0.38	0.31
	MOV1_IN1	39	1.13	0.61	0.18	1.68	0.96	0.20	0.11
	MOV1_IN2	57	1.86	0.68	0.26	2.57	1.47	0.32	0.15
	MOV1_OUT	67	1.92	0.61	0.17	2.69	1.75	0.41	0.22
	MOV1 RE_c	-45.2%	-28.8%	-6.2%	23.9%	-27.2%	-45.0%	-38.9%	-48.8%
	p-value***	0.544[a]	0.235[c]	0.671[a]	0.823[a]	0.505[c]	0.195[c]	0.442[c]	0.382[c]
Treatment n = 16	MOV2_OUT**	17*	1.66	1.24	0.13	2.84	1.41	0.4	0.29
	MOV1_IN1	22*	1.46	0.65	0.22	2.07	1.14	0.25	0.10
	MOV1_IN2	72*	1.75	0.78	0.24	2.66	1.5	0.4	0.15
	MOV1_OUT	26*	1.23	0.65	0.14	1.90	1.08	0.23	0.12
	MOV1 RE_c	58.9%	28.0%	30.0%	39.2%	24.5%	21.8%	22.8%	22.9%
	p-value***	**0.005**[a]	0.051[c]	0.059[a]	**0.001**[c]	**0.001**[c]	0.083[c]	0.062[b]	0.318[a]

**n = 14 to minimize issues with sediment buildup on the MOV1_OUT sample intake*
***MOV2_OUT remained unchanged as the control*
****bold values are significant*
*****REc = $(C_{in} - C_{out})/C_{in}$*
[a]*p-value calculated with Student's t-test*
[b]*p-value calculated with log transform of Student's t-test*
[c]*p-value calculated with the Wilcoxon signed rank test*

ANCOVA was used to account for variability among samples where applicable. There were correlations between TSS concentrations in MOV1_OUT and MOV1_IN or MOV2_OUT, preventing the use of ANCOVA. However, ANCOVA was applied to all other parameters and showed significant reductions in outlet EMCs for all pollutants except $NO_{2,3}$-N (*Figure 3-16*). An interaction effect was present in both TKN and ON analysis. Least squares mean (LSM)

differences between monitoring periods were 63%, 76%, 78%, and 77% for TP, TN, TKN, and ON, respectively. In analysis of O-PO_4^{3-} and TAN EMCs, linear relationships existed between MOV1_OUT and both MOV1_IN and MOV2_OUT. The regression models for these parameters reflect these relationships (*Table 3-14*). LSM differences between monitoring periods were 63.2% for O-PO_4^{3-} and 73.0% for TAN.

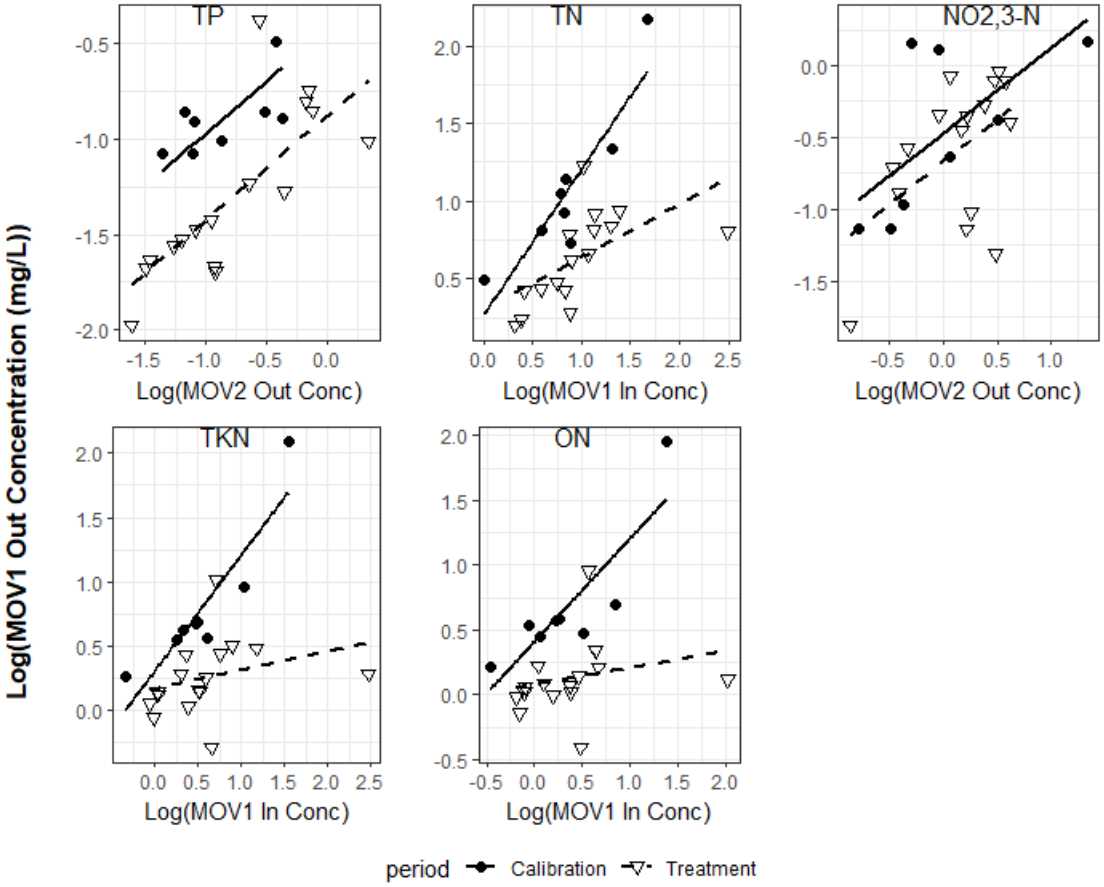

Figure 3-16: ANCOVA for log of MOV1 concentrations for TP, TN, $NO_{2,3}$-N, TKN, and ON with respect to indicated covariate.

Load estimations were affected by total inflow and outflow volumes. Due to baseflow and increased drawdown times within MOV1, storm inflow and outflow volume calculations significantly increased in the treatment phase. Increased volumes led to increased total load estimates, making direct comparisons between monitoring periods difficult. However, ANCOVA and LSM analysis adjusted for variability in influent and control pond concentrations between

periods effectively compare effluent concentrations. For all analytes except $NO_{2,3}$-N, significant decreases were detected, with LSM reductions ranging from 60 – 89% of annual loads (*Table 3-14*). A significant linear relationship detected between effluent TSS loads at the MOV1 and MOV2 outlets allowed for ANCOVA in TSS load comparisons . TSS effluent load reduction was the greatest among all analytes.

Table 3-14: MOV1 influent and effluent loads during both phases of monitoring periods with annual loading and event load reductions. The difference between MOV1 effluent loads were detected using LSM.

Pollutant	**Period**	**Location**	**Cumulative Loads** *g*	**Annual Loading** *kg/ha/yr*	**Event Load Reduction**	**Pre-Post LSMs**
TSS	Pre	Inflow	78618	106	4%	**88.5%**
		Outflow	75668	102		
	Post	Inflow	191823	208	64%	
		Outflow	68790	75		
TP	Pre	Inflow	1386	1.87	6%	**60.0%**
		Outflow	1301	1.76		
	Post	Inflow	2597	2.29	18%	
		Outflow	2118	1.86		
$O\text{-}PO_4^{3-}$	Pre	Inflow	968	1.31	25%	**61.7%**
		Outflow	728	0.98		
	Post	Inflow	1136	1.06	16%	
		Outflow	952	0.89		
TN	Pre	Inflow	6784	9.15	-14%	**70.9%**
		Outflow	7743	10.5		
	Post	Inflow	19261	17.0	27%	
		Outflow	13966	12.3		
TAN	Pre	Inflow	1037	1.40	31%	**68.9%**
		Outflow	710	0.96		
	Post	Inflow	3764	3.31	66%	
		Outflow	1266	1.11		
TKN	Pre	Inflow	4962	6.69	-16%	**74.5%**
		Outflow	5751	7.76		
	Post	Inflow	14700	12.9	35%	
		Outflow	9538	8.39		
ON	Pre	Inflow	3913	5.28	-29%	**74.7%**
		Outflow	5029	6.79		
	Post	Inflow	10915	9.61	24%	
		Outflow	8250	7.26		
$NO_{2,3}\text{-}N$	Pre	Inflow	1823	2.46	-9%	39.3%
		Outflow	1992	2.69		
	Post	Inflow	4561	4.01	3%	
		Outflow	4428	3.90		

**bold values are significant to a level of α = 0.05*

Regression equations for ANCOVA were developed for all but one parameter in MOV1, for both concentration and load data sets (*Table 3-15*). All equations were developed on a log-log scale. T and p-values are also reported.

Table 3-15: Regression equations of MOV1 defining the relationship between the covariate(s) and MOV1 effluent EMC/load during both monitoring periods for all pollutants using ANCOVA.

	Pollutant	Regression Equations		ANCOVA		
				Intercept		LSM
		Calibration Period (n = 8)	Treatment Period (n = 16)	t	p-value	Difference**
Concentration mg/L	TSS*	LogR = 0.0682LogUS + 3.410	LogR = 0.3307LogUS + 1.9311	3.41	0.0047	**58.9%**
	TP	LogR = 0.5508LogC - 0.4222	LogR = 0.5508LogC - 0.8764	4.21	0.0004	**64.9%**
	$O\text{-}PO_4^{3-}$	LogR = 0.7706LogC + 0.2666LogUS -0.23095	LogR = 0.7706LogC + 0.2666LogUS - 0.73469	4.59	0.0002	**68.6%**
	TN	LogR = 0.9414LogUS + 0.265	LogR = 0.9414LogUS + 0.2995	5.46	<0.0001	**75.7%**
	$NO_{2,3}$-N	LogR= 0.5929LogC - 0.4175	LogR= 0.5929LogC - 0.6163	1.03	0.3141	36.7%
	TKN	LogR= 0.9002LogUS + 0.3083	LogR= 0.1447LogUS + 0.1683	5.60	<0.0001	**81.8%**
	TAN	LogR = 0.2754LogC + 0.5053LogUS -0.3821	LogR = 0.2754LogC + 0.5053LogUS - 0.9161	2.29	0.0328	**70.8%**
	ON	LogR = 0.7982 LogUS + 0.411	LogR = 0.1278 LogUS + 0.079	5.21	<0.0001	**79.8%**
Loads kg/ha/yr	TSS	LogR = 0.5062LogC + 6.2755	LogR = 0.5062LogC + 5.3356	4.02	0.0007	**88.5%**
	TP	LogR = 0.7525LogC + 2.0923	LogR = 0.7525LogC + 1.6940	2.82	0.0103	**60.0%**
	$O\text{-}PO_4^{3-}$	LogR = 1.080LogC + 0.543LogUS - 0.105LogC*LogUS -0.15533	LogR = 1.080LogC + 0.543LogUS - 0.105LogC*LogUS - 0.51651	2.98	0.0080	**56.5%**
	TN	LogR = 0.7857LogUS + 1.727	LogR = 0.7857LogUS + 1.1904	3.86	0.0009	**70.9%**
	$NO_{2,3}$-N	LogR = 0.3272LogC + 0.7197LogUS + 0.0593	LogR = 0.3272LogC + 0.7197LogUS - 0.15742	2.48	0.2501	39.3%
	TKN	LogR = 0.6525LogUS + 2.540	LogR = 0.6525LogUS + 1.946	3.39	0.0028	**74.5%**
	TAN	LogR = 0.2944LogC + 0.5480LogUS + 1.0098	LogR = 0.2944LogC + 0.5480LogUS 0.50211	2.26	0.0353	**68.9%**
	ON	LogR = 0.6639LogUS + 2.492	LogR = 0.6639LogUS + 1.8948	3.483	0.0022	**74.7%**

C = MOV2 Outlet (control pond), US = MOV1 inlet, R = MOV1 Outlet
**neither MOV2_OUT nor MOV1_IN were significant covariates to predict MOV1_OUT. A paired Student's t-test was used to compare influent and effluent concentrations of MOV1 in the treatment period. Median RE_c is reported.*
***bold values indicate significance at a level of $\alpha = 0.05$*

Retrofit implementation altered the trajectory of MOV1 cumulative loads. For example, TSS effluent loads had been higher than those of the inflow at the end of the calibration period. By January 2018 of the treatment period, the inverse was true (Figure 3-17). Similar trends were seen for TP, TN, and TKN. Differences in influent and effluent pollutant loads were most apparent for TAN; cumulative effluent loads were less than half of those of influent at the end of

the treatment period. Thus, mechanisms (plant uptake and nitrification) to remove soluble nitrogen were present. Visual observations of increased total mass of live plant material post-retrofit suggested the potential for plant uptake. TAN may also have been converted to $NO_{2,3}$-N through nitrification, which can occur when turbulent inflows increase dissolved oxygen (DO) concentrations (Kadlec and Wallace, 2009).

The lack of change in $NO_{2,3}$-N removal requires further investigation. Denitrification, which converts $NO_{2,3}$-N to N_2 or N_2O, may have been limited in MOV1 during storm events. It is also possible that $NO_{2,3}$-N was denitrified, but concentrations remained static due to conversion of N though nitrification. In soil samples taken pre-retrofit, total organic carbon (TOC) content ranged from 0.6 – 1.5%, which is a sufficient carbon source for denitrification (Webster and Goulding, 1989). TOC was lower in samples taken from nearly permanently inundated soils, while higher TOC content was found in soils that were intermittently inundated (*Appendix G*)**,** suggesting that denitrification occurred in inundated soils during the calibration period.

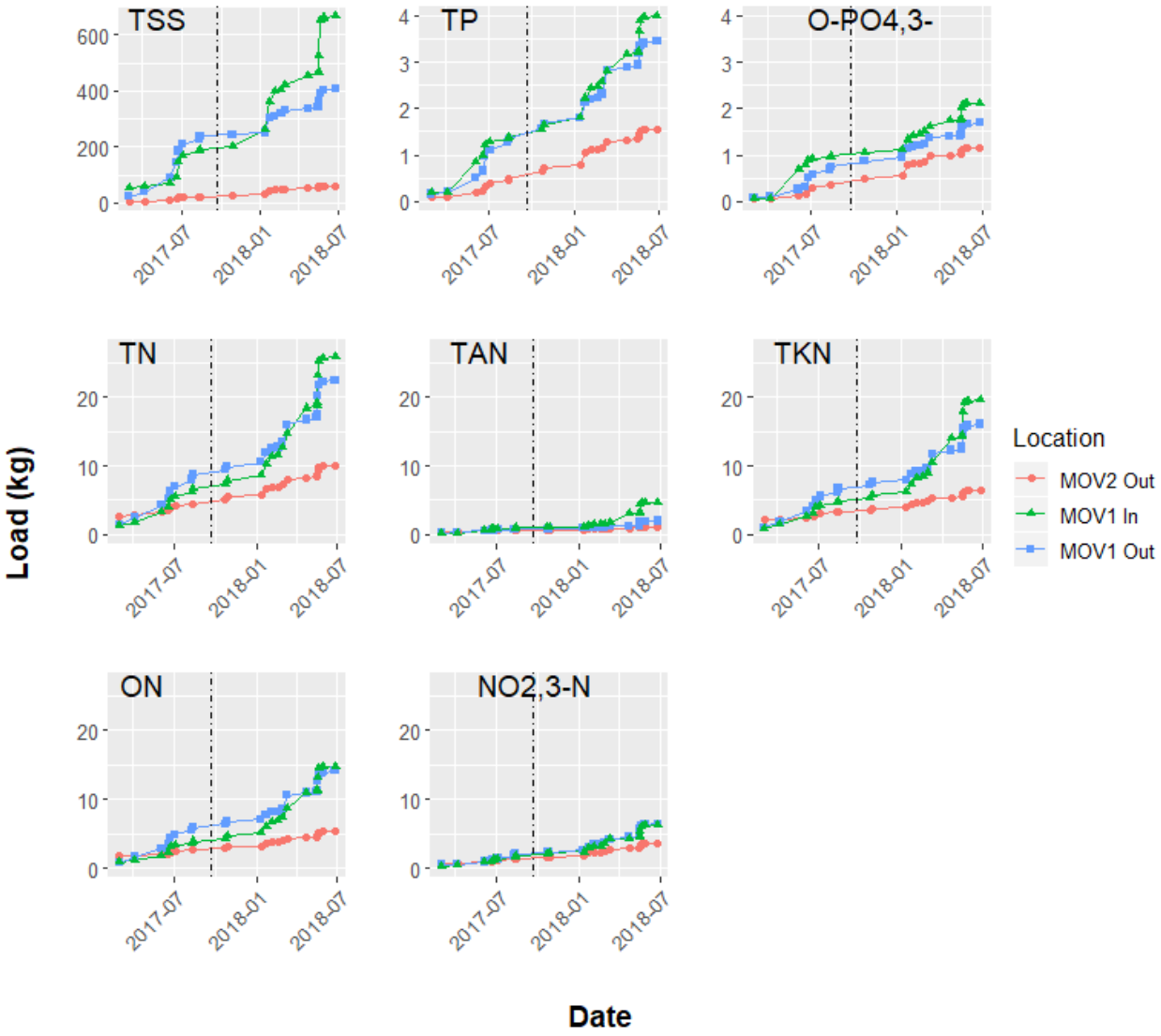

Figure 3-17: Cumulative loads at all Morrisville sites for the entire monitoring period. The vertical line indicates when the pond was retrofitted in September 2017.

ANCOVA plots used to detect differences in mass export rates of MOV1 for both phases of the retrofit have visible reductions for TSS, TP, TN, TKN, and ON (*Figure 3-18*). No significant changes were detected for $NO_{2,3}$-N. O-PO_4^{3-} and TAN loads significantly reductions were detected with both MOV1_IN and MOV2_OUT as explanatory variables, for which visualization is more challenging (*Appendix H*).

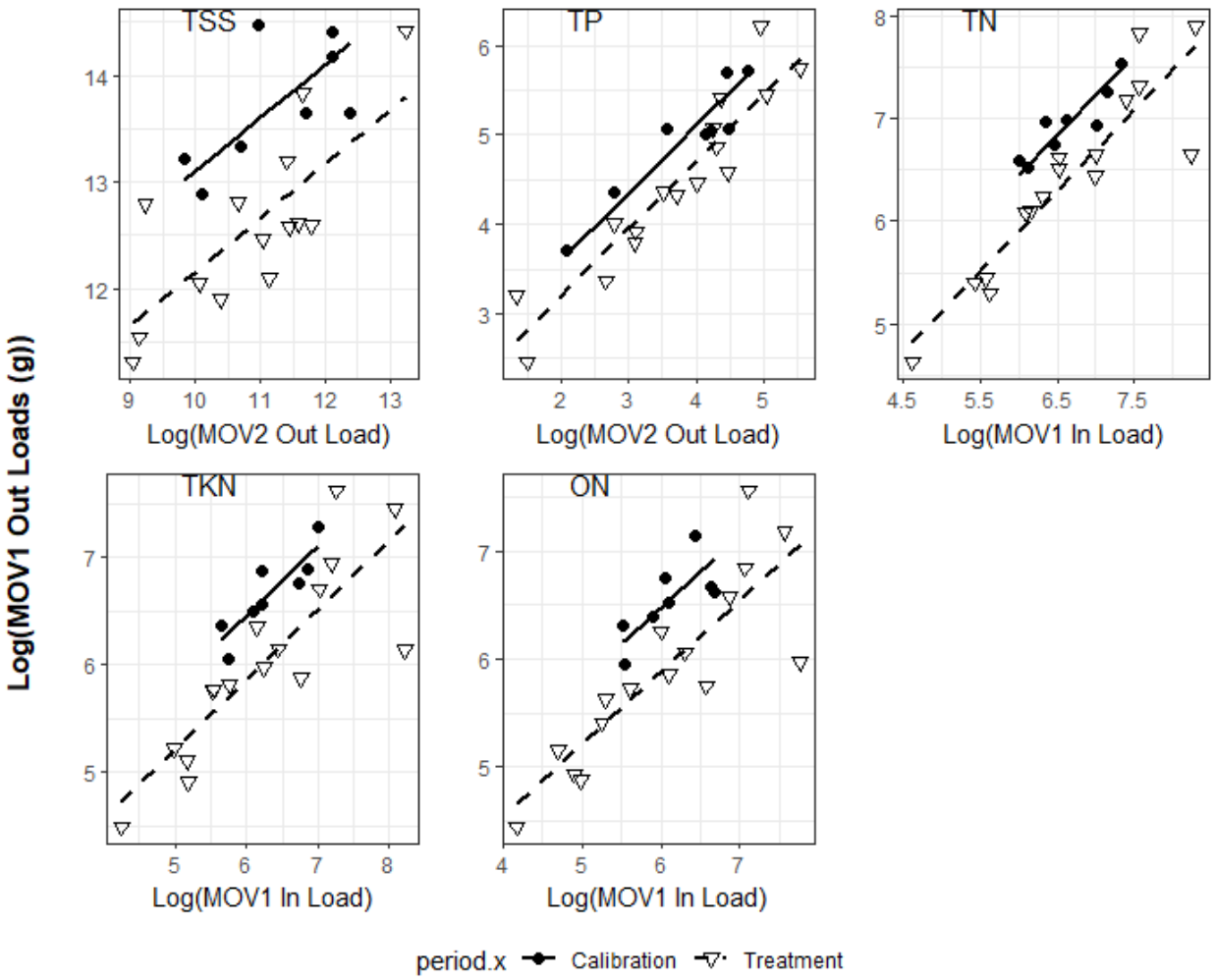

Figure 3-18: ANCOVA of the log of MOV1 TSS, TP, TN, TKN, and ON loads with respect to the indicated covariate.

Calibration and treatment period effluent EMCs of MOV1 were also compared to ambient water quality targets. Barrett et al. (2004) suggested that effluent TSS concentrations should not exceed 25 mg/L to maintain downstream ecosystem health. Median nutrient concentrations in streams with a "good" macroinvertebrate rating in the Piedmont Ecoregion were used as target ambient conditions (McNett et al., 2010). MOV1 effluent EMCs from both periods were compared to these thresholds, with EMCs at MOV2_OUT during the treatment period as a reference (*Figures 3-19 to 3-21*). MOV2 effluent met TSS targets more frequently than that of MOV1 from either monitoring period. While clearly performing better post-retrofit,

MOV1_OUT in the treatment period still only met TSS targets in 53% of storms. Calibration outflows of MOV1 were never less than TSS targets.

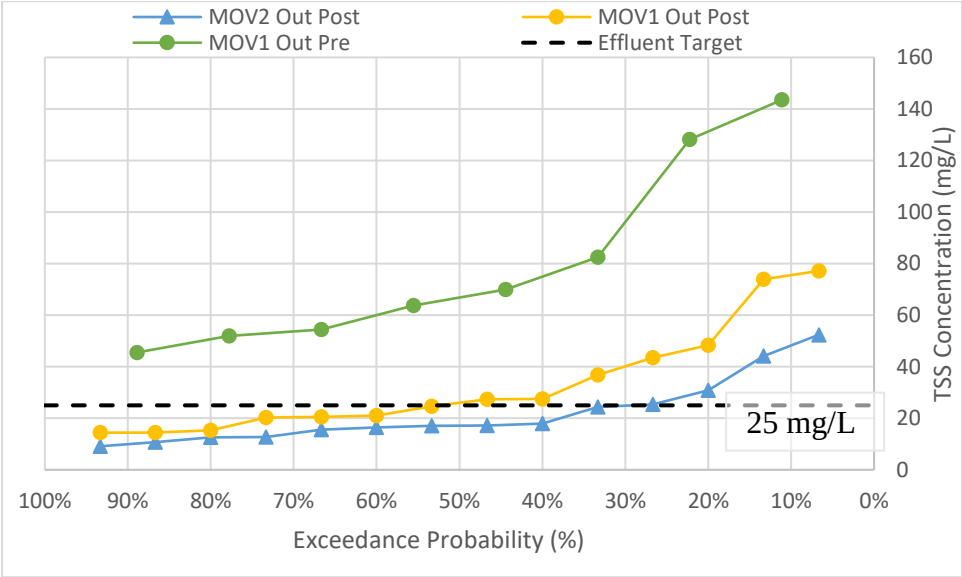

Figure 3-19: Exceedance probabilities of effluent concentrations at MOV1 for both monitoring periods and MOV2 post-retrofit with respect to TSS target concentrations (Barrett et al., 2004).

Additionally, pre- and post-retrofit MOV1 effluent nutrient EMCs were compared to those assigned to dry ponds and CSWs by NCDEQ. These concentrations are based on North Carolina-centric studies. Dry ponds have assigned TP and TN effluent concentrations of 0.66 mg/L and 1.65 mg/L, respectively (NCDEQ, 2017b). These values for dry ponds are subject to change because dry ponds studied in the state are limited to date. Assigned CSW effluent concentrations for TP and TN are 0.18 mg/L and 1.12 mg/L, respectively (NCDEQ, 2017b). The majority of TP samples were lower than that assigned by NCDEQ (2017b) to dry ponds. Effluent TP concentrations in MOV1 never met the McNett et al. (2010) threshold and exceeded DEQ's CSW assigned value in 94% of storms (*Figure 3-20*).

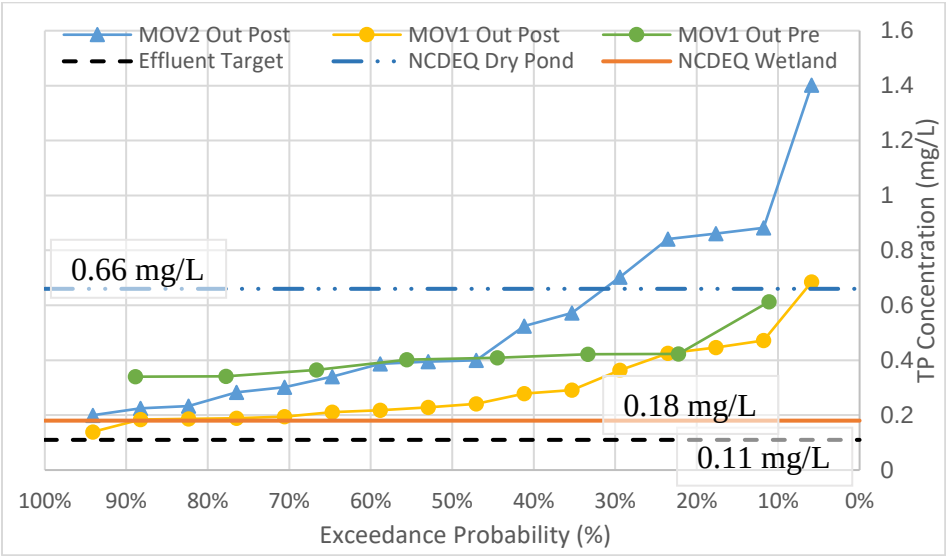

Figure 3-20: Exceedance probabilities for effluent concentrations of TP at MOV1 for both monitoring periods and MOV2 post-retrofit with respect to ambient water quality thresholds (McNett et al., 2010) and NCDEQ (2017b) assigned concentrations for dry ponds and wetlands.

Effluent MOV1 TN EMCs substantially decreased post-retrofit (*Figure 3-21*). However, only 44% of these concentrations were less than NCDEQ (2017b) assigned dry pond effluent concentrations. All effluent concentrations were much higher than ambient water quality thresholds.

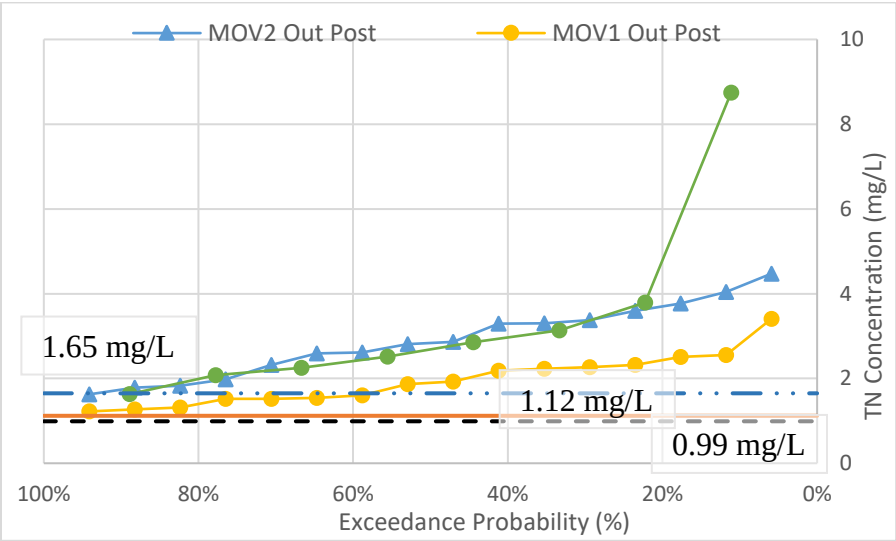

Figure 3-21: Exceedance probabilities of effluent concentrations for TN at MOV1 during both monitoring periods and MOV2 post-retrofit with respect to ambient water quality thresholds (McNett et al., 2010) and NCDEQ (2017b) assigned concentrations for dry ponds and wetlands.

Exceedance probabilities of water quality targets only changed for TSS and TN between monitoring periods (*Table 3-16*). In general, the converted MOV1 did not treat pollutants to a level that met ambient water quality thresholds. In the treatment period, TN effluent concentrations of MOV1 were below NCDEQ values assigned to dry ponds for credit more frequently than in the calibration period. The probability that TP MOV1 effluent concentrations were lower than NCDEQ's assigned values for CSWs increased (2017b) from pre- to post-retrofit. In general, post-retrofit MOV1 treated pollutants to effluent EMCs that were better than those of dry ponds and approaching those of CSWs. Limitations of the design, including lack of storage capacity and a lack of CSW bathymetry may have limited treatment potential of this conversion. However, it is clear that the retrofit improved effluent water quality.

Table 3-16: Exceedance probabilities of MOV1 effluent EMCs during both phases of monitoring with respect to the indicated water quality targets.

	Parameter	**WQ Target Reference**	**WQ Target** *mg/L*	**Exceedance Probability** Calibration	Treatment
Targets	TSS	Barrett et al. (2004)	25	86%	50%
	TP	McNett et al. (2010)	0.11	100%	100%
	TN	McNett et al. (2010)	0.99	100%	100%
	$NO_{2,3}$-N	McNett et al. (2010)	0.59	57%	56%
	TKN	McNett et al. (2010)	0.4	100%	100%
	TAN	McNett et al. (2010)	0.04	100%	88%
Dry Pond Credit	TP	NCDEQ (2017b)	0.66	7%	6%
	TN	NCDEQ (2017b)	1.65	93%	56%
CSW Credit	TP	NCDEQ (2017b)	0.18	100%	94%
	TN	NCDEQ (2017b)	1.12	100%	100%

3.4.2.2: WS

The low directly connected impervious area of WS created a high runoff threshold. An extended calibration period was required to capture a sufficient number of pre-retrofit storms.

Since retrofitting in March 2018, only four storms have been sampled for water quality. The results reported are preliminary and may change as new data are collected.

Median EMCs during both phases of monitoring are reported in *Table 3-17*. WS influent water quality was used as a covariate to detect changes between calibration and treatment period water quality. Significant correlations were found between influent and effluent concentrations of all analytes, allowing for ANCOVA. This analysis detected no significant changes in effluent water quality between monitoring periods, but the post-retrofit sample size was too small to easily detect differences (n = 4). Pre-retrofit water quality treatment was also relatively good in WS, especially compared to other dry ponds (see Chapter 2). The analysis of the retrofit's effect on water quality in WS was moved to Appendix I because the treatment period sample size was very small.

Table 3-17: Median influent and effluent event mean concentrations of WS and concentration removal efficiencies (RE_c) during both phases of monitoring.

Period	**Location**	**TSS**	**TKN**	**$NO_{2,3}$-N**	**TAN**	**TN**	**ON**	**TP**	**O-PO_4^{3-}**
		mg/L	*mg/L*	*mg/L*	*mg/L*	*mg/L*	*mg/L*	*mg/L*	*mg/L*
Calibration n = 10	WS_IN	105	1.87*	0.21	0.18	2.09	1.67	0.52	0.15
	WS_OUT	53	1.06*	0.14	0.13	1.25	0.98	0.43	0.12
	WS RE_c**	**49%[a]**	**38%[b]**	**34%[b]**	**29%[a]**	**41%[b]**	**42%[b]**	**22%[a]**	31%[a]
	p-value	0.002	0.004	0.002	0.016	0.002	0.002	0.015	0.189
Treatment n = 4	WS_IN	170	2.32	0.21	0.26	2.53	1.87	0.69	0.17
	WS_OUT	90	1.30	0.14	0.2	1.43	1.10	0.35	0.11
	WS RE_c	35%	19%	16%	11%	19%	17%	31%	34%
	p-value	0.238[a]	0.288[a]	0.244[a]	0.195[a]	0.273[a]	0.386[a]	0.101[a]	-

**n = 9, data with quality control issues indicated by the lab were removed*
***bold values are significant*
[a]*Student's t-test*
[b]*Wilcoxon Signed Rank test*

3.4.3: SCM Effluent Concentration Comparison

Dry pond effluent EMC has been highly variable in previous research, particularly for TSS and $NO_{2,3}$-N. Pre-retrofit concentrations of both MOV1 and WS were within the ranges of those for other dry ponds. TSS mean EMC of MOV1 from the treatment period was similar to those of constructed wetlands from previous studies (*Table 3-18*). $NO_{2,3}$-N concentrations leaving WS from both periods were more similar to CSWs than dry ponds, while $NO_{2,3}$-N concentrations in the retrofitted MOV1 remained similar to those of other dry ponds. Effluent concentrations of TSS and TKN in WS remained high and comparable to those of other dry ponds. Differences detectable in effluent concentrations between monitoring periods at WS are limited due to few data.

Generally, TN concentrations from CSWs are slightly elevated due to levels of organic matter, which cannot be easily treated. However, MOV1 TN effluent EMCs were much higher than those of CSWs, suggesting that MOV1 failed to treat nitrogen as completely as a "typical" wetland. This may be due, in part, to low levels of denitrification (Greenway, 2004), as effluent $NO_{2,3}$-N EMCs of the MOV1 post-retrofit were much higher than those of CSWs. Post-retrofit MOV1 effluent concentrations of TAN were also somewhat higher than those of CSWs from the literature. As CSWs are designed to treat both of these pollutants, the lack of treatment in MOV1 suggests limited capability of the retrofit. It is likely that undersizing and the lack of wetland bathymetry may limit treatment potential of the pond-turned-wetland (Tucker, 2007).

Table 3-18: Effluent EMCs of MOV1 and WS pre- and post-retrofit for pollutants monitored compared to those of previously studied dry ponds and constructed stormwater wetlands.

SCM Type	Reference	Name/Location	Mean Effluent Concentration (mg/L)						
			TSS	TP	O-PO_4^{3-}	TN	TKN	$NO_{2,3}$-N	TAN
Dry Pond	herein	MOV1_OUT (pre-retrofit)	80	0.41	0.22	3.38	2.66	0.71	0.32
Dry Pond/CSW	herein	MOV1_OUT (post-retrofit)	33	0.30	0.14	1.95	1.35	0.61	0.16
Dry Pond	herein	WS_OUT (pre-retrofit)	85	0.48	0.18	1.29	1.13	0.14	0.14
Dry Pond/CSW	herein	WS_OUT (post-retrofit)	101	0.50	0.22	1.74	1.55	0.19	0.20
Dry Pond	Carpenter (2014)	Quebec City, Canada	75	-	-	-	-	-	0.19
Dry Pond	Birch et al. (2006)	Sydney, Australia	93	0.23	-	2.91	1.13	1.82	-
Dry Pond	Caltrans (2004)	California[a]	39	0.32	0.14	2.83	1.85	0.98	-
Dry Pond	Stanley (1996)	Greenville, NC	32	-	0.10	-	-	0.36	0.12
CSW	Lenhart & Hunt (2011)	River Bend, NC	41	0.23	0.09	1.11	0.94	0.17	0.08
CSW	Hathaway & Hunt (2010)	Mooresville, NC	9	0.01	-	0.72	0.67	0.07	0.03
CSW	Line et al. (2008)[b]	Asheville, NC	31	0.12	0.01	0.94	0.79	0.15	0.08
CSW	Line et al. (2008)[b]	Piedmont Region, NC	18	0.99	0.13	1.00	0.87	0.13	0.14

[a]*Compilation of six basins*
[b]*median effluent EMC*

3.4.4: Wetland Chemistry

Nitrogen and phosphorus are two nutrients of concern. Changes in soil and water chemistry, induced by the retrofit, can affect TN and TP effluent concentrations. In aerobic conditions, phosphorus can be stored in the soil, attached to soil particles through iron bridging (Stumm and Morgan, 1996). When soils become anaerobic, iron that bonds phosphorus to soil particles is reduced and becomes soluble, releasing available soil phosphorus, creating higher dissolved phosphorus concentrations in water. In this study, much of MOV1 was transitioned to permanently inundated conditions, which could potentially release soil phosphorus (Stumm and Morgan, 1996). Soil samples were taken during the calibration phase to determine the P-index, a measure of the saturation of phosphorus in soils, of which there is a limited capacity (Hardy et al. 2009). Both in MOV1 and WS, The P-index was found to be very low (1-3) in both MOV1 and WS (*Appendix G*), with a high capacity for P storage and low potential to release P in anaerobic

conditions. These levels were similar to P-indices from Hunt et al. (2006), which resulted in high rates of P removal in bioretention cells, another vegetation-based SCM. Removal rates of TP and O-PO_4^{3-} greatly increased during the treatment phase, indicating that soil phosphorus levels did not inhibit pollutant removal.

Treatment wetlands are commonly implemented to reduce nitrogen concentrations, particularly $NO_{2,3}$-N. Yet, this was the only pollutant for which the retrofit in MOV1 did not significantly reduce effluent concentrations. $NO_{2,3}$-N is most commonly removed in wetlands through plant uptake and denitrification (Greenway, 2004). For denitrification to occur, both anaerobic conditions and an organic carbon source must exist (Lefevre et al., 2015). Pre-retrofit, soil samples were taken MOV1, and percent TOC in the soil was found range from 0.47 – 1.62% (*Appendix G*). These rates are considered sufficient for some denitrification (Webster and Goulding, 1989).

Water chemistry during baseflow can help to characterize dynamics in MOV1. Longer inter-event residence times may have altered the basin's treatment potential. RE_c of baseflow $NO_{2,3}$-N was found to be consistently high and significant (*Table 3-19*). Effluent concentrations of $NO_{2,3}$-N were below the practical quantitation limit (PQL) in 75% of samples, and mean effluent concentration was 0.008 mg/L. It is likely that this removal was due to denitrification because, in MOV1, residence time ranged from 2 to 19 days. Maximum denitrification rates occur after 5 days of soil contact time (Xue et al., 1999).

There were significant reductions in baseflow concentrations of O-PO_4^{3-}. This reduction further illustrates that soil phosphorus content did negatively impact dissolved P in the dry pond-turned-wetland. There were significant increases in concentrations of TKN, ON, and TP (*Table 3-19*). However, ON was the greatest contributor to TN in MOV1, and mean TN effluent EMC

was consistent with typical CSW TN effluent concentrations (*Table 3-18*). Mean effluent concentration of TAN was only 0.057 mg/L, much less than effluent concentrations associated with storm events. These concentrations were also similar to effluent water quality thresholds for TAN (McNett et al., 2010).

Table 3-19: Concentration removal efficiency (RE_c) of pollutants during baseflow at MOV1 (n = 7).

		TKN	**$NO_{2,3}$-N**	**TAN**	**TN**	**ON**	**TP**	**O-PO_4^{3-}**	**TSS**
Mean C_i - C_o	*mg/L*	-0.85	1.13	-0.017	0.29	-0.83	-0.080	0.022	-7.17
Median RE_c**	*%*	-241.9%	99.4%	-88.8%	33.3%	-291.2%	-43.0%	18.4%	-51.1%
Mean C_o	*mg/L*	1.12	0.008	0.057	1.13	1.06	0.23	0.10	25.0
p-value		**<0.0001**	**<0.0001**	0.0613	0.297	**<0.0001**	**0.011**	**0.0490**	0.462

**bold values significant*
***Negative sign indicates an increase in concentration from inlet to outlet*

$NO_{2,3}$-N removal rates in baseflow suggest that conditions exist for denitrification. These removal rates are not reflected during storm events. During baseflow, there was no evidence to suggest that TAN was nitrified, which could be manifested through an increase in $NO_{2,3}$-N concentrations. However, during storm events, TAN loads were greatly reduced, which could have been in part though nitrification. This is likely, as turbulent flows during storm events can increase DO concentrations, increasing nitrification potential (Fondriest Environmental 2013). Nitrification yields increased $NO_{2,3}$-N concentrations, which consequently limits the ability to detect $NO_{2,3}$-N treatment. Additionally, denitrification occurs with increased residence time (Zarnetske et al., 2011). Storm duration of sampled events from initial precipitation to final drawdown ranged from 17 to 76 hours, with an average duration of 51 hours. Outflow overtopped the WQV height in 38% of storms, with overflow volumes ranging from 2 – 45% of total effluent volumes in these events. No significant correlations were found between effluent nitrate concentration and storm duration or peak flow rate (*Appendix J*). Often, seasonality has an effect on nitrate removal, but there were no seasonal trends detected. Based on these differences

and observations, a lack of nitrate removal could be due to a lack of anaerobic conditions and contact time with the soil. It is possible that a reduced orifice size, which would increase residence time of the WQV, could improve denitrification rates.

3.4.5: Cost Benefits

In many areas of North Carolina, regulations for allowable loads reaching rivers and lakes are stringent. The burden of reducing pollutant loads in existing developments where SCMs are already in place often falls to municipalities. Implementing new SCMs can become expensive (Weiss et al., 2007). When developing, some water quality treatment must be on site (implementation of local SCMs), while the balance can be nutrient credits from the NC Department of Mitigation Services (DMS; *Table 3-20*) or another source to offset nutrient loads (NCDEQ, 2018b).

To quantify the return on investment of the MOV1 retrofit, the nutrient offset credit equivalent of the load reduction of the 4.15 ha watershed due to the retrofit over 30 years was calculated for three major watersheds in North Carolina (*Table 3-21*). Credit values were estimated at the rate charged for nutrient offset credits by DMS (NCDEQ, 2018a). Credit equivalent values ranged from $48,800 to $365,000, which was provided by the $2,000 retrofit installation. The wide range is attributed to unequal costs of offset credits among watersheds (*Table 3-20*).

Table 3-20: Nutrient offset credit costs as listed by DMS (NCDEQ, 2018a) in three major NC watersheds.

	NC DMS Cost ($/kg)		
	Falls	Jordan	Tar-Pamlico
TP	$ 413	$ 757	$ 260
TN	$ 23	$ 290	$ 18

Table 3-21: Cost of nutrient credits equivalent to the pollutant reduction of the MOV1 retrofit.

	Effluent Loads*			**Buy Down Credit Equivalency****		
	Calibration	Difference	Reduction	Falls	Jordan	Tar-Pamlico
TP Load (kg/yr)	7.29	60%	4.37	$ 54,000	$ 99,100	$ 34,000
TN Load (kg/yr)	43.4	71%	30.7	$ 21,300	$ 268,000	$ 16,800
	Total Savings			**$ 75,400**	**$ 366,900**	**$ 50,800**
	- Investment			$ -2000	$ -2000	$ -2000
	Totals			**$ 73,400**	**$ 364,900**	**$ 48,800**

**Calibration period loads reported previously were multiplied by watershed area for total load*
***Credits were calculated by multiplying unit costs in Table 3-20 by total load reduction over a 30-year period*

With such low cost of implementation, the savings would pay for retrofit costs between 1 and 6 years. Such low retrofit costs were due in part to the exclusion of earthwork required by retrofit design. The evidence suggests that despite the lack of a flow path and designated wetland zones, typical of constructed stormwater wetland designs, the wetland retrofit of MOV1 still improved water quality treatment.

Data still needs to be collected in WS for a proper evaluation of this retrofit. However, limited storage capacity and a lack of a control orifice likely limit the effectiveness of this retrofit.

3.5: Summary and Conclusions

This study examined the viability of implementing a dry pond retrofit to create a quasi-stormwater wetland. Both water quality treatment enhancement and retrofit cost were considered. Two dry ponds were evaluated in this study, but only one of which thoroughly.

- Using a combination of a paired watershed and upstream/downstream design, the retrofit in MOV1 showed significant effluent EMC reductions of 58%, 65%, 69%, 64%, 82%, 71%, and 80% for TSS, TP, O-PO_4^{3-}, TN, TKN, TAN, and ON, respectively. Concentrations of $NO_{2,3}$-N were not significantly reduced.

- Load reductions in MOV1 due to the retrofit were significant for TSS, TP O-PO_4^{3-}, TN, TKN, TAN, and ON with reductions of 89%, 60%, 57%, 71%, 75%, 69%, and 75%, respectively. Export loads of $NO_{2,3}$-N did not change significantly.
- Although greatly improved, effluent water quality in the converted MOV1 did not meet ambient water quality thresholds proposed by McNett et al. (2010) for most analytes. Exceedance probabilities of these thresholds were less than 100% for TSS, $NO_{2,3}$-N, and TAN, at 50%, 56%, and 88%, respectively.
- In MOV1, effluent TP concentrations post-retrofit (0.30 mg/L) were much lower than expected effluent concentrations for dry ponds (0.66 mg/L). These concentrations approached, but did not match, expected CSW effluent concentrations (0.18 mg/L). Effluent TN concentrations (1.95 mg/L) from the retrofit that ranged from exceeded expected dry pond concentrations (1.65 mg/L) in 56% of storms. These concentrations were much higher than expected effluent CSW concentrations (1.12 mg/L). MOV1 storage volume was only 65% of design requirements for CSWs (NCDEQ, 2017a), which may have limited treatment.
- Baseflow sampled in MOV1 during the treatment period showed significant reductions in $NO_{2,3}$-N, indicating denitrification potential in the converted basin. Lack of $NO_{2,3}$-N removal during storm events may have been due to lack of storage or short detention times. However, high reductions of TAN suggest nitrification during storm events. A lack of apparent removal of $NO_{2,3}$-N during storm events may be due in part to the production of $NO_{2,3}$-N in the basin. If so, denitrification also occurred during storm events.
- Preliminary data do not show any significant difference in effluent concentrations in WS between the calibration and treatment periods. No significant differences were detected in

total mass exported between monitoring periods, but any significant findings are nearly impossible here due to a very small sample size.

- The retrofits in MOV1, a single dry pond, generated nutrient reductions equivalent to nutrient credits worth $34,000 – $99,000, and $17,000 - $268,000 for TN. Pollutant loads were greatly reduced downstream with simple and low-cost retrofit designs. Potential exists to replicate such retrofits in dry ponds throughout North Carolina and the United States to improve water quality.

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CHAPTER 4: Dry Pond Retrofit Design Recommendations and Future Research Considerations

NCDEQ (2017a) no longer allows a dry pond to be implemented as a stand-alone stormwater control measure (SCM) because it does not efficiently remove TSS from stormwater. Additionally, low nutrient removal rates assigned to dry ponds (NCDEQ, 2017a) may often require developers to implement other forms of mitigation. This may often include purchasing buy down credits. There is potential to enhance water quality treatment of dry ponds by modifying (or retrofitting) them. One potential dry pond retrofit is the incorporation of wetland features.

Recommendations for design of such a retrofit are detailed herein. These modifications are prioritized by effectiveness for conversion, ease of implementation, and cost.

4.1: Outlet structure modification

4.1.1: Creation of a permanent pool

One of the main differences between a CSW and a dry pond is the existence of a permanent pool of water in a CSW above the bottom of the basin (*Figure 4-1 and 4-2*). A permanent pool facilitates sedimentation, reduces resuspension of particles, and creates zones for nitrification/denitrification. Diverse wetland bathymetry, including deep pools and shallow water zones, may additionally enhance pollutant removal. Exact specifications of depth and zone requirements vary by state, but the designer may be able to optimize permanent pool depth to emulate standards for CSWs. However, as the level of permanent pool is raised, pond storage volume decreases, which alters basin hydraulics. The extent to which these alterations are acceptable will vary by pond. Finally, verifying that the new permanent pool depth does not negatively influence hydraulic grade line is important.

Figure 4-1: MOV1 dry pond without a permanent pool.

Figure 4-2: MOV1 was retrofitted to include a permanent pool. Wetland vegetation had not yet matured.

In some cases, the riser structure need not be directly modified to establish a permanent pool. A structure setting new permanent pool elevation, orifice diameter, and water quality volume (WQV) elevation can built around the original orifice. Avoiding direct modification of outlets minimizes costs.

When directly modifying the outlet structure, means of altering the elevation of the water quality orifice is dictated by the outlet structure itself. An upturned elbow can be a simple and cost-effective way to alter the elevation of the water quality orifice. Upturned elbow orientation must be considered carefully, as orifice placement can affect both performance and maintenance

(Hunt et al., 2011). Upturned elbow retrofits have previously been installed on bioretention cells for nutrient removal and infiltration enhancement (Brown and Hunt, 2011). No evidence suggests a preferential orientation. However, there are some common practices in outlet design, which may be applied to these retrofits. In CSW designs, downturned elbows are often installed to limit clogging from floating debris (*Figure 4-3*).

Figure 4-3: CSWs can have downturned elbows to limit clogging from floating debris (NCDEQ, 2017).

Somes and Wong (1997) studied a siphon outlet, which was reported to perform well hydrologically, including a relatively steady drawdown rate (*Figure 4-4*). With modifications of this outlet type, clogging from both floating debris and sediment is potentially minimized.

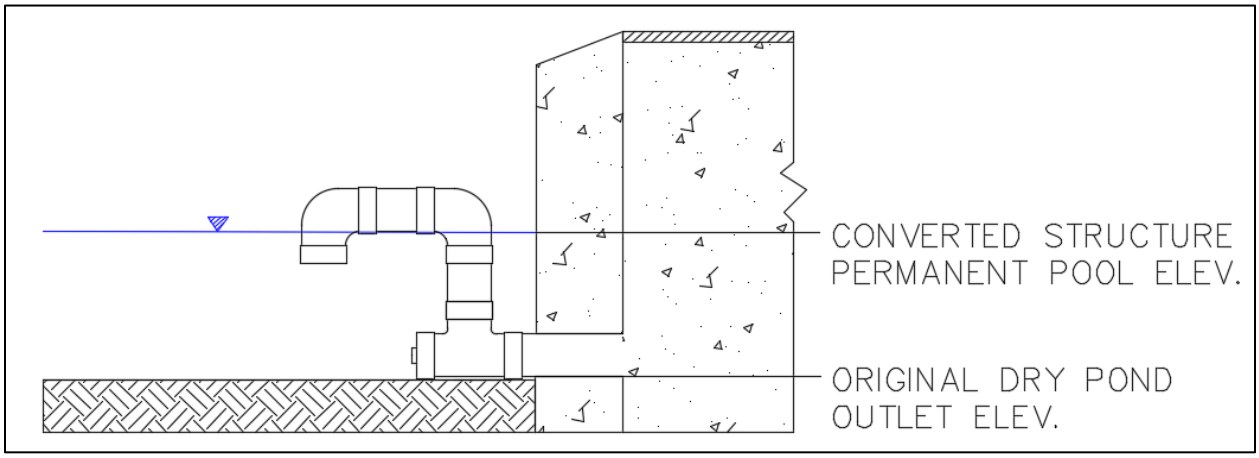

Figure 4-4: A siphon outlet structure can improve hydrologic effectiveness and may reduce clogging.

No matter the configuration, clogging within the upturned elbow can be a concern. A tee can be installed over an elbow with an attached removable cap to allow for ease of cleanout in case of clogging. Covering the original orifice and drilling a new orifice should be avoided if possible. This alternative would greatly increase costs and potentially lead to maintenance issues.

4.1.2: Drawdown Optimization

In addition to raising the permanent pool, drawdown times must be adjusted to meet water quality regulations for CSWs. Although the principal orifice in dry ponds must be sized for a drawdown time of 2 – 5 days (*Figure 4-5;* NCDEQ 2007, 2017b), many dry ponds exist that do not meet this criterion. Some dry ponds may not be sized for water quality volumes, often predating newer design standards; the orifice does not detain water for a sufficient period of time to meet water quality regulations. When retrofitting a dry pond into a CSW for water quality, detention time is essential, and the orifice size should be reduced to meet newer detention time standards. Orifice size can be modified by incorporating an orifice plate or cap with an orifice diameter as small as 25 mm; however, Hoyt and Brown (2005) suggested 75 mm be the minimum orifice size to prevent clogging.

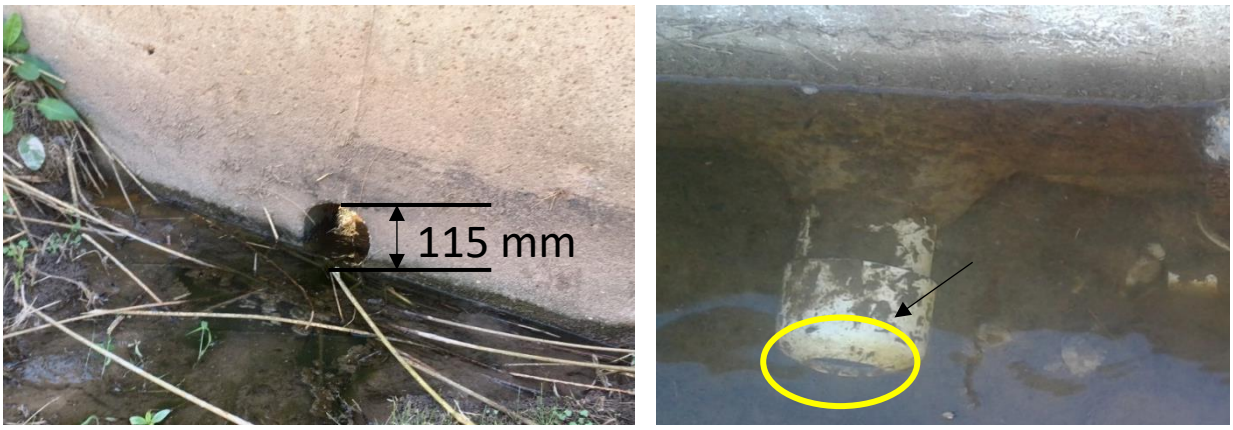

Figure 4-5: A dry pond's 115-mm orifice provided little detention time for the watershed area (left). The orifice was reduced to 50 mm to increase detention time to 2 days (right).

4.1.3: Peak Flow Mitigation / Maintaining Flood Control

Because changing the principal drawdown orifice will alter hydraulics in the basin, flood control concerns must be addressed. Reduced storage volume for runoff and a restricted drawdown rate will likely increase peak outflows for larger storm events. Additionally, it may be necessary to modify the outlet structure (e.g. with new orifices and weirs) after adjusting the initial orifice.

4.2: Adding Plants to Enhance Pollutant Removal

Although increased hydraulic retention time (HRT) and a permanent pool alone have been found to be highly correlated with pollutant removal (Middleton and Barrett, 2008; Shammaa et al., 2002), other design features associated with CSWs can be implemented to enhance pollutant reduction.

Greenway (2004) demonstrated that plants have been shown to play a big part in pollutant removal – both for particulate and dissolved pollutants. In CSWs, plants can provide surface area for particle adhesion, enhance sedimentation, promote uniform flows, and stabilize existing soils within the basin (Wong et al., 1999). Permanent inundation will kill turf grass (most typical vegetation of a dry pond), destabilizing the dry pond bottom. Barren areas will need vegetation that are adapted to wetland conditions. While shallow open water will attract certain wetland species, ultimately, mosquito habitat could be created if vegetation is not carefully considered (Hunt et al. 2006). This is especially a concern near urban areas.

Many ecosystem services beyond hydrologic and water quality benefits have been attributed to wetland plants. There is potential for additional habitat, educational and recreational opportunities, carbon sequestration, air quality improvement, and resource harvesting ((Moore and Hunt 2012; De Groot, 2006; MEA, 2005) by including plants in a dry pond retrofit.

Although incorporating select plants may require investment, there are likely sufficient benefits to merit the effort. Plant selection, spacing, and placement should match those required for CSWs. Facultative plants should be selected in areas where a permanent pool is not guaranteed. This is possible in areas with infiltrating soils and in basins where deep pools do not exist to maintain standing water in times of drought.

4.3: Altering Basin Topography

Another signature design feature of CSWs is wetland bathymetry, which is usually a mix of shallow water, deep pools, and temporary inundation zones along a designated flowpath (NCDEQ, 2017b). Varied topography is intended to create multiple opportunities for pollutant removal, while the flowpath increases HRT, maximizing hydrologic effectiveness, which is the interaction between storage volume, runoff capture, and detention time (Wong et al., 1999).

Both hydrologic effectiveness and hydraulic efficiency, which quantifies mixing and the proportion of basin volume used, contribute to treatment efficiency of a wetland (Wong et al. 1999). While a flowpath that prevents short-circuiting in a CSW can increase hydraulic efficiency (Persson et al., 1999), the vegetation-wetland bathymetric interface is the greatest factor affecting hydraulic efficiency (Wong et al., 1999). Proper placement of vegetation and calculation of the correct proportion of wetland zones may enhance the efficiency by providing a stable ecosystem with many pollutant removal mechanisms (Greenway, 2004). These modifications (previously discussed) are relatively inexpensive compared to flowpath modification, which could require substantial earthwork and mobilization costs. High costs could deter retrofitting.

In North Carolina, nutrient offset credits can be purchased when in-place SCMs do not meet required pollutant load removal. Proposed retrofits must be lower than the cost of offset

credits to merit investment. Therefore, it is ideal that retrofit costs are optimized with respect to pollutant removal. From this retrofit study, it has been shown that enhanced pollutant removal can occur without expensive earthwork. Small investments like outlet modification and plant installation led to large improvements in water quality treatment in the basin without the need for excavating.

There are some cases where “spot” earthwork may be beneficial or even necessary. Dry ponds are often not designed with a deep pool or sump near the outlet. In these cases, it may be beneficial to add this feature to minimize sediment build up near the orifice. Persson et al. (1999) found that adding a small island or submerged berm near the inlet can greatly increase hydraulic efficiency. Both modifications require minimal earthwork and could enhance performance. Also, large slopes between the inlet and outlet may limit coverage of a permanent pool, in which case, low-lying berms may be installed on grades across the basin to promote ponding throughout the SCM (*Figure 4-6*).

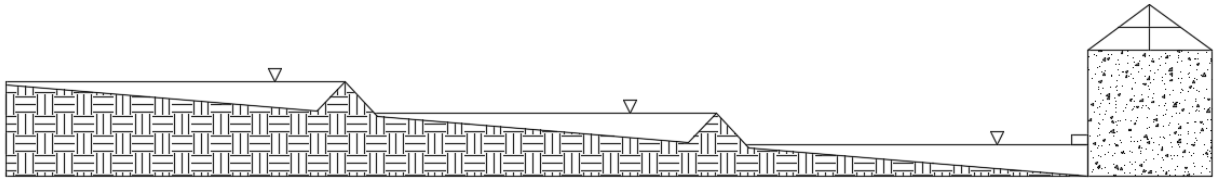

Figure 4-6: Berms may be constructed to increase permanent pool area when basin elevation changes are drastic.

This study only addressed retrofits without earthwork. More investigation is needed to better understand how modifying bathymetry affects pollutant removal relative to simpler modifications. The retrofit studied substantially improved nutrient capture, even without earthwork.

4.4: Other Design Considerations

4.4.1: Effects of Sizing

In addition to these design considerations, other factors may affect the success of dry pond retrofits and should be considered before investing in changes. Standard storage space for the WQV is often specified to include runoff from the first 25 mm of an event (NCDEQ, 2017b; VDEQ, 2013; CWP, 2009). By limiting maximum ponding depth requirements, CSWs have larger surface areas than other SCMs. Maximum ponding depth of the WQV is not a limitation for dry ponds, which may result in undersized CSWs upon conversion, potentially limiting pollutant removal potential. Several studies have quantified pollutant removal capabilities of undersized wetlands, with varying success (*see Table 1-7*). For example, Hathaway and Hunt (2010) found that a wetland sized to treat 27% of the WQV reduced pollutants with RE_cs of 84%, 52%, and 62% for TSS, TN, and TP, respectively. In the conversion studied in this thesis, the resulting CSW treated 82% of the WQV, with significant pollutant removal increases from the original dry pond. More research is needed to determine the impact of sizing ratio on retrofitted dry ponds.

4.4.2: Maintenance

Maintenance of the pond post-retrofit should resemble that for CSWs (NCDEQ, 2017b; Blecken et al., 2017; Hunt et al., 2011). When a deep pool does not exist near the outlet, special attention should be paid to sediment buildup near and around the orifice. Vegetation should be monitored to exclude cattails and other mosquito-protecting species from the basin (Hunt et al., 2006).

4.5: References

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APPENDICES

Appendix A: Evaporation and Infiltration

Daily evaporation rates were taken from the NC Cronos for the monitoring period. Using a surface created in AutoCAD Civil 3D, water surface area was estimated in time steps based on water level Total evaporation was estimated for storm events using average daily evaporation rates and water surface areas determined using a rendered surface in AutoCAD Civil 3D. Total volume evaporated was estimated for the five storms with the longest duration (those assumed to be most influenced by evaporation). Estimated evaporation volumes and these volumes as a percentage of total runoff volume were estimated for MOV1 and MOV2 (*Table A-1* and *A-2*). Based on these estimates, evaporation volumes were excluded from the water balance in Chapters 2 and 3.

Table A-1: Evaporation Volumes as a percentage of total volume in MOV1.

Duration	**$V_{outflow}$**	**V_{ET}**	**ET % of V_{out}**
h	*cf*	*cf*	
72:02	163521	152	0.09%
35:02	25989	73	0.28%
28:00	14047	13	0.09%
21:30	17423	54	0.31%
21:22	13011	17	0.13%

Table A-2: Evaporation Volumes as a percentage of total volume in MOV1.

Duration	**$V_{outflow}$**	**V_{ET}**	**ET % of V_{out}**
h	*cf*	*cf*	
24:50	2838	<0.0	0.00%
70:44	77411	87	0.11%
32:46	9078	11	0.12%
24:44	87	<0.0	0.00%
21:48	2857	0.1	0.00%

An MPD infiltrometer from Upstream Technologies™, was used to estimate infiltration rates in all studied dry ponds (*Table A-3*).

Table A-3: Infiltration rates of studied dry ponds.

Pond	**n**	**Mean**	**25%**	**Median**	**75%**	**Std Error**
		cm/hr	cm/hr	*cm/hr*	*cm/hr*	
MOV1	7	1.6	*0*	0.10	*2.8*	2.2
MOV2	5	6.64	0	0	0	14.9
WS	4	1.70	0.6	1.9	2.9	14.2

Many of the tests were considered NULL because of a lack of drawdown in the device. All tests were left for 24 hours, and infiltration was considered to be 0 cm/hr when water was still present in the infiltrometer upon completion of the test. Detailed results of these tests are reported in *Table A-4.*

Table A-4: Results of all measured infiltration tests during the study.

Test Date	**Pond**	**Test Name**	**Inf. Rate *mm/hr***	**Comments**
6/9/2017	Wet	MDPWET1	NULL	
6/9/2017	Wet	MDPWET1.1	37	*Test only lasted 2 hours, drew down to 21 cm
6/9/2017	Wet	MDPWET3		
6/28/2017	Wet	Morrisville Wet Out	54	*drew down to 12.72 cm in 1 hour
7/6/2017	Wet	Morrisville Wet	NULL	
7/10/2017	Wet	MDPWET3	1	*moisture content adjusted by manufacturer
7/11/2017	Wet	MDPWET4	0	*didn't draw down by the next day
7/17/2017	Wet	MDPWET4.2	0	*didn't draw down by the next day
7/18/2017	Wet	MDPWET5	19	*7-hour test. Drew down to 15 cm
9/10/2017	Wet	MDPWET7	0	*didn't draw down by the next day
7/31/2017	Dry	MDPDRY1	332	*drew down to 4.8 cm in 21 min
8/1/2017	Dry	MDPDRY2	NULL	*test ripped up by maintenance guys
10/3/2017	Dry	MDPDRYOCT2	0	*didn't draw down by the next day
10/5/2017	Dry	MDPDRYOCT5	0	*didn't draw down by the next day
10/17/2017	Dry	MDPDRYOCT17	0	*didn't draw down by the next day
10/18/2017	Dry	MDPDRYOCT18	0	*didn't draw down by the next day
6/14/2017	WS	WS1	NULL	
10/27/2017	WS	WSOCT27	0	*didn't draw down by the next day
6/28/2018	WS	SherwoodForestElem1	26	
6/28/2018	WS	SherwoodForestElem2	11	
6/28/2018	WS	SherwoodForestElem3	31	

Figure A-1: Infiltration tests were left for 24 hours, and many had water remaining in this column after this period.

Web Soil Survey was used to identify underlying soil conditions in Morrisville (NRCS, 2018).

Table A-5: Soil types of Morrisville catchment area, taken from Web Soil Survey.

Symbol	**Soil Type**	**Rating**	**% of Area**
CaB	Carbonton-Brickhaven complex, 2-6 % slopes	D	16.0%
CaC	Carbonton-Brickhaven complex, 6-10 % slopes	D	12.0%
ChA	Chewacla and Wehadkee soils, 0-2% slopes, frequently flooded	B/D	4.2%
Ur	Urban land		67.8%

Figure A-2: The majority of soils within the watersheds of MOV1 and 2 are considered urban land or in hydrologic soil group D.

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Appendix B: Infiltrometer Report

K_{sat} best-fit site average could not be calculated with these tests

GPS Infiltration Test Site Map

Map Pin #	Test #	Test Name	Ksat (cm/sec)	Ksat (in/hr)	C (cm)	RMS Error of Regression
1		mdpwet4	NULL	NULL	NULL	NULL

* NULL: Test Data collected was not viable. Ksat cannot be calculated

** NULL tests were removed from the site average calculation

This report summarizes the results of a set of Modified Philip Dunne (MPD) Infiltrometer tests performed at the above referenced site. North Carolina State University personnel performed the field tests. The software used to compute saturated hydraulic conductivity (K_{sat}) and generate this report assumes that the field personnel used infiltrometers manufactured by Upstream

Technologies Inc. and followed the procedures outlined in “Manual – Modified Philip - Dunne Infiltrometer” by Ahmed, Gulliver, and Nieber.

The following paragraphs describe the individual tests, input values used in the analysis, and methods used to compute the K_{sat} value.

After individual K_{sat} values were calculated, the method used to determine the overall site K_{sat}value ($K_{best\text{-}fit}$) is described in "Effective Saturated Hydraulic Conductivity of an Infiltration-Based Stormwater Control Measure" by Weiss and Gulliver 2015, "A relationship to more consistently and accurately predict the best-fit value of saturated hydraulic conductivity used a weighted sum of 0.32 times the arithmetic mean and 0.68 times the geometric mean."

METHOD USED TO COMPUTE K_{sat}

The MPD Infiltrometer software uses the following procedure described in "The Comparison of Infiltration Devices and Modification of the Philip-Dunne Permeameter for the Assessment of Rain Gardens" by Rebecca Nestigen, University of Minnesota, November 2007.

The steps are as follows:

1. For each measurement of head, use the following equation to find thecorresponding distance to the sharp wetting front.

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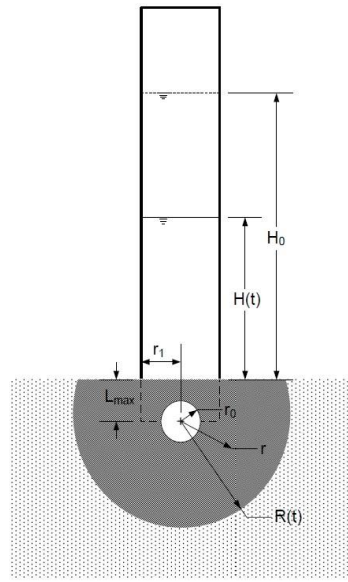
2. Estimate the change in head with respect to time and the change inwetting front distance with respect to time by using the backward difference for all values of R(t) equal to or greater than the distance $\sqrt{r_1^2 + L_{max}^2}$
3. Make initial guesses for K and C.

$$\Delta P(t) = \frac{\pi^2}{8}\left\{\theta_1 - \theta_0 \frac{[R(t)^2] + [R(t)]L_{max}}{K}\frac{dr}{dt} - 2r_0^2\right\}\frac{ln[\frac{R(t)[r_0+L_{max}]}{r_0[R(t)+L_{max}]}]}{L_{max}}$$

$$\Delta P(t) = C - H(t) - L_{max} + \frac{L_{max}}{K}\frac{dh}{dt}$$



4. Solve the following equations for ΔP(t) at each incremental value of t.
5. Minimize the absolute difference between the two solutions foundin Step 4 by adjusting the values of K and C.

$$[H_0 - H(t)]r_1^2 = \frac{\theta_1 - \theta_2}{3}[2[R(t)]^3 + 3[R(t)]^2 L_{max} - L_{max}^3 - 4r_0^3]$$

Parameters for Equations

Θ_0 = volumetric water content of soil before MPD test

Θ_1 = volumetric water content of soil after MPD test

mdpwet4

Date	7/11/2017
Time	7:46 AM
Latitude	35.853346
Longitude	-78.847319
Initial Volumetric Moisture	3.00 %
Final Volumetric Moisture	100.00 %
Cylinder Size	3 Liter

mdpwet4 Results

Map Pin #	1
Test Number	
Ksat - cm/sec	NULL
Ksat - in/hr	NULL
Capillary Pressure C cm	NULL
RMS Error of Regression	NULL

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Readings

#	Time	Head	#	Time	Head
1	0 s	31.2 cm	26	36688 s	25.64 cm
2	3178 s	29.13 cm	27	37078 s	25.51 cm
3	28198 s	29.0 cm	28	37678 s	25.38 cm
4	28528 s	28.75 cm	29	38578 s	25.26 cm
5	28678 s	28.62 cm	30	39268 s	25.13 cm
6	28948 s	28.36 cm	31	39748 s	25.0 cm
7	29278 s	28.09 cm	32	40198 s	24.87 cm
8	29638 s	27.98 cm	33	40558 s	24.74 cm
9	29938 s	27.85 cm	34	41098 s	24.61 cm
10	30358 s	27.71 cm	35	41638 s	24.48 cm
11	30838 s	27.58 cm	36	42178 s	24.35 cm
12	31198 s	27.45 cm	37	42658 s	24.23 cm
13	31558 s	27.32 cm	38	43108 s	24.1 cm
14	31798 s	27.19 cm			
15	32008 s	27.07 cm			
16	32218 s	26.94 cm			
17	32428 s	26.81 cm			
18	32638 s	26.68 cm			
19	32938 s	26.55 cm			
20	33478 s	26.42 cm			
21	34108 s	26.29 cm			
22	34558 s	26.17 cm			
23	35128 s	26.04 cm			
24	35698 s	25.91 cm			
25	36178 s	25.77 cm			

Appendix C: Hydrology of Storm Events for all Dry Ponds

Table A-6: Hydrology of discrete storms for MOV1.

Start Date	Duration	Precipitation	5-min Peak Intensity	Runoff Depth	Peak Runoff	Runoff Volume
	(hr:min)	*(in)*	*(in/hr)*	*(in)*	*(cfs)*	*(cf)*
2/15/2017	4:14	0.36	0.44	0.10	0.47	3714
3/1/2017	8:34	0.85	2.07	0.35	0.65	13190
3/13/2017	28:00	0.80	0.24	0.38	0.43	14047
3/18/2017	14:22	0.20	0.24	0.09	0.26	3322
3/21/2017	1:32	0.13	0.46	0.02	0.36	922
3/28/2017	10:10	0.21	1.80	0.07	0.50	2678
3/31/2017	8:32	0.48	1.28	0.18	0.50	6598
4/3/2017	5:16	0.49	2.94	0.18	0.57	6689
4/5/2017	5:22	0.31	0.33	0.11	0.48	4254
4/21/2017	1:12	0.17	0.9	0.03	0.44	957
4/23/2017	72:02	0.94[1]	0.3	4.39	9.19	163521
5/1/2017	3:20	0.29	0.54	0.06	0.41	2312
5/4/2017	16:22	1.79	3.66	0.98	4.38	36449
5/10/2017	3:54	0.46	1.72	0.13	0.54	4853
5/12/2017	8:42	0.23	0.63	0.06	0.43	2305
5/22/2017	21:22	1.11	1.20	0.35	0.56	13011
5/24/2017	16:28	0.86	2.23	0.45	0.60	16763
5/29/2017	10:14	0.13	0.14	0.03	0.11	1055
6/4/2017	35:02	1.88	3.15	0.70	0.71	25989
6/15/2017	3:42	0.19	0.99	0.03	0.41	1074
6/16/2017	13:52	1.79	4.67	0.64	0.75	23871
6/17/2017	4:54	0.10	0.63	0.06	0.46	2379
6/18/2017	2:28	0.12	0.63	0.05	0.47	1862
6/19/2017	19:02	0.58	1.41	0.36	0.67	13392
6/21/2017	8:48	0.31	0.31	0.14	0.39	5179
6/24/2017	21:30	0.96	3.95	0.47	0.76	17423
6/30/2017	6:30	0.40	1.27	0.08	0.50	3090
7/3/2017	1:28	0.11	0.45	0.01	0.26	544
7/4/2017	9:32	1.19	3.42	0.42	0.80	15628
7/17/2017	1:06	0.12	0.70	0.01	0.10	269
7/23/2017	5:36	0.77	1.40	0.16	0.55	5953
8/7/2017	13:50	0.29	1.02	0.12	0.47	4328
8/8/2017	19:10	0.34	1.02	0.12	0.54	4591
8/11/2017	11:16	0.98	3.00	0.36	0.67	13450
8/13/2017	5:48	0.53	1.76	0.20	0.61	7620
8/14/2017	5:04	0.20	0.66	0.06	0.45	2088
8/23/2017	2:28	0.11	0.42	0.02	0.33	756

8/28/2017	12:22	0.17	0.12	0.06	0.06	2118
9/1/2017	5:02	0.11	0.50	0.02	0.32	872
9/1/2017	19:24	1.38	4.87	0.61	0.86	22581
9/6/2017	2:28	0.42	1.20	-	-	-
9/11/2017	24:44	0.46	0.53	-	-	-

[1]*rain gauge clogged*

Table A-7: Hydrology of discrete storms for MOV2.

Start Date	**Duration** *(hr:min)*	**Precipitation** *(in)*	**5-min Peak Intensity** *(in/hr)*	**Runoff Depth** *(in)*	**Peak Runoff** *(cfs)*	**Inflow Volume** *(cf)*	**Outflow Volume** *(cf)*	**Inflow-Outflow Diff.** *(%)*	**ET** *(cf)*
2/15/2017	5:24	0.36	0.44	0.04	0.19	5[3]	1006	-21833%	0
3/1/2017	6:28	0.85	2.07	0.23	0.35	4014	5224	-30%	0
3/13/2017	24:50	0.80	0.24	0.13	0.17	0[3]	2838	NA	0.00
3/18/2017	10:16	0.20	0.24	0.00	0.02	0[3]	72	NA	0
3/21/2017	1:28	0.13	0.46	0.01	0.07	0[3]	135	NA	0
3/28/2017	9:02	0.21	1.80	0.01	0.24	762	319	58%	0
3/31/2017	9:12	0.48	1.28	0.11	0.26	631	2365	-275%	0
4/3/2017	5:58	0.49	2.94	0.11	0.31	3637	2379	35%	0
4/5/2017	5:32	0.31	0.33	0.05	0.23	1820	1097	40%	0
4/21/2017	1:00	0.17	0.9	0.00	0.03	482	32	93%	0
4/23/2017	70:44	0.94[1]	0.3	3.44	3.87	124384	77411	38%	87
5/1/2017	3:18	0.29	0.54	0.01	0.08	450	299	33%	0
5/4/2017	16:54	1.79	3.66	0.79	0.52	0[3]	17762	NA	48
5/10/2017	2:26	0.46	1.72	0.04	0.25	0[3]	802	NA	0
5/12/2017	8:12	0.23	0.63	0.01	0.10	798	296	63%	0
5/22/2017	21:48	1.11	1.20	0.13	0.29	7599	2857	62%	0.14
5/24/2017	13:44	0.86	2.23	0.20	0.32	7801	4585	41%	2
5/29/2017	10:14	0.13	0.14	0.00	0.00	554	0	100%	0
6/4/2017	32:46	1.88	3.15	0.40	0.38	14868	9078	39%	11
6/15/2017	3:42	0.19	0.99	0.00	0.00	351	0.00	100%	0.00
6/16/2017	8:26	1.78	4.67	0.41	0.37	14898	9179.56	38%	12.81
6/17/2017	1:40	0.10	0.63	0.02	0.17	990	379.39	62%	0.00
6/18/2017	1:08	0.12	0.63	0.01	0.09	581	112.86	81%	0.00
6/19/2017	13:36	0.58	1.41	0.18	0.35	6124	3986.23	35%	3.39
6/20/2017	11:30	0.31	0.31	0.02	0.09	2257	386.90	83%	0.00
6/24/2017	5:36	0.96	3.95	0.28	0.41	7361	6253.00	15%	13.49
6/30/2017	6:12	0.40	1.27	0.01	0.13	1308	172.09	87%	0.00
7/3/2017	1:28	0.11	0.45	0.00	0.00	263	0.00	100%	0.00
7/4/2017	7:36	1.19	3.42	0.30	0.42	8447	6783.12	20%	12.78
7/17/2017	1:06	0.12	0.70	0.00	0.00	121	0.00	100%	0.00
7/23/2017	5:36	0.77	1.40	0.02	0.21	2818	562.44[2]	80%	0.01

8/7/2017	13:50	0.29	1.02	0.00	0.00	782	0.00[2]	100%	0.00
8/8/2017	4:02	0.34	1.02	0.01	0.11	2324	262.44[2]	89%	0.00
8/11/2017	12:18	0.98	3.00	0.16	0.35	7376	3539.50	52%	2.82
8/13/2017	5:20	0.53	1.76	0.06	0.31	2815	1354.37	52%	0.38
8/14/2017	5:14	0.20	0.66	0.00	0.00	942	0.00	100%	0.00
8/23/2017	2:32	0.11	0.42	0.00	0.00	87	0.00	100%	0.00
8/28/2017	12:22	0.17	0.12	0.00	0.00	5	0.00	100%	0.00
9/1/2017	2:00	0.11	0.62	0.00	0.00	14	0.00	100%	0.00
9/1/2017	5:56	1.38	4.87	0.27	0.42	0[3]	6092.96	NA	7.16
9/6/2017	3:12	0.42	1.20	0.00	0.00	0[3]	0.00	NA	0.00
9/11/2017	24:44	0.46	0.53	0.00	0.04	243	87.14	64%	0.00

[1]rain gauge clogged, [2]faulty bubbler, [3]malfunctioned AVM, NA where either inflow or outflow was not recorded

Table A-8: Hydrology of discrete storms for WS.

Start Date	Duration	Precipitation	5-min Peak Intensity	Runoff Depth	Peak Outflow	Inflow Volume	Outflow Volume
	(hr:min)	*(in)*	*(in/hr)*	*(in)*	*(cfs)*	*(cf)*	*(cf)*
2/8/2017	4:52	0.19	0.86	0.00	0.000	0	0
2/15/2017	5:30	0.26	0.23	0.00	0.000	0	0
3/1/2017	15:30	0.59	1.53	0.02	0.127	225	180
3/13/2017	14:32	0.59	0.23	0.00	0.000	0	0
3/17/2017	6:06	0.10	0.23	0.00	0.000	0	0
3/26/2017	13:36	0.22	0.29	0.00	0.000	0	0
3/28/2017	5:06	0.16	1.43	0.00	0.000	0	0
3/30/2017	40:40	0.72	1.34	0.02	0.025	221	150
4/3/2017	18:22	1.26	4.44	0.17	0.868	1489	1432
4/6/2017	10:12	0.75	2.23	0.11	0.198	942	910
4/14/2017	11:26	0.75	2.28	0.07	0.317	591	555
4/19/2017	7:06	0.12	0.18	0.00	0.001	29	7
4/21/2017	24:26	0.54	0.24	0.08	0.300	782	706
4/24/2017	55:30	0.06[1]	0.00	2.06	0.56	17400	17228
4/25/2017	58:34	0.20[1]	0.00	0.14	0.020	1334	1152
4/27/2017	20:28	0.72	1.06	0.08	0.334	903	647
5/1/2017	11:08	0.96	0.88	0.09	0.427	816	745
5/4/2017	25:44	1.55	1.85	0.25	0.748	2167	2096
5/9/2017	7:30	0.16	0.19	0.00	0.000	0	0
5/9/2017	3:38	0.13	0.32	0.00	0.000	0	0
5/12/2017	11:06	0.18	0.47	0.00	0.000	0	0
5/19/2017	0:16	0.11	0.60	0.00	0.000	0	0
5/21/2017	11:16	0.68	0.33	0.00	0.000	0	0
5/23/2017	31:42	4.87	3.52	1.75	1.935	14712	14589
5/24/2017	34:30	0.45	0.72	0.09	0.091	862	755
5/27/2017	11:08	0.64	3.06	0.04	0.201	342	308
5/31/2017	4:16	0.14	0.78	0.00	0.000	17	3
6/5/2017	13:48	0.93[1]	1.92	0.02	0.086	200	180
8/11/2017	0:12	0.13	0.90	0.00	0.000	0[2]	0[2]
8/12/2017	10:30	0.58	3.02	0.00	0.000	60	28
8/14/2017	7:04	0.25	0.49	0.00	0.000	23	1
8/14/2017	12:12	1.87	7.57	0.31	1.841	2598	2560
8/29/2017	2:36	0.20	0.24	0.00	0.000	0	0
8/31/2017	1:22	0.14	0.66	0.00	0.000	0	0
9/1/2017	25:48	1.07	2.03	0.01	0.032	143	84
9/7/2017	0:02	0.12	2.20	0.00	0.000	0	0
9/10/2017	1:02	0.40	2.26	0.00	0.000	0	0
9/11/2017	21:12	1.11	0.56	0.00	0.000	0[2]	0[2]
9/14/2017	8:06	0.20	0.96	0.00	0.000	0	0

10/7/2017	5:46	0.42	0.96	0.00	0.000	0	0
10/8/2017	8:34	0.24	0.30	0.00	0.000	0	0
10/13/2017	1:56	0.10	0.17	0.00	0.000	0	0
10/23/2017	15:52	1.48	3.47	0.08	0.294	644	628
LOSS OF STORM DATA DUE TO EQUIPMENT MALFUNCTION – 10/24/2017 – 1/10/2018							
1/12/2018	11:10	0.53	0.66	0.00	0.000	0	0
1/18/2018	5:26	0.40	0.24	0.00	0.000	0	0
1/27/2018	42:46	1.12	0.24	0.05	0.030	452	386
2/4/2018	28:40	0.85	0.33	0.10	0.179	917	838
2/7/2018	27:16	1.37	0.63	0.32	0.248	2731	2646
2/10/2018	13:44	0.21	0.19	0.00	0.000	0	0
2/10/2018	44:42	0.78	1.40	0.10	0.186	982	843
2/28/2018	21:18	0.48	0.19	0.00	0.00	0	0
3/6/2018	23:02	0.66	0.31	0.00	0.01	90	41
3/11/2018	46:58	0.83	0.25	0.02	0.02	234	151
3/17/2018	2:10	0.16	0.30	0.00	0.00	0	0
3/19/2018	21:28	0.54	1.14	0.02	0.05	215	159
3/20/2018	40:40	0.34	0.60	0.01	0.00	185	59

[1]*Rain gauge clogged,* [2]*bubbler came unattached*

Appendix D: Water Quality Samples of all Dry Ponds

Table A-9: Water quality of sampled events in MOV1_IN1.

Start Date	TKN (mg/L)	$NO_{2,3}$-N (mg/L)	TAN (mg/L)	TN (mg/L)	ON (mg/L)	TP (mg/L)	OPO_4^{3-} (mg/L)	TSS (mg/L)
3/1/2017	3.38*	1.17	0.82	4.55	2.56	0.41	0.16	114.5
3/13/2017	1.48	1.64	0.61	3.12	0.87	0.25	0.18	9.3
4/3/2017	3.08	0.46	0.24	3.54	2.84	0.49	0.07	249.6
4/5/2017	9.09	0.68	1.03	9.76	8.05	0.22	0.10	19.2
5/4/2017	1.15	0.64	0.23	1.79	0.93	0.35	0.10	163.9
5/10/2017	1.43	0.76	0.48	2.19	0.94	0.23	0.08	86.5
5/12/2017	-	-	-	-	-	-	-	-
5/22/2017	9.23	2.62	1.71	11.85	7.52	0.59	0.12	19.1
5/24/2017	1.41	0.85	0.30	2.27	1.11	0.18	0.07	34.0
6/4/2017	1.47	0.60	0.51	2.07	0.96	0.20	0.12	20.4
6/19/2017	1.07	0.35	0.12	1.43	0.96	0.20	0.11	41.2
6/24/2017	1.19	0.33	0.15	1.52	1.04	0.30	0.20	80.4
7/4/2017	0.57	0.25	0.06	0.82	0.51	0.15	0.06	37.3
7/23/2017	0.71	0.52	0.34	1.24	0.37	0.07	0.02	10.1
8/8/2017	0.704	0.378	0.130	1.082	0.574	0.075	0.029	17.9
8/11/2017	0.95	0.90	0.21	1.85	0.74	0.16	0.08	53.5
8/13/2017	0.71	0.63	0.03	1.34	0.68	0.09	0.03	17.4

**sample spike out of limit*

Table A-10: Storm-based water quality samples in MOV1_IN2.

Start Date	TKN (mg/L)	$NO_{2,3}$-N (mg/L)	TAN (mg/L)	TN (mg/L)	ON (mg/L)	TP (mg/L)	OPO_4^{3-} (mg/L)	TSS (mg/L)
3/1/2017	2.54	0.81	0.29	3.35	2.25	0.57	0.15	178.2
3/13/2017	3.60	1.23	1.16	4.84	2.44	0.31	0.19	27.7
4/3/2017	2.60	0.40	0.23	3.00	2.37	0.36	0.06	171.8
4/5/2017	2.79	0.65	0.59	3.44	2.20	0.24	0.12	42.2
5/4/2017	-	-	-	-	-	-	-	-
5/10/2017	-	-	-	-	-	-	-	-
5/12/2017	0.98	0.61	0.22	1.59	0.75	0.13	0.05	29.3
5/22/2017*	2.62	1.16	0.77	3.78	1.85	0.25	0.19	17.6
5/24/2017	1.72	0.93	0.27	2.66	1.45	0.25	0.17	22.7
6/4/2017	1.73	0.65	0.79	2.38	0.94	1.35	1.31	23.3
6/19/2017	1.52	0.45	0.14	1.97	1.38	0.40	0.27	57.7
6/24/2017	2.13	0.70	0.19	2.83	1.94	0.57	0.25	145.6
7/4/2017	0.77	0.32	0.09	1.09	0.68	0.23	0.12	56.8
7/23/2017	1.94	0.81	0.73	2.75	1.21	0.24	0.11	57.8
8/8/2017	0.97	0.48	0.09	1.45	0.88	0.19	0.05	38.9
8/11/2017	1.43	0.91	0.23	2.35	1.20	0.21	0.10	58.0
8/13/2017	1.99	0.77	0.42	2.75	1.57	0.24	0.14	32.9

**Overprediction of level led to oversampling. Samples from this storm were discarded*

Table A-11: Storm-based water quality samples in MOV1_OUT.

Start Date	TKN (mg/L)	$NO_{2,3}$-N (mg/L)	TAN (mg/L)	TN (mg/L)	ON (mg/L)	TP (mg/L)	OPO_4^{3-} (mg/L)	TSS (mg/L)
3/1/2017	2.62	1.18	0.60	3.80	2.02	0.42	0.21	63.7
3/13/2017	1.57	1.46	0.54	3.03	1.03	0.25	0.16	24.0
4/3/2017	5.45*	0.69	0.47	6.14	4.97*	0.78	0.09	516.7*
4/5/2017	8.06	0.69	1.01	8.75	7.05	0.34	0.09	128.2
5/4/2017	2.16	0.52	0.27	2.68	1.89	0.44	0.10	210.1
5/10/2017	3.75	0.82	0.31	4.57	3.45	0.97	0.09	571.3
5/12/2017	3.49	0.58	0.28	4.07	3.21	1.04	0.05	662.6
5/22/2017	2.77	1.18	0.53	3.95	2.24	0.29	0.17	36.5
5/24/2017	1.90	0.97	0.27	2.86	1.62	0.26	0.15	31.5
6/4/2017	1.99	0.53	0.27	2.52	1.72	0.40	0.23	69.9
6/19/2017	1.88	0.38	0.10	2.25	1.78	0.42	0.14	143.5
6/24/2017	1.75	0.32	0.16	2.08	1.60	0.61	0.37	82.5
7/4/2017	1.31	0.32	0.06	1.63	1.25	0.34	0.18	54.4
7/23/2017	2.22*	0.17	0.05	2.39	2.17 *	0.53	0.26	337.8*
8/8/2017	1.71	0.27	0.03	1.98	1.67	0.62	0.53	17.6
8/11/2017	1.74	1.11	0.16	2.85	1.57	0.41	0.29	45.5
8/13/2017	1.96	1.17	0.17	3.13	1.79	0.37	0.26	51.9

**Omitted due to sediment build up on sampler*

Table A-12: Storm-based water quality samples of MOV2_IN.

Start Date	TKN (mg/L)	$NO_{2,3}$-N (mg/L)	TAN (mg/L)	TN (mg/L)	ON (mg/L)	TP (mg/L)	OPO_4^{3-} (mg/L)	TSS (mg/L)
3/1/2017	14.95	3.59	3.15	18.54	11.80	0.66	0.40	94
3/13/2017	-	-	-	-	-	-	-	-
4/3/2017	2.13	2.25	0.28	4.38	1.86	0.49	0.27	61
4/5/2017	1.63	1.75	0.12	3.38	1.50	0.30	0.15	44
5/4/2017	-	-	-	-	-	-	-	-
5/22/2017	1.49	1.28	0.42	2.77	1.07	0.22	0.16	13
5/24/2017	4.18	1.19	1.73	5.37	2.45	1.55	1.45	62
6/4/2017	1.44	0.95	0.28	2.39	1.16	0.37	0.29	17
6/19/2017	1.21	0.62	0.11	1.83	1.09	0.37	0.28	21
6/24/2017	1.53	0.99	0.25	2.52	1.28	0.52	0.34	70
7/4/2017	1.42	0.62	0.09	2.05	1.34	0.34	0.21	68
7/23/2017	1.13	0.70	0.41	1.83	0.72	0.13	0.07	42
8/11/2017	1.83	1.46	0.56	3.29	1.27	0.63	0.51	57
8/13/2017	1.07	0.81	0.11	1.88	0.96	0.36	0.29	37

Table A-13: Storm-based water quality samples of MOV2_OUT.

Start Date	TKN (mg/L)	$NO_{2,3}$-N (mg/L)	TAN (mg/L)	TN (mg/L)	ON (mg/L)	TP (mg/L)	OPO_4^{3-} (mg/L)	TSS (mg/L)
3/1/2017	14.88	3.79	3.50	18.67	11.38	0.60	0.41	46
3/13/2017	-	-	-	-	-	-	-	-
4/3/2017	-	-	-	-	-	-	-	-
4/5/2017	1.37	1.66	0.10	3.03	1.28	0.26	0.17	17
5/4/2017	1.33	0.48	0.23	1.81	1.10	0.46	0.22	92
5/22/2017	1.59	1.19	0.11	2.79	1.48	0.33	0.24	23
5/24/2017	-	-	-	-	-	-	-	-
6/4/2017	1.01	1.06	0.11	2.07	0.90	0.33	0.25	20
6/19/2017	1.08	0.69	0.07	1.77	1.01	0.31	0.23	15
6/24/2017	1.55	0.61	0.16	2.17	1.40	0.66	0.51	29
7/4/2017	0.92	0.46	0.07	1.37	0.84	0.33	0.23	18
7/23/2017*	0.81**	0.65	0.02	1.45	0.79	0.25	0.16	15
8/11/2017	1.37	0.96	0.13	2.32	1.23	0.69	0.60	12
8/13/2017	1.16	0.75	0.10	1.91	1.06	0.42	0.38	18

**Malfunction of bubbler led to underprediction of outflow volume and a false EMC sample. Samples from this event were not included.*
***analyzed outside of holding time*

Table A- 14: Storm-based water quality samples of WS_IN.

Start Date	TKN (mg/L)	$NO_{2,3}$-N (mg/L)	TAN (mg/L)	TN (mg/L)	ON (mg/L)	TP (mg/L)	OPO_4^{3-} (mg/L)	TSS (mg/L)
3/30/2017	-	-	-	-	-	-	-	-
4/3/2017	4.84	0.20	0.18	5.05	4.66	0.86	0.11	270
4/6/2017	2.19	0.21	0.11	2.40	2.07	0.51	0.09	193
5/23/2017*	1.30	0.16	0.14	1.47	1.16	0.58	0.22	107
6/5/2017	1.32	0.39	0.18	1.71	1.14	0.85	0.66	33
8/14/2017	1.87	0.26	0.32	2.13	1.55	0.93	0.66	262
10/23/2017	2.21**	0.41	0.20	2.62	2.01	1.35	0.60	384
1/27/2018	1.28	0.22	0.18	1.50	1.10	0.34	0.18	37
2/4/2018	2.23	0.11	0.26	2.34	1.97	0.52	0.18	107
2/7/2018	1.61	0.16	0.23	1.77	1.38	0.46	0.12	75
2/10/2018	1.93	0.13	0.15	2.05	1.78	0.44	0.09	101
3/19/2018	1.64	0.19	0.09	1.83	1.55	0.42	0.12***	103

**Approximately 59% of the storm was sampled. Storm was not used in analysis.*
***Acc QC recover low at 88%*
****exceeded holding time*

Table A-15: Storm-based water quality samples of WS_OUT.

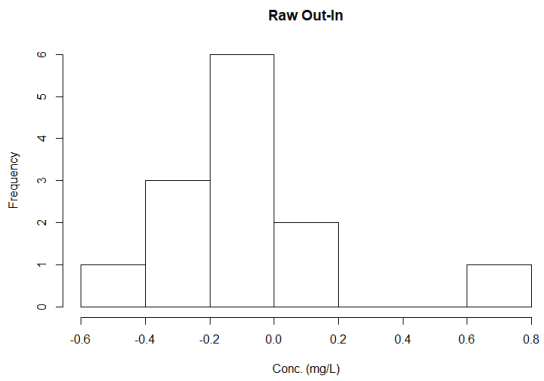
Start Date	**TKN** *(mg/L)*	**$NO_{2,3}$-N** *(mg/L)*	**TAN** *(mg/L)*	**TN** *(mg/L)*	**ON** *(mg/L)*	**TP** *(mg/L)*	**OPO_4^{3-}** *(mg/L)*	**TSS** *(mg/L)*
3/30/2017	1.63	0.12	0.19	1.75	1.44	0.47	0.10	51
4/3/2017	1.45	0.14	0.07	1.60	1.39	0.66	0.09	161
4/6/2017	1.20	0.14	0.08	1.33	1.12	0.44	0.06	77
5/23/2017*	1.47	0.09	0.11	1.56	1.36	0.47	0.20	108
6/5/2017	1.26	0.18	0.22	1.44	1.04	0.34	0.20	27
8/14/2017	1.24	0.16	0.18	1.40	1.06	0.58	0.23	182
10/23/2017	1.28**	0.19	0.13	1.47	1.15	0.82	0.31	199
1/27/2018	0.86	0.18	0.12	1.04	0.74	0.38	0.14	13***
2/4/2018	1.06	0.10	0.21	1.16	0.85	0.48	0.09	55
2/7/2018	1.04	0.13	0.15	1.17	0.89	0.42	0.46	44
2/10/2018	1.05	0.09	0.12	1.13	0.93	0.35	0.09	50
3/19/2018	1.01	0.12	0.10	1.13	0.91	0.28	0.09****	45

**Approximately 27% of the storm was sampled. Storm was not used in analysis.*
***Acc QC recover low at 88%*
****less than PQL, used 0.5*PQL*
*****exceeded holding time*

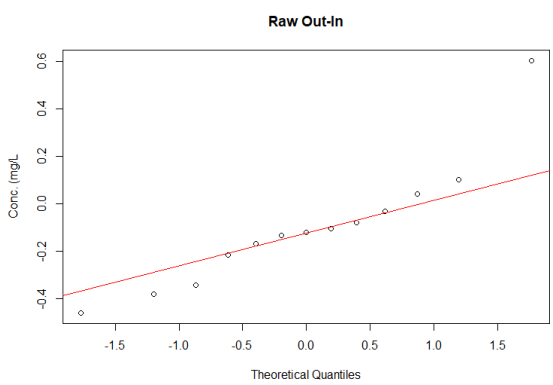
Appendix E: Sample R Code (based off code from Koryto (2016))

MOV1 (wet) Concentrations

```
#Load needed packages
library("reshape")
library("ggplot2")
library("plyr")
library("dplyr")
library("nortest")
library("lawstat")
library("PMCMR")

#set up data frame
wet_emc <- read.csv("wet_emc.csv",header=TRUE)
wet_emc$location <- as.factor(wet_emc$location)
wet_emc$location <- factor(wet_emc$location, levels
= c('wet.in1', 'wet.in2', 'wet.out'),ordered=TRUE)
sapply(wet_emc,class)

#Create subset to omit 5/22/17 storm
wet_emc=subset(wet_emc, wet_emc$date!='5/22/2017',
drop=TRUE)

#TP
#Extract just TP dataset and remove NAs
tp.w_emc <-
wet_emc[c("date","event","location","tp")]
sapply(tp.w_emc,class)
tp.w_emc.narm <-na.omit(tp.w_emc)

#Split dataset into In and Out
emc.in1.tp.wet=subset(tp.w_emc,
tp.w_emc$location=='wet.in1')
emc.in2.tp.wet=subset(tp.w_emc,tp.w_emc$location=='
wet.in2')
emc.out.tp.wet=subset(tp.w_emc,tp.w_emc$location=='
wet.out')

#summary statistics
summary(emc.in1.tp.wet$tp)
summary(emc.in2.tp.wet$tp)
summary(emc.out.tp.wet$tp)

#Create Paired Datasets: In to Out
#Merge In and Out Datasets and create paired
differences column w/ NA removed
emc.in1.out.tp.merge.wet <-
merge(emc.in1.tp.wet,emc.out.tp.wet,by="event")
in.out.tp.merge.wet <-
merge(emc.in1.out.tp.merge.wet,emc.in2.tp.wet,by="e
vent")
in.out.tp.merge.wet$tp.z <-
0.3*in.out.tp.merge.wet$tp.x +
0.7*in.out.tp.merge.wet$tp
in.out.tp.merge.wet$paired <-
in.out.tp.merge.wet$tp.z - in.out.tp.merge.wet$tp.y
in.out.tp.merge.wet$log10.tp.y <-
log10(in.out.tp.merge.wet$tp.y)
in.out.tp.merge.wet$log10.tp.z <-
log10(in.out.tp.merge.wet$tp.z)
in.out.tp.wet <- na.omit(in.out.tp.merge.wet)
in.out.tp.wet$log10.paired <-
log10(1+in.out.tp.wet$paired-
min(in.out.tp.wet$paired))
in.out.tp.wet$re <- (in.out.tp.wet$tp.z -
in.out.tp.wet$tp.y)/in.out.tp.wet$tp.z*100
in.out.tp.wet$log10.re <- (log(in.out.tp.wet$re+1-
min(in.out.tp.wet$re)))

#Normality tests
shapiro.test(in.out.tp.wet$paired)# p = 0.08225
lillie.test(in.out.tp.wet$paired)# p = 0.4074
ad.test(in.out.tp.wet$paired)# p = 0.1247

#Significance test In to Out, tp,
reduction = -0.0997, p = 0.2023 not significant
t.test(in.out.tp.wet$tp.z, in.out.tp.wet$tp.y,
paired=TRUE)


#Concentration TP Assumption assessment

#Histogram
hist(in.out.tp.wet$paired,xlab="Conc. (mg/L)",
main="Raw Out-In")
```



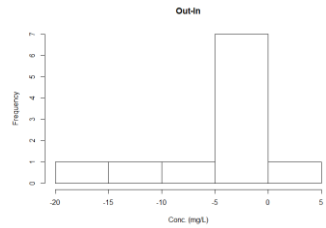

```
#QQ Normality plot - In to Out
qqnorm(in.out.tp.wet$paired,ylab="Conc. (mg/L",
main="Raw Out-In")
qqline(in.out.tp.wet$paired, col="red")
```



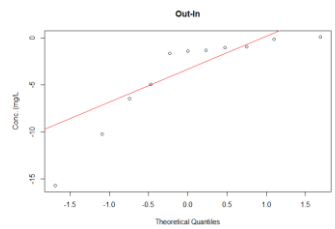
WS Load Removal Comparisons

```
#load needed packages
library("reshape")
library("ggplot2")
library("plyr")
library("dplyr")
library("nortest")
library("lawstat")
library("PMCMR")
library("BSDA")

#set up data frame
ws_load <- read.csv("ws_load.csv",header=TRUE)
ws_load$location <- as.factor(ws_load$location)
ws_load$location <- factor(ws_load$location, levels
= c('ws.in','ws.out'),ordered=TRUE)
sapply(ws_load,class)

#TAN
#Extract just TAN dataset and remove NAs
tan_load <-
ws_load[c("date","event","location","tan")]
sapply(tan_load,class)
tan_load.narm <-na.omit(tan_load)

#Split dataset into In and Out
load.in.tan.ws=subset(tan_load,
tan_load$location=='ws.in')
load.out.tan.ws=subset(tan_load,tan_load$location==
'ws.out')

#summary statistics
summary(load.in.tan.ws$tan)
sum(load.in.tan.ws$tan)
summary(load.out.tan.ws$tan)
sum(load.out.tan.ws$tan)

#Create Paired Datasets: In to Out
#Merge In and Out Datasets and create paired
differences column w/ NA removed
in.out.tan.merge.ws <-
merge(load.in.tan.ws,load.out.tan.ws,by="event")
in.out.tan.merge.ws$paired <-
in.out.tan.merge.ws$tan.y -
in.out.tan.merge.ws$tan.x
in.out.tan.merge.ws$log10.tan.y <-
log10(in.out.tan.merge.ws$tan.y)
in.out.tan.merge.ws$log10.tan.x <-
log10(in.out.tan.merge.ws$tan.x)
in.out.tan.ws <- na.omit(in.out.tan.merge.ws)
in.out.tan.ws$re <- (in.out.tan.ws$tan.x -
in.out.tan.ws$tan.y)/in.out.tan.ws$tan.x*100
in.out.tan.ws$log10.paired <-
log10(1+in.out.tan.ws$paired-
min(in.out.tan.ws$paired))
in.out.tan.ws$log10.re <- log(in.out.tan.ws$re+1-
min(in.out.tan.ws$re))

#Load TAN assessment
#Normality tests - not normal
shapiro.test(in.out.tan.ws$paired)# p = 0.00053
lillie.test(in.out.tan.ws$paired)# p = 0.0033
ad.test(in.out.tan.ws$paired)# p = 0.0059

##Log transform
#Log Normality tests - not log normal
shapiro.test(in.out.tan.ws$log10.paired)# p < 0.001
lillie.test(in.out.tan.ws$log10.paired)# p = 0.006
ad.test(in.out.tan.ws$log10.paired)# p < 0.001

#Symmetry test p = 0.02, not symmetrical
symmetry.test(in.out.tan.ws$paired)

SIGN.test(in.out.tan.ws$tan.x,in.out.tan.ws$tan.y)
#p = 0.01172, significant difference
```

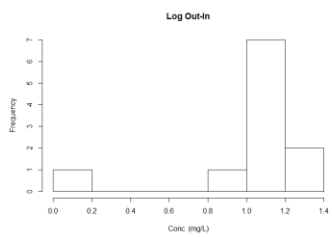
```
#Histogram and QQ Normality plot - raw - In to Out
hist(in.out.tan.ws$paired,xlab="Conc. (mg/L)",
main="Out-In")
```



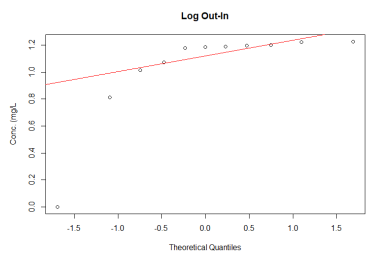

```
qqnorm(in.out.tan.ws$paired,ylab="Conc. (mg/L",
main="Out-In")
qqline(in.out.tan.ws$paired, col="red")
```



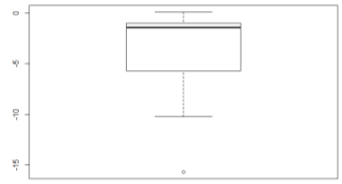

```
#Histogram and QQ Normality plot - Log - In to Out
hist(in.out.tan.ws$log10.paired,xlab="Conc.
(mg/L)", main="Log Out-In")
```




```
qqnorm(in.out.tan.ws$log10.paired,ylab="Conc.
(mg/L", main="Log Out-In")
qqline(in.out.tan.ws$log10.paired, col="red")
```




```
#Boxplot symmetry test for Wilcoxon Signed Rank
Test
boxplot(in.out.tan.ws$paired)
```



Appendix F: MOV1 and WS Wetland Conversion Design Plans

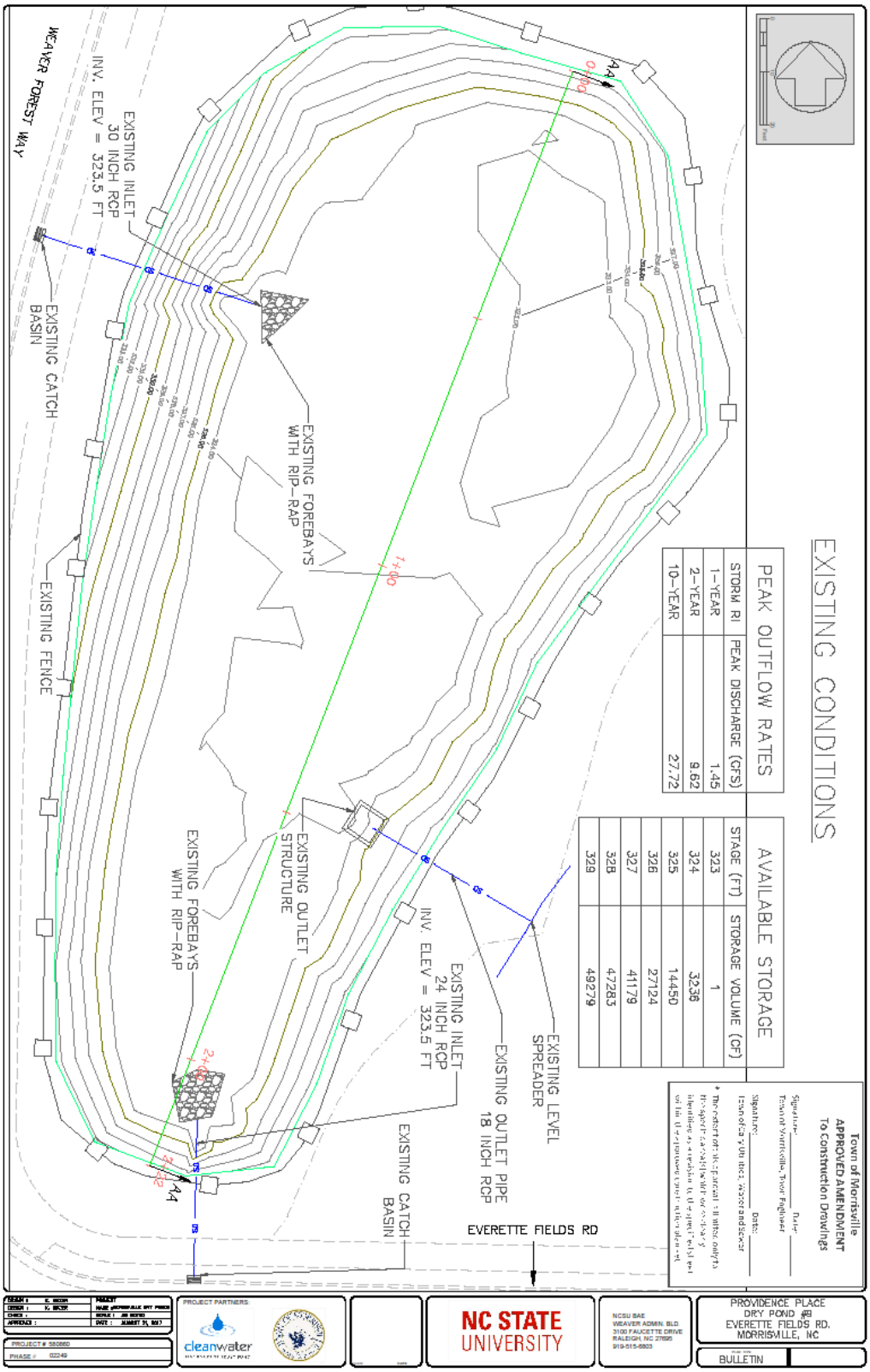


Figure A-3: Dry pond original conditions.

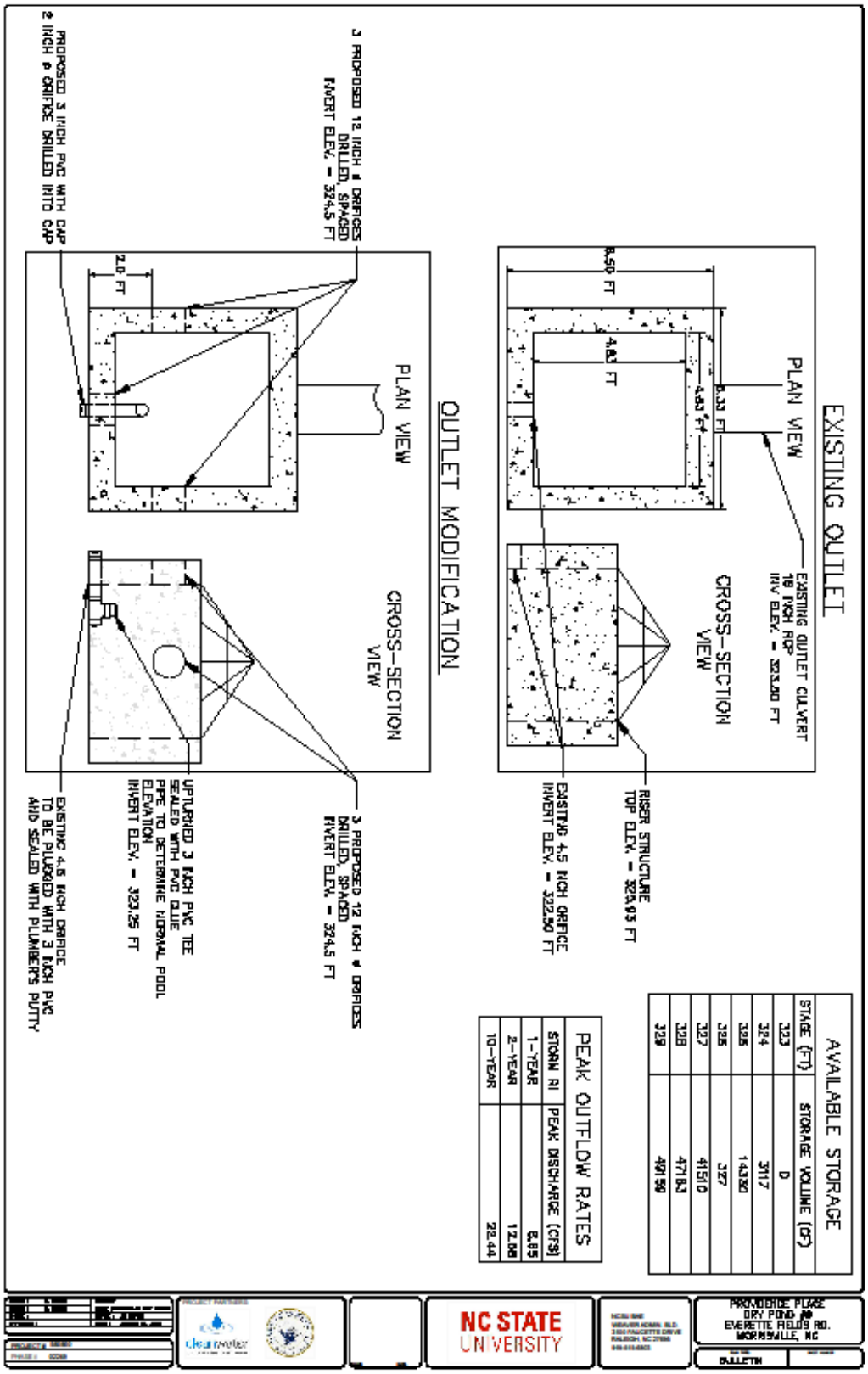


Figure A-4: Outlet structure modification.

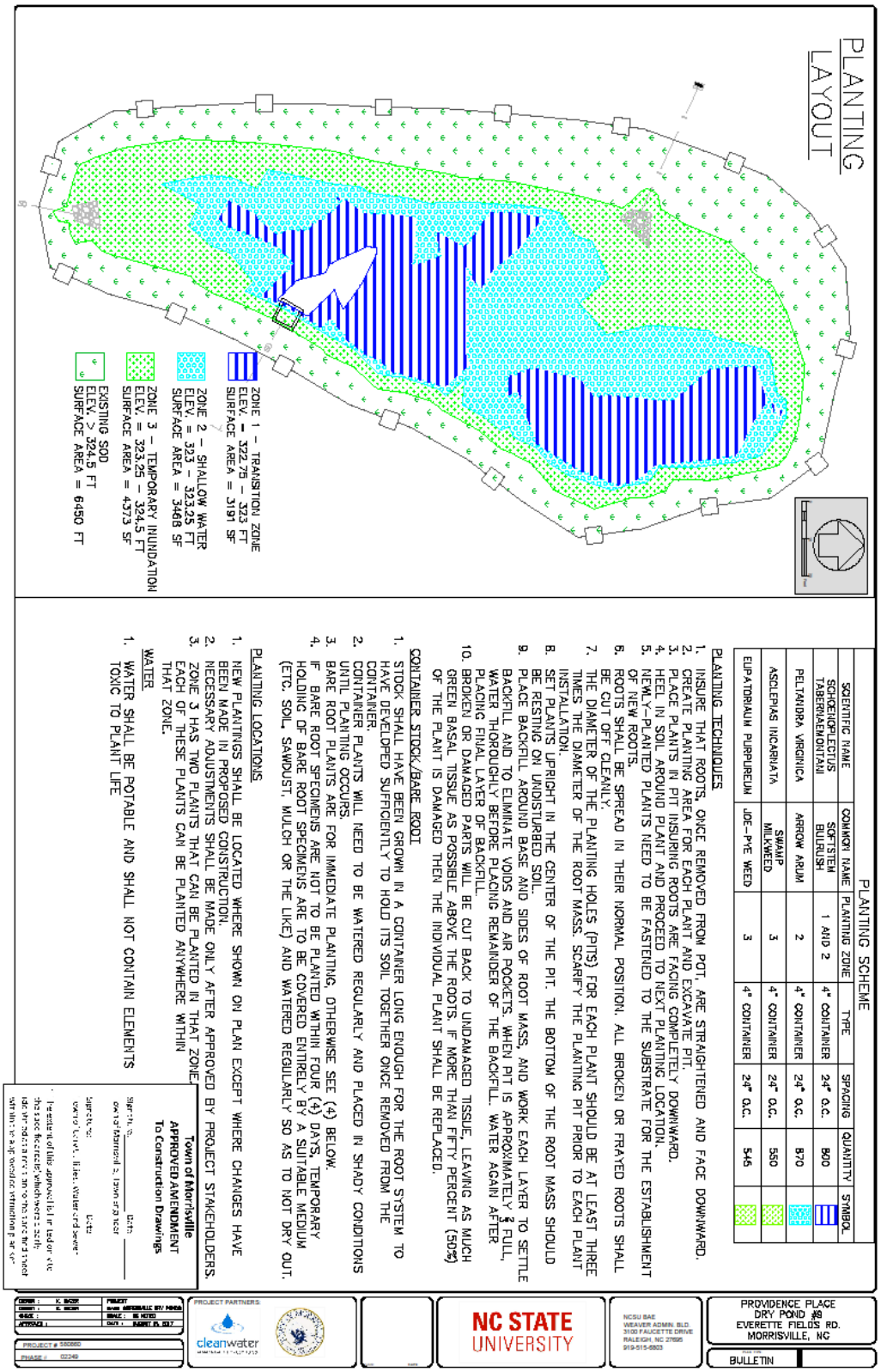


Figure A-5: Planting plan for the MOV1 wetland conversion.

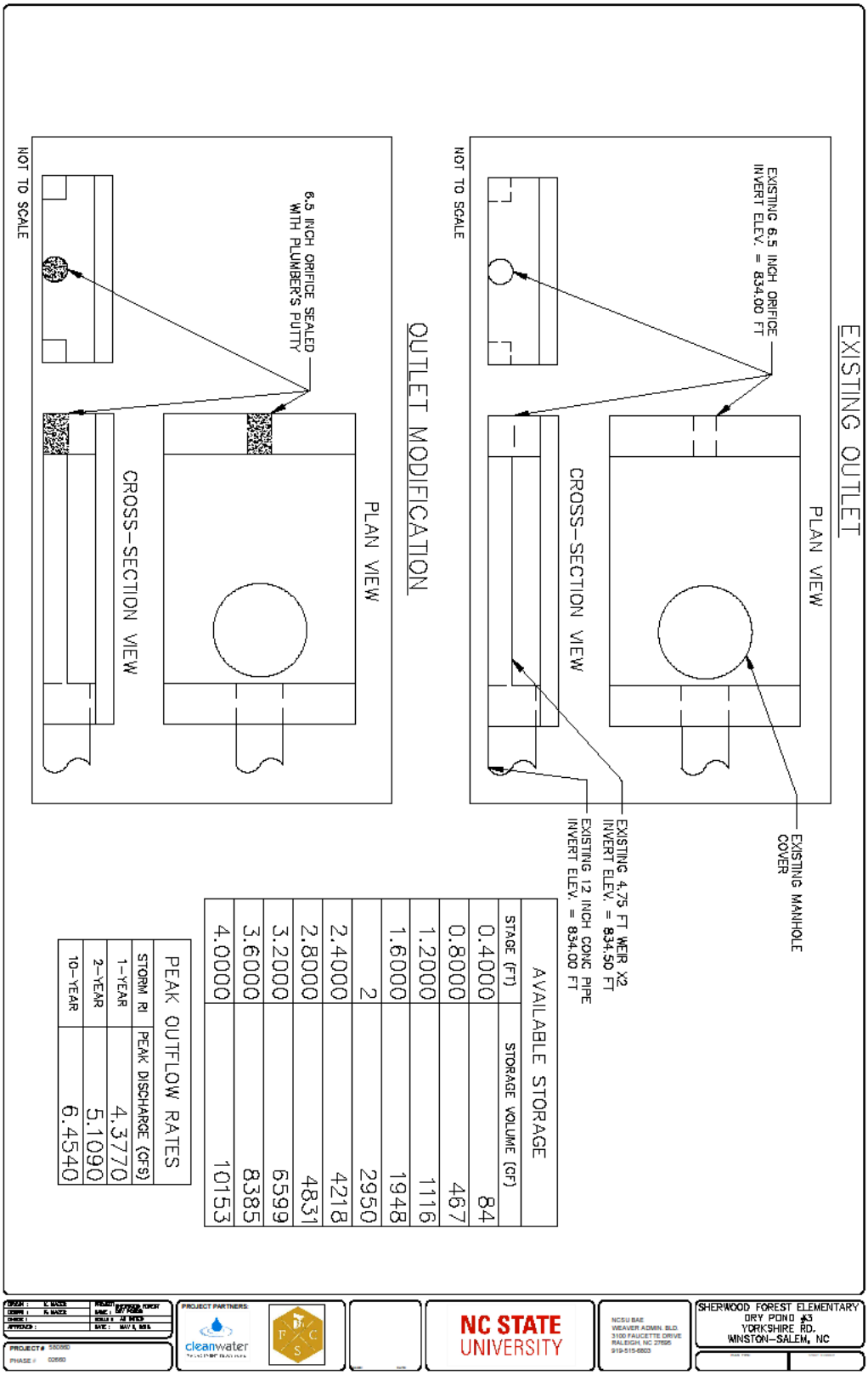


Figure A-6: WS outlet modification for wetland conversion.

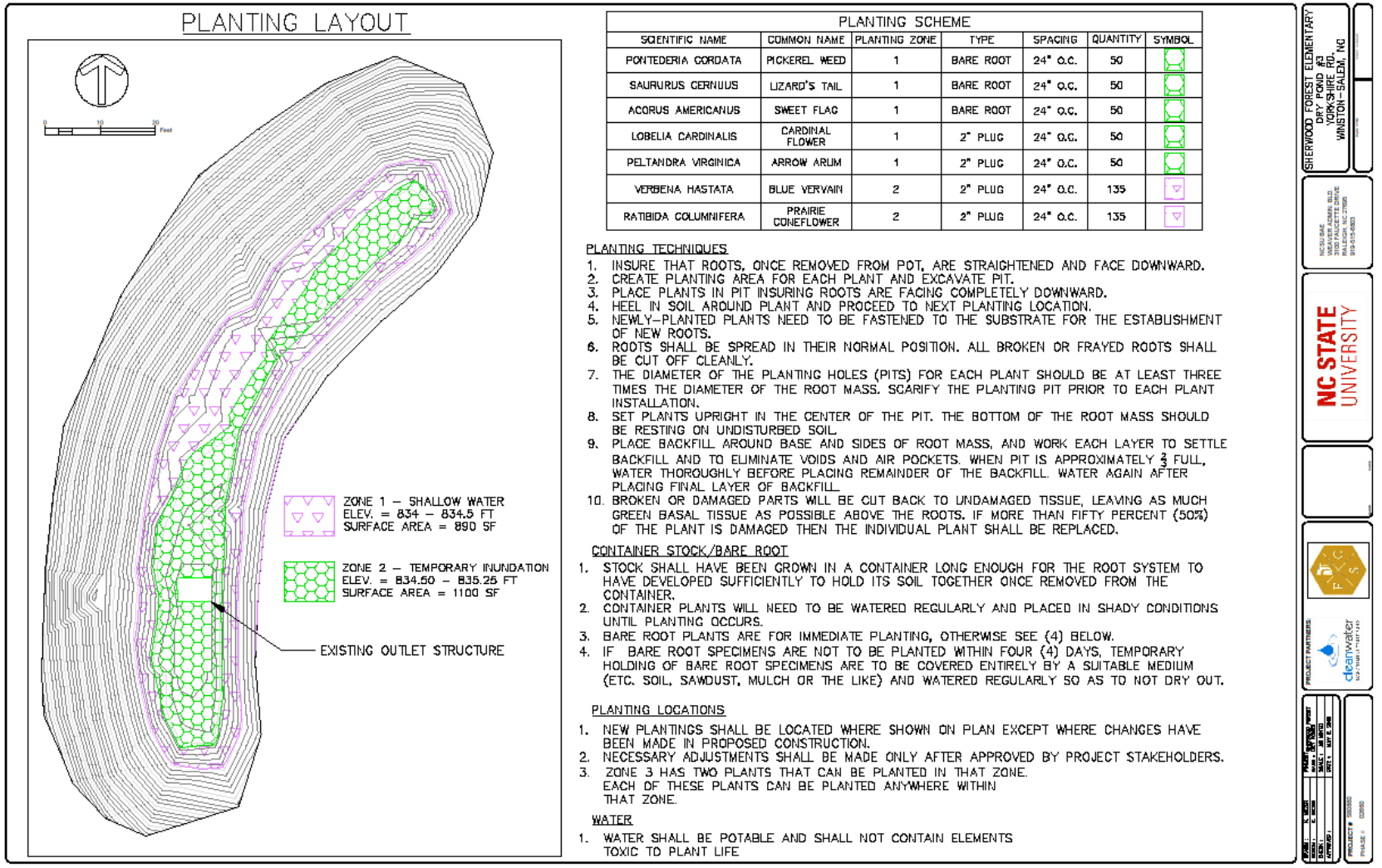


Figure A-7: Planting plan for WS wetland conversion.

Table A- 16: Hydraflow from AutoCAD peak discharge output for retrofit designs.

Pond	**Condition**	**Peak Outflow**			
		1-yr *L/s*	2-yr *L/s*	5-yr *L/s*	10-yr *L/s*
MOV1	SCS Runoff	740.5	982.6	-	1617.5
	Calibration	41.7	274.1	-	805.3
	Treatment	244.9	356.2	-	635.4
WS	SCS runoff	189.1	244.5	325.4	389.1
	Calibration	123.9	144.3	165.9	184.0
	Treatment	168.8	207.0	256.0	288.3

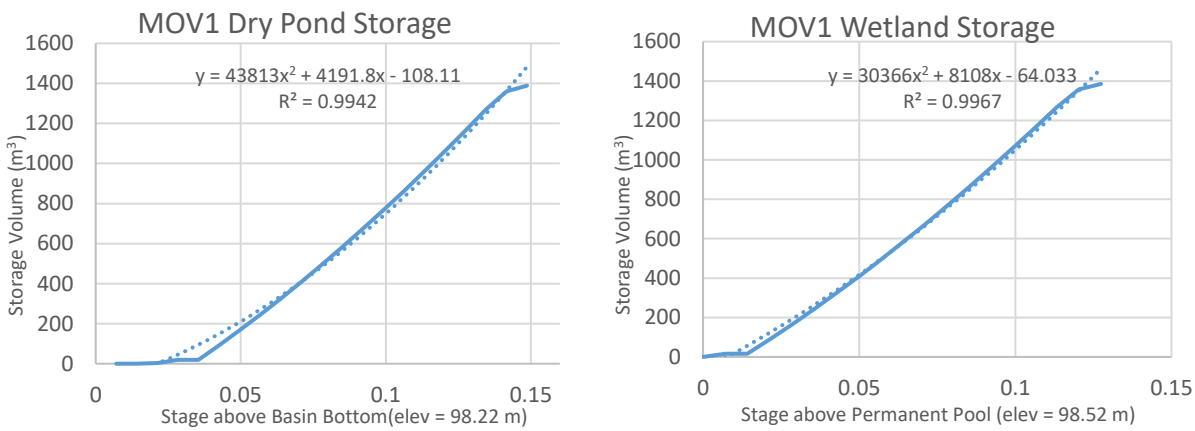


Figure A-8: MOV1 available storage volume before and after wetland conversion retrofit.

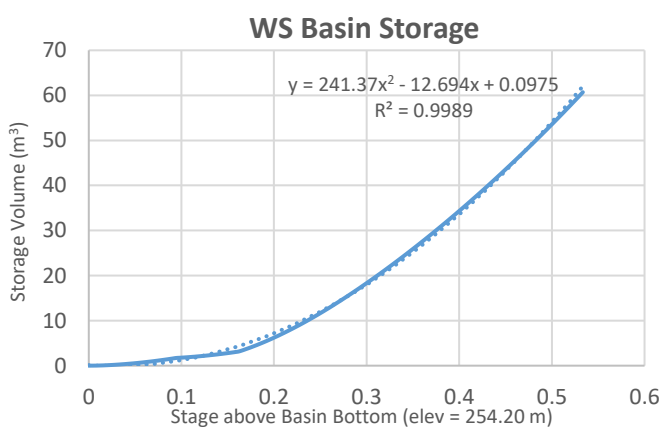


Figure A-9: Available storage volume of WS during both phases of monitoring.

Appendix G: Soil Sampling

To test whether nitrogen concentration could be attainable in this environment, soil samples were taken from the dry pond in pre-conversion conditions. It was assumed that organic matter present in the dry pond would still remain in the soil upon conversion. Also, after planting, it is likely that organic matter would build up in the wetland from natural decomposition processes occurring. Soil samples were taken from six locations within the dry pond, three each along the flow path between each inlet and the outlet. Samples were taken from the surface to a depth of 0.1 m. A hammer corer with cylindrical cores was used to extract each samples. Upon extraction, cores were cut flush with the core sample tube and capped with polyethylene lids for transport. Two samples were taken from each location and composited for an average sample measurement.

In the lab, bulk density was measured by weighing the samples after they were dried in an oven at 105°C for 24 hours. Bulk volume was taken to be the inner volume of the aluminum core minus the depth that the sample had shrunken away from the edge. To only include soil and not large pieces of debris or gravel present in the sample, the ground samples were sieved to only include soil particles <2 mm. The mass of the sieved sample was recorded to determine fine bulk density.

It was at this point that samples from each location were composited to create one average soil sample for each site. Approximately 150 grams of the composite sample was analyzed for total percent carbon and available phosphorus in the Environmental Analysis Lab in the Department of Biological and Agricultural Engineering at NC State.

Total organic carbon (TOC) was measured by combusting samples at 1350 C and then measuring C using infrared detection. Molybdate-antimony-absorbic acid colorimetry was used to water extract available P utilizing an Autoanalyzer System.

Table A-17: Results of soil sample analysis from WS and MOV1.

Date	**Sample ID**	**Available P**	**P-Index**	**pH**	**TOC**
		mg/kg			*%*
11/8/2017	WS1	1.13	1.36	3.62	0.565
	WS2	0.72	0.86	3.64	0.649
	WS3	1.02	1.22	3.70	0.474
	MOV1-1	1.03	1.24	4.88	1.628
	MOV1-2	1.15	1.38	6.67	0.836
	MOV1-3	0.67	0.80	5.86	0.657
	MOV1-4	1.05	1.25	5.29	0.848
	MOV1-5	1.04	1.25	5.47	0.795
	MOV1-6	1.20	1.43	7.14	1.001

Figure A-10: Soil samples were taken at six locations in MOV1 and in three locations in WS, as shown.

Appendix H: ANCOVA and R Code for the Dry Pond Retrofit

In the Morrisville monitoring, there were several monitoring stations. Often, changes in water quality due to watershed alterations are measured in one of two ways: a paired watershed study or an upstream/downstream design. However, in this study, there was opportunity for both types of monitoring designs. Instead of choosing one method and negating the value of the other method, both designs were combined into one, where both MOV2_OUT and MOV1_IN were used as covariates in the initial model. In many cases, neither variable was significantly correlated to MOV1_OUT, rendering them useless as covariates. *Table A.3* summarizes the relationships that were found significant between explanatory variables and the response variable (MOV1_OUT). It was with these relationships that ANCOVA was run.

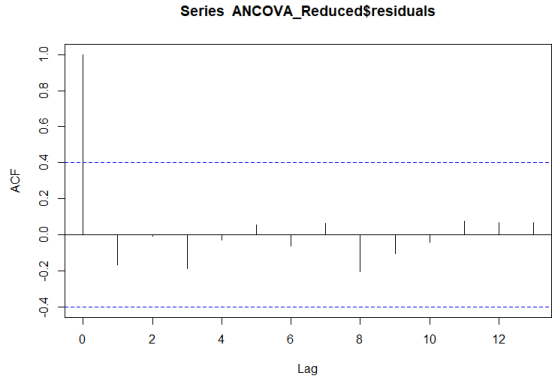
Table A-18: Significant Relationships between Explanatory Values and MOV1-OUT.

Pollutant	**Model**	**p-value (MOV1, MOV2)**
TSS EMC	No significant relationship present	0.5534, 0.1707
TP EMC	MOV2 is significantly correlated to MOV1_OUT	0.0007
Ortho-P EMC	MOV1_IN and MOV2 are significantly correlated to MOV1_OUT	0.0106, <0.0001
TN EMC	MOV1_IN is significantly correlated to MOV1-OUT	0.0006
TKN EMC	MOV1_IN is significantly correlated to MOV1_OUT	0.0188
ON EMC	MOV1_IN is significantly correlated to MOV1_OUT	0.0281
TAN EMC	MOV1_IN and MOV2 are significantly correlated to MOV1_OUT	0.0019, 0.0351
$NO_{2,3}$-N EMC	MOV1_IN is significantly correlated to MOV1_OUT	0.0046
TSS Loads	MOV2 is significantly correlated to MOV1_OUT	0.0004
TP Loads	MOV2 is significantly correlated to MOV1_OUT	<0.0001
Ortho-P Loads	MOV1_IN and MOV2 are significantly correlated to MOV1_OUT	0.0025, <0.0001
TN Loads	MOV1_IN is significantly correlated to MOV1_OUT	<0.0001
TKN Loads	MOV1_IN is significantly correlated to MOV1_OUT	<0.0001
ON Loads	MOV1_IN is significantly correlated to MOV1_OUT	<0.0001
TAN Loads	MOV1_IN and MOV2 are significantly correlated to MOV1_OUT	0.0002, 0.0022
$NO_{2,3}$-N Loads	MOV1_IN and MOV2 are significantly correlated to MOV1_OUT	0.0007, 0.0175

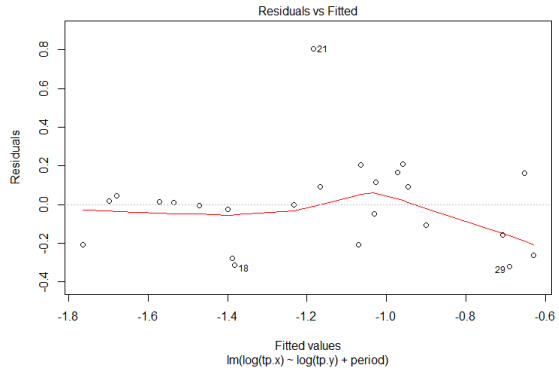
Sample R-Code

```
#Load needed packages
library(ggplot2)
library(readr)
library(dplyr)
library(lsmeans)

#Load pollutant concentration data
mov <- subset(read.csv('all2.csv', header = TRUE),
date != "5/22/2017")
mov.tp <- select(mov, date, tp, location, period)

#Variable isolation: MOV1
mov1.out <- subset(mov.tp,location=='wet.out')

#Covariate 1: MOV2-Out
mov2.out <- subset(mov.tp, location=='dry.out')

#Covariate 2: Mov1-In
mov1.in1 <- subset(mov.tp, location=='wet.in1')
mov1.in2 <- select(subset(mov.tp,
location=='wet.in2'), date, tp)
mov1.in <- full_join(mov1.in1, mov1.in2, by="date")
mov1.in$tp.in <- 0.3*mov1.in$tp.x +
0.7*mov1.in$tp.y
mov1.in <- select(mov1.in, date, tp.in, period)

#Combine
mov.paired <- select(full_join(mov1.out, mov2.out,
by="date"), date, tp.x, tp.y)
mov.all <- select(full_join(mov.paired, mov1.in,
by="date"), date, tp.in, tp.x, tp.y, period)
mov.all <- na.omit(mov.all)

#Test for relationship
test <- lm(tp.x ~ tp.y*tp.in, data=mov.all)
summary(test)
plot(test) # normality not good. transform

#Log transform
logtest <- lm(log(tp.x)~log(tp.y)*log(tp.in), data
= mov.all)
plot(logtest)
summary(logtest)# not sig., remove interactions
acf(logtest$residuals)

#Main effects only
logtest2 <- lm(log(tp.x)~log(tp.y)+log(tp.in), data
= mov.all)
acf(logtest2$residuals)
summary(logtest2)#In not significant. use only MOV2

#Simplify to variables necessary: MOV2
test3 <- lm(log(tp.x) ~ log(tp.y), data=mov.all)
summary(test3) # significant

#Full ANCOVA Model
ANCOVA_Full <- lm(formula = log(tp.x) ~
log(tp.y)*period, data = mov.all)
summary(ANCOVA_Full) #No significant interactions,
reduced model is valid

#Reduced Model
ANCOVA_Reduced <- lm(formula = log(tp.x) ~
log(tp.y)+period, data = mov.all)
summary(ANCOVA_Reduced) #significant

#LS Means
TP.means <- lsmeans(ANCOVA_Reduced,~period)
pairs(TP.means)
TP.means
#Reduction:1-10^ls-treatment/10^ls-calibration
reduction <- (1-(10^-1.2516339/10^-0.7974062))*100
reduction
```

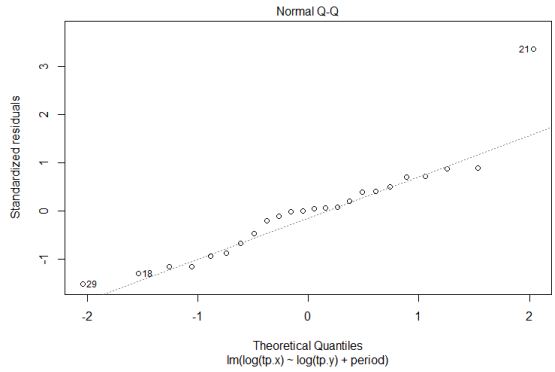
```
#Assumption Tests for ANCOVA
acf(ANCOVA_Reduced$residuals)#not autocorrelated

#TP Autocorrelation Plot
```




```
plot(ANCOVA_Reduced) #variance and normality
assumptions met

#TP Variance Plot
```




```
#TP Normality Plot
```



Multivariate ANCOVA Plots MOV1

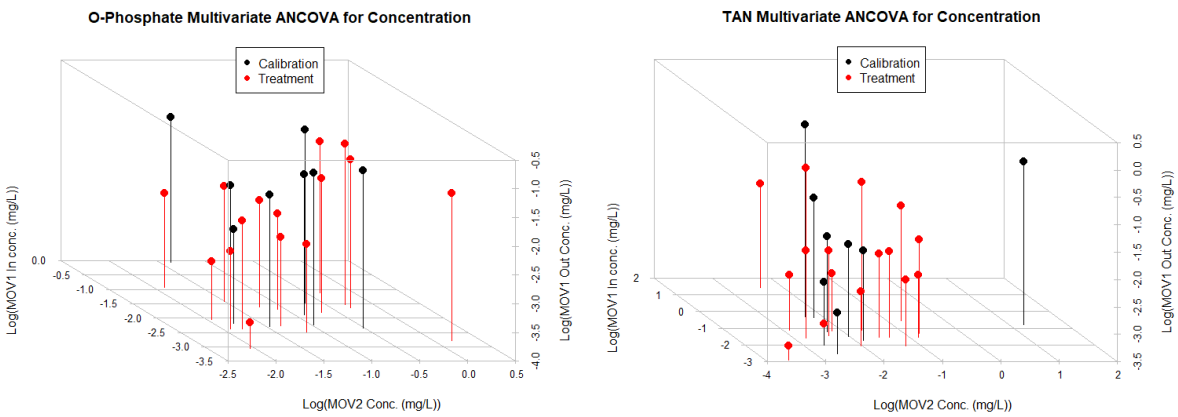


Figure A-11: 3D ANCOVA of O-PO_4^{3-} and TAN EMCs with multiple continuous covariates.

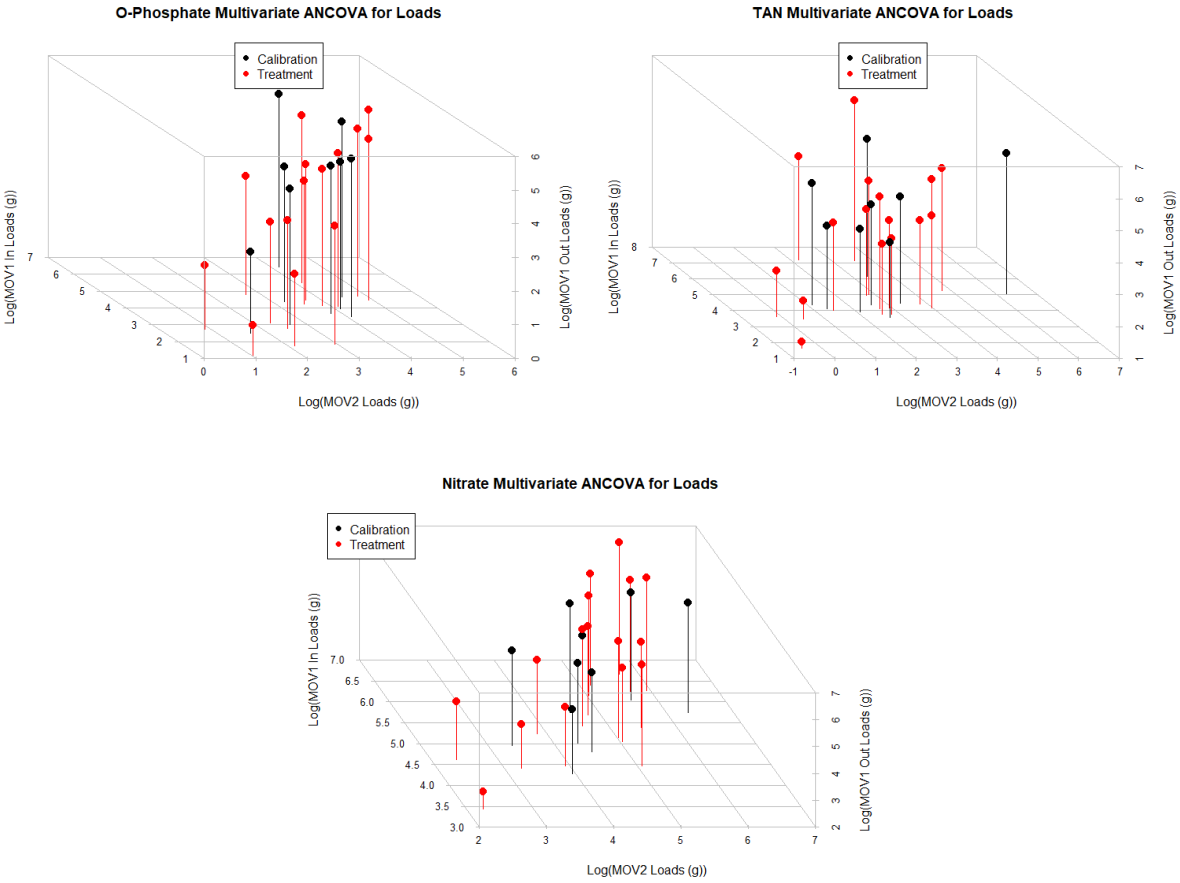


Figure A-12: *3D ANCOVA of O-PO_4^{3-}, TAN, and $NO_{2,3}$-N loads with both MOV1_IN and MOV2 as covariates.*

MOV Concentration Box Plots

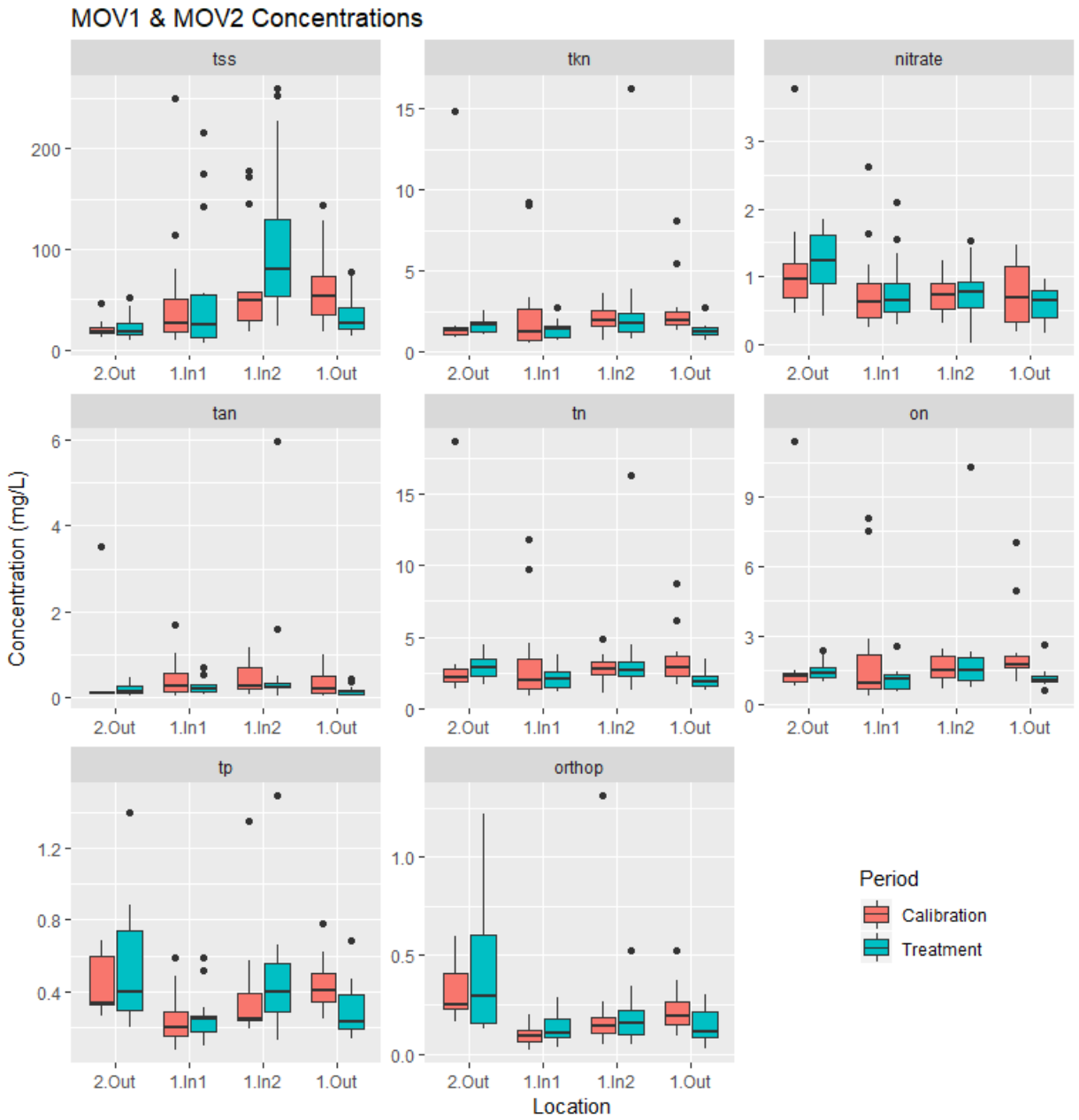


Figure A-13: Concentration boxplots of MOV1 and MOV2 during both phases of monitoring.

WS Concentration Box Plots

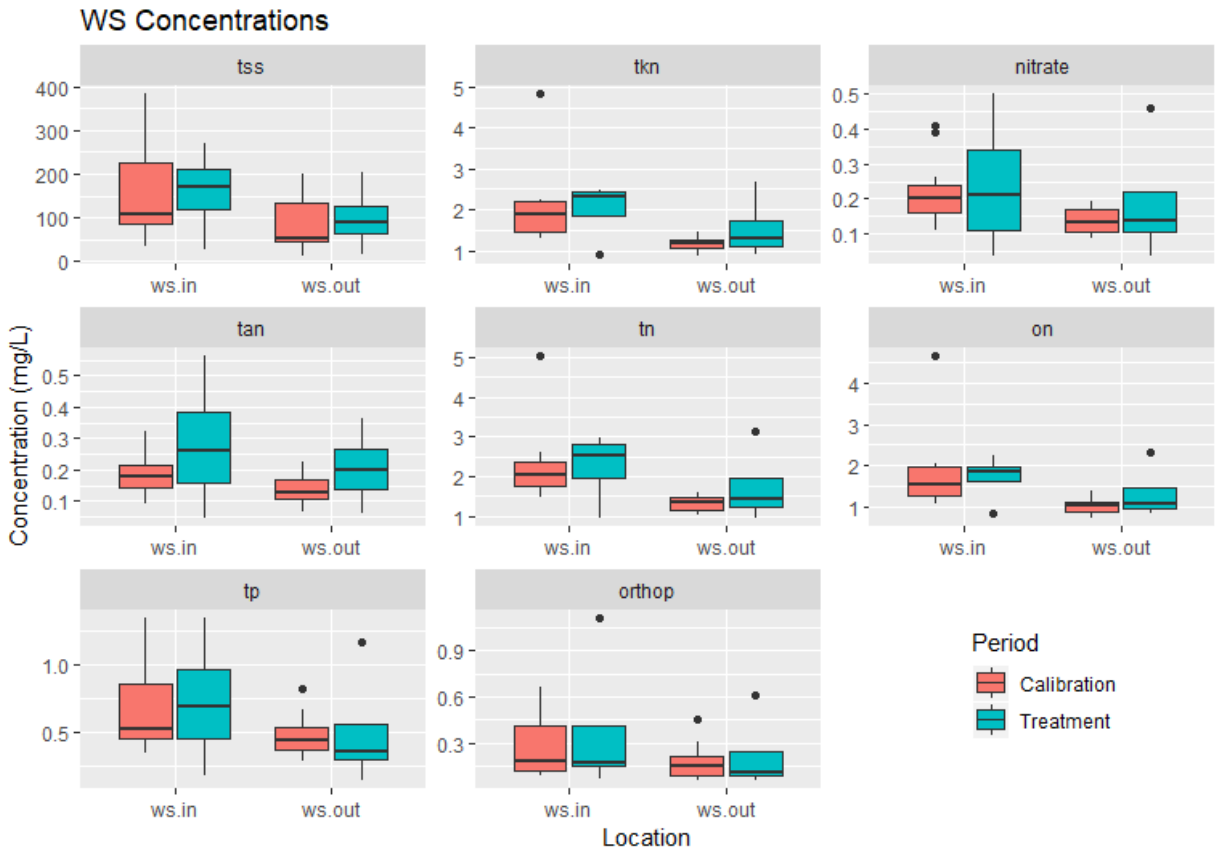


*Figure A-14:*Concentration boxplots of WS_IN and WS_OUT during both phases of monitoring.

Appendix I: WS Retrofit Analysis

The modification of the outlet structure did not increase detention times in the pond, which limited pollutant removal potential. It is possible that the pond design was not sufficiently altered to impact water quality or that insufficient data were collected to detect differences. Alternatively, enhanced treatment could have been limited because WS pre-retrofit treated water quality better than other dry ponds (Chapter 2).

Preliminary analysis revealed no detectable differences between periods in ANCOVA plots (*Figure A-15*), as many best-fit lines were nearly overlapping when graphed. This may be due to a small treatment period (n = 4).

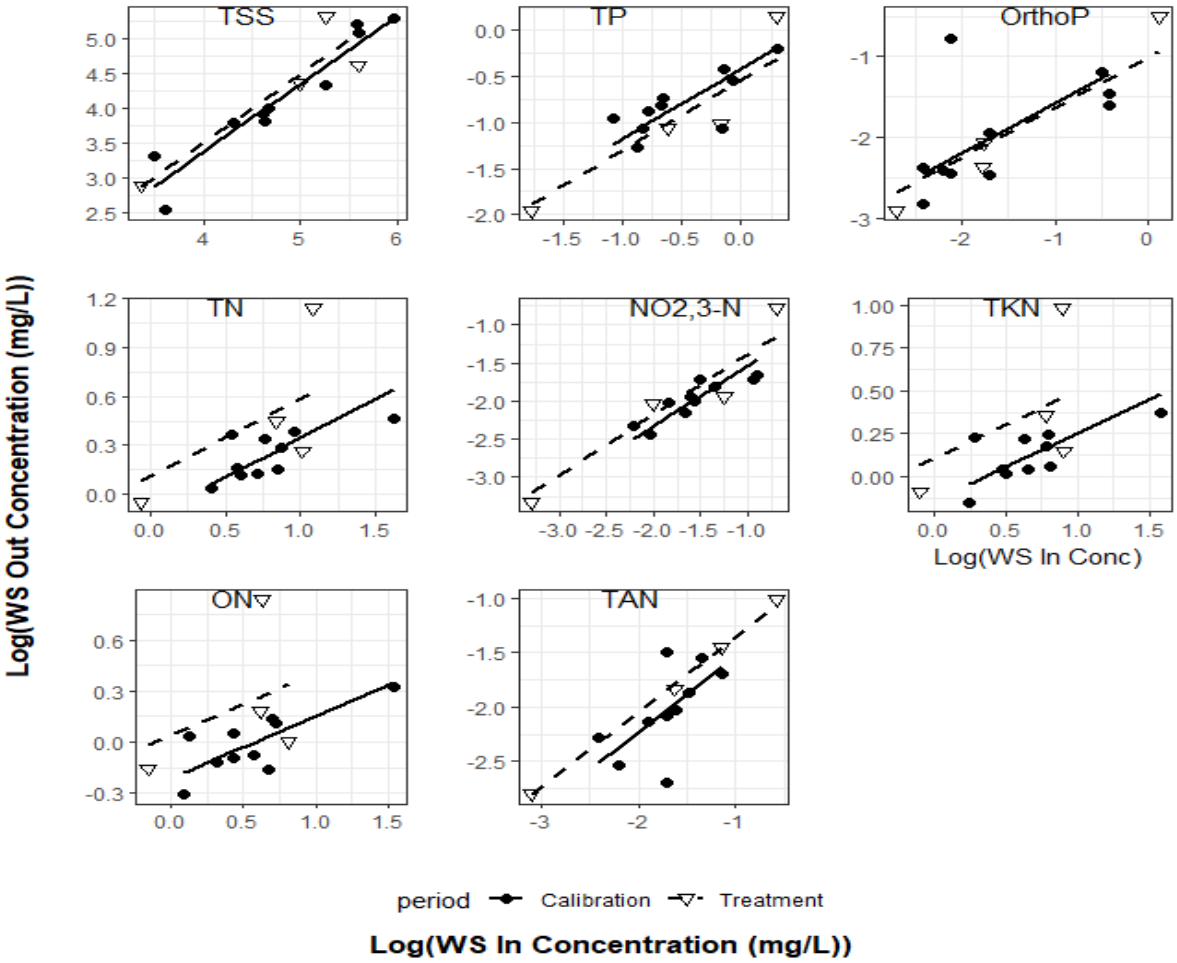


Figure A-15: ANCOVA plots of differences in WS effluent concentrations between monitoring periods.

Both influent and effluent WS loads appeared to have greatly decreased in the treatment phase of monitoring (Table A-19:). This large reduction is partially due to the lack of large storm events, where a larger proportion of rainfall is converted to runoff. Additionally, while the WS outlet retrofit did not include an orifice to detain flows and enhance treatment, the 4.7 m^3 of storage created within the basin likely reduced total effluent loads. Due to in-situ soil conditions, water ponded during storm events typically infiltrated within 48 hours, reducing total effluent volumes, and, therefore, pollutant loads. Preliminary results do not suggest that effluent loads significantly changed.

Regression equations were created for effluent concentrations of TSS, TP, O-PO_4^{3-}, TN, TAN, and $NO_{2,3}$-N and for effluent loads of all analytes (*Table A-20*). No significant differences were found between treatment periods. For those analytes where influent and effluent concentrations were not significantly correlated (TKN and ON), a Student's t-test was performed to assess differences in outlet concentrations between monitoring periods. The difference between calibration and treatment period LSMs are also provided in *Table A-20*. LSMs are reported as a percent reduction from pre-retrofit conditions. Negative values indicate that LSMs increased. There were no significant changes detected between pre- and post-retrofit effluent concentrations.

Table A-19: WS calibration and treatment period loads.

Pollutant	Period	Location	Cumulative Loads *g*	Event Loading *kg/ha/yr*	Event Load Reduction	Pre-Post LSMs*
TSS	Pre	Inflow	55500	206	43%	9.8%
		Outflow	31500	118		
	Post	Inflow	16200	58	52%	
		Outflow	7920	28		
TP	Pre	Inflow	216	0.80	61%	50.0%
		Outflow	153	0.58		
	Post	Inflow	75.9	0.27	45%	
		Outflow	42.9	0.15		
$O\text{-}PO_4^{3-}$	Pre	Inflow	89.8	0.33	22%	42.1%
		Outflow	69.0	0.26		
	Post	Inflow	37.9	0.14	52%	
		Outflow	18.5	0.06		
TN	Pre	Inflow	776	2.88	49%	-36.1%
		Outflow	394	1.48		
	Post	Inflow	268	0.96	36%	
		Outflow	176	0.62		
TAN	Pre	Inflow	70.3	0.26	39%	-15.6%
		Outflow	42.9	0.16		
	Post	Inflow	29.9	0.11	41%	
		Outflow	18.0	0.06		
TKN	Pre	Inflow	670	2.67	52%	-39.8%
		Outflow	328	1.29		
	Post	Inflow	244	0.88	36%	
		Outflow	160	0.56		
ON	Pre	Inflow	640	2.38	51%	-48.6%
		Outflow	308	1.16		
	Post	Inflow	215	0.77	35%	
		Outflow	142	0.50		
$NO_{2,3}\text{-}N$	Pre	Inflow	65.7	0.24	35%	-7.5%
		Outflow	42.4	0.16		
	Post	Inflow	23.3	0.08	36%	
		Outflow	15.3	0.05		

**Negative sign indicates treatment period loads were greater than calibration period loads*

Table A-20: WS Regression equations for pollutant concentrations and loads.

	Pollutant	Regression Equations		ANCOVA		
				Intercept		LSM
		Calibration Period (n = 11)	Treatment Period (n = 4*)	t	p-value	Difference
Concentration mg/L	TSS	LogR = 0.9914LogUS - 0.6049	LogR = 0.9914LogUS - 0.4655	-0.718	0.448	-37.9%
	TP	LogR = 0.7645LogUS - 0.4144	LogR = 0.7645LogUS - 0.539	0.768	0.459	24.9%
	O-PO_4^{3-}	LogR = 0.6223LogUS - 0.9497	LogR = 0.6223LogUS - 1.0023	0.167	0.870	11.4%
	TN	LogR = 0.4758LogUS - 0.1317	LogR = 0.4758LogUS + 0.1099	-1.87	0.088	-74.4%
	$NO_{2,3}$-N	LogR = 0.7886LogUS - 0.7470	LogR = 0.7886LogUS - 0.5933	-1.169	0.267	-42.4%
	TKN	LogR = 0.3966LogUS - 0.1414	LogR = 0.3966LogUS + 0.1057	-1.902	0.084	-76.6%
	TAN	LogR = 0.6888LogUS - 0.8500	LogR = 0.6888LogUS - 0.6661	-1.095	0.297	-52.7%
	ON*	LogR = 0.3313LogUS - 0.1961	LogR = 0.4550LogUS - 0.0025	1.0121	0.386	17%
	TSS	LogR = 1.0514LogUS - 1.1469	LogR = 1.0514LogUS -1.19185	0.201	0.8442	9.8%
	TP	LogR = 1.0997LogUS - 0.6551	LogR = 1.0997LogUS -0.9564	1.547	0.1501	50.0%
	O-PO_4^{3-}	LogR = 0.9816LogUS - 0.4113	LogR = 0.9816LogUS -0.6485	0.601	0.5603	42.1%
	TN	LogR = 0.9300LogUS - 0.3672	LogR = 0.9300LogUS -0.2335	-0.686	0.5067	-36.1%
	$NO_{2,3}$-N	LogR = 1.0990LogUS - 0.6673	LogR = 1.0990LogUS -0.63588	-0.194	0.85	-7.5%
	TKN	LogR = 0.8962LogUS - 0.2489	LogR = 0.8962LogUS -0.1033	-0.704	0.496	-39.8%
	TAN	LogR = 0.8267LogUS - 0.1662	LogR = 0.8267LogUS -+ 0.1032	-0.372	0.717	-15.6%
	ON	LogR = 0.8959LogUS - 0.2847	LogR = 0.8959LogUS -0.1127	-0.772	0.456	-48.6%

US = WS inlet, R = WS Outlet
**No significant relationship was found between US and R. A Student's t-test was used to compare treatment period influent and effluent water quality. Median RE_c is reported.*

Cumulative loads in WS were plotted continuously, with a vertical line indicating the date of the retrofit (March 2018). With little data, no obvious shifts in trends were detectable in the treatment period of monitoring (*Figure A-16*:).

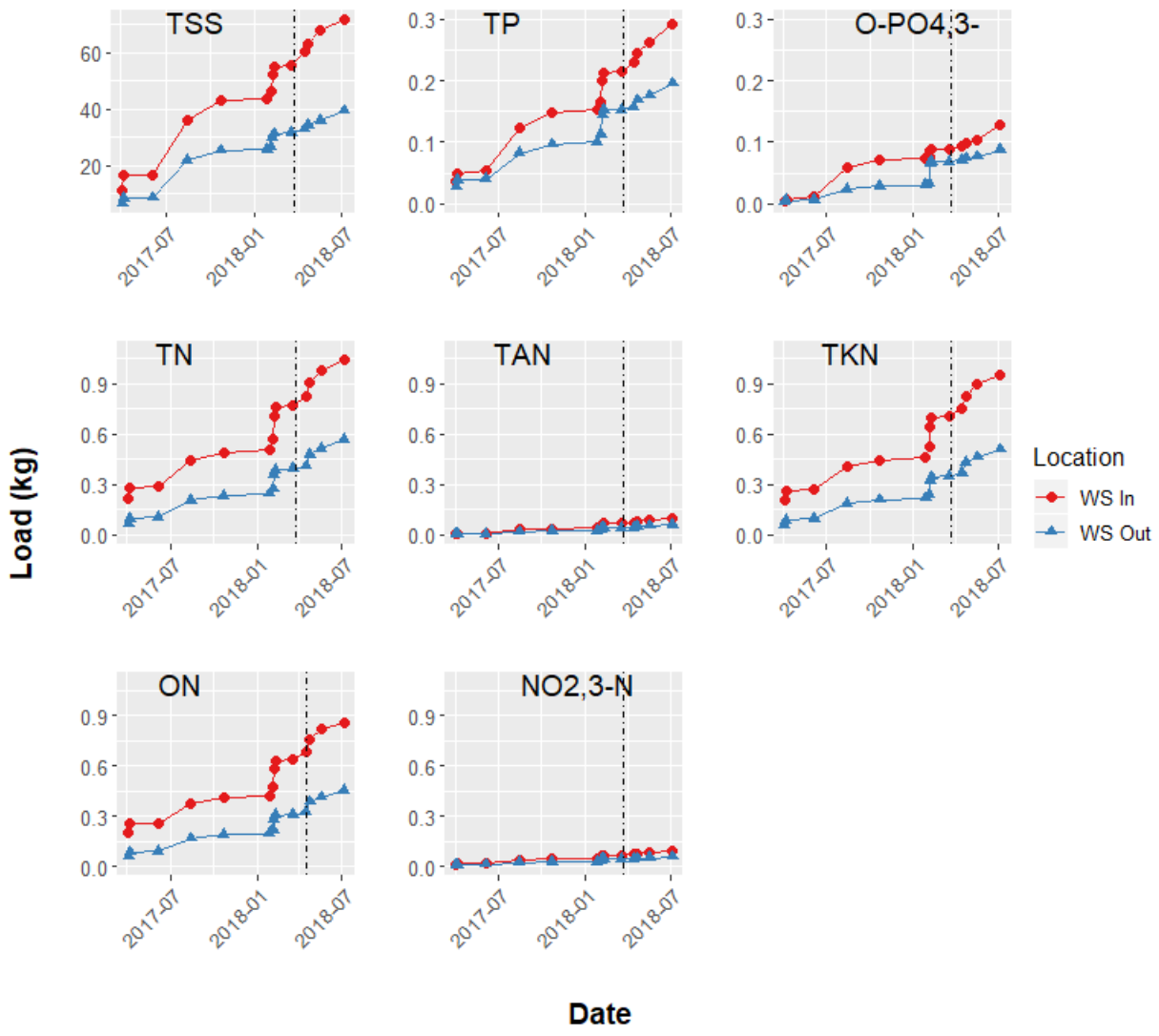


Figure A-16: Influent and effluent cumulative loads of WS throughout monitoring. The vertical line indicates the retrofit on 3/23/2017.

WS effluent water quality was compared to ambient water quality standards and typical dry pond effluent EMC. Probability plots suggest that treatment period effluent water quality of WS did not meet water quality standards for TSS and was similar to the calibration phase (*Figure A-17*:).

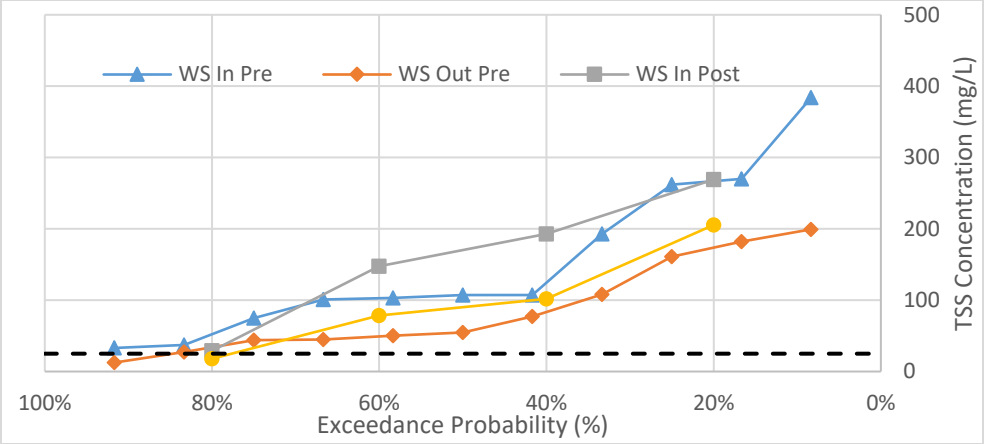


Figure A-17: Exceedance probabilities of WS_IN and _OUT during both monitoring periods with respect to ambient water quality target for TSS (Barrett et al., 2004).

In general, effluent TP concentrations in WS were much lower than expected dry pond conditions in both phases of monitoring (*Figure A-18*:). There appeared to be some reduction in concentration in both phases, which may indicate that this dry pond in its original state reduced TP concentrations more effectively than other dry ponds. However, all concentrations exceeded water quality thresholds, and effluent concentrations were much higher than those assigned by NCDEQ (2017) to wetlands (0.18 mg/L).

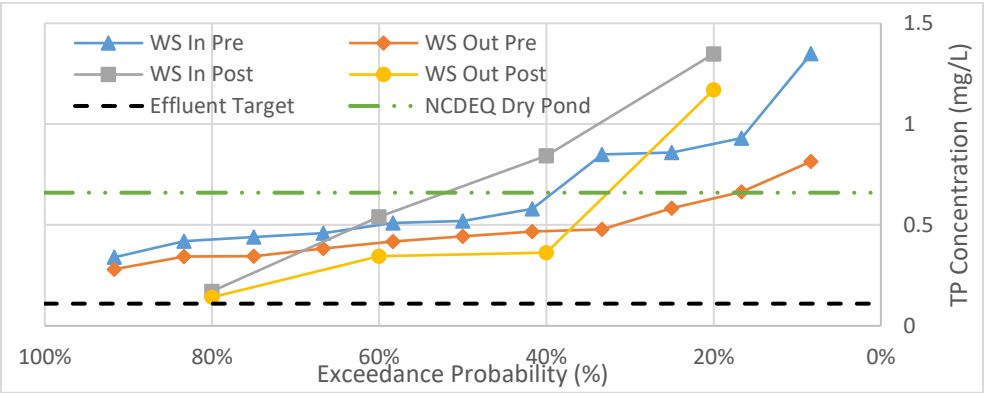


Figure A-18: Exceedance probabilities of TP concentrations at WS_IN and _OUT during both monitoring periods with respect to ambient water quality thresholds (McNett et al., 2010) and NCDEQ (2017) assigned values for dry ponds.

In both monitoring periods, effluent TN concentrations were lower than average dry pond discharge concentrations (NCDEQ, 2017; *Figure A-19*:). However, between the two periods, concentrations did not appear to change. TN concentrations did not meet water quality thresholds

from McNett et al. (2010) and were lower than expected CSW EMCs (NCDEQ, 2017) in 1 of 4 storms.

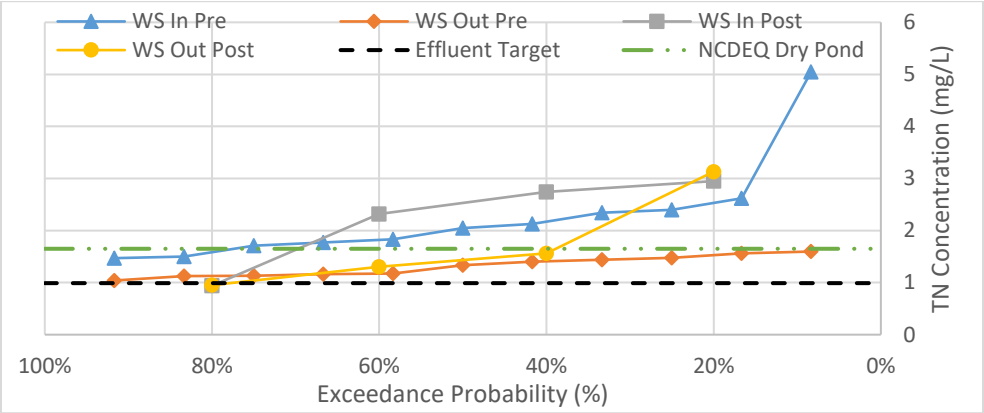


Figure A-19: Exceedance probabilities of TN concentrations at WS_IN and WS_OUT with respect to ambient water quality thresholds (McNett et al., 2010) and NCDEQ (2017) assigned values for dry ponds.

Appendix J: Nitrate Correlation Diagnostic Tests

Several were factors were tested for correlation to predict $NO_{2,3}$-N outlet concentrations and $NO_{2,3}$-N reductions in MOV1. No significant correlations were found.

```
#Outlet Level (Peak Flow) vs Outlet Conc.
model1 <- lm(formula = Nitrate.Out~Max.level, data = nitrate)
summary(model1)#not significant

#Full Model Outlet Level*Inlet Conc. vs Outlet Conc.
model2 <- lm(formula = Nitrate.Out~Max.level*nitrate.in, data = nitrate)
summary(model2)#no significant interactions, reduce

#Reduced Model Outlet Level + Inlet Conc. vs Outlet Conc.
model3 <- lm(formula = Nitrate.Out~Max.level + nitrate.in, data = nitrate)
summary(model3)#not significant

#Full Model Outlet Level*Storm Duration vs Outlet Conc.
model4 <- lm(formula = Nitrate.Out~duration*Max.level, data = nitrate)
summary(model4)#not significant, reduce

#Reduced Model Outlet Level + Storm Duration vs Outlet Conc.
model5 <- lm(formula = Nitrate.Out~ duration + Max.level, data = nitrate)
summary(model5)#not significant

#Subset to exclude storms that exceeded water quality volume
wqv <- subset(nitrate, Max.level < 1.25, -X.1)

#Storm Duration vs Outlet Conc.
model6 <- lm(formula = Nitrate.Out ~ duration, data = wqv)
summary(model6)#not significant

#Nitrate In - Out Reductions
wqv$nitrate.red <- wqv$nitrate.in - wqv$Nitrate.Out
#Storm Duration vs Nitrate Reductions
model7 <- lm(formula = nitrate.red ~ duration, data = wqv)
summary(model7)#not significant

#Max Level (WQV) vs Nitrate Reduction
model8 <- lm(formula = nitrate.red ~ Max.level, data = wqv)
summary(model8)#not significant
```

When storm duration was plotted versus total concentration removal in MOV1, there was a slight negative trend (*Figure A. 11)*. However, this trend was not significantly different.

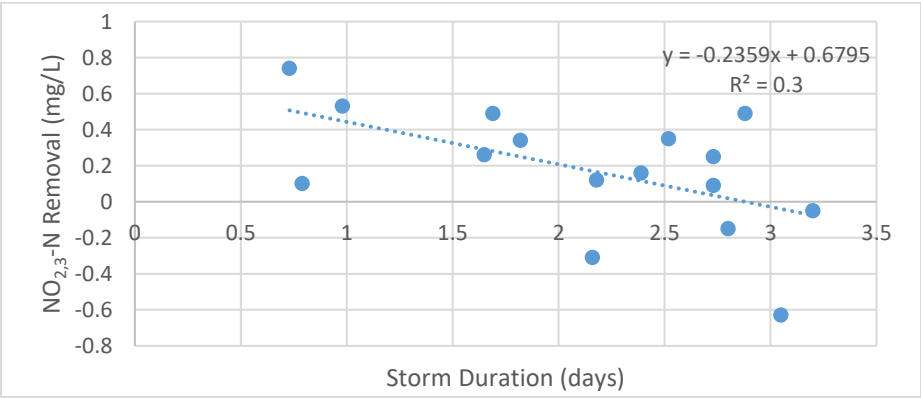


Figure A-20: Total $NO_{2,3}$-N removal over time, which appeared to be the highest for shorter storm events.

Appendix K: Post-retrofit Hydrology

Table A-21: MOV1 treatment period hydrology.

Start Date	Duration *(hr:min)*	Precipitation *(mm)*	5-min Peak Intensity *(mm/hr)*	Runoff Depth *(mm)*	Runoff Fraction	Peak Runoff *(L/s)*	Inflow Volume *(m^3)*	Outflow Volume *(m^3)*
10/16/2017	24:44	7.37	8.84	2.00	0.27	2.02	71.0	83.1
10/23/2017	52:20	30.23	113.14	11.89	0.39	50.40	443.6	493.3
10/28/2017	57:24	21.73	40.24	8.50	0.39	6.01	316.9	352.6
11/8/2017	37:16	11.54	9.11	1.16	0.10	1.01	29.0	47.9
11/12/2017	60:30	18.03	39.67	6.34	0.35	2.88	233.5	263.2
12/6/2017	16:34	6.10	13.72	0.71	0.12	1.17	19.4	29.5
12/8/2017	85:50	43.12	8.51	19.92	0.46	27.01	755.4	826.3
12/20/2017	33:24	11.62	4.86	3.03	0.26	2.01	106.6	125.7
1/12/2018	51:54	13.92	53.01	7.36	0.53	3.00	282.6	305.5
1/23/2018	65:26	25.91	37.17	16.92	0.65	68.62	659.1	701.7
1/28/2018	107:22	48.96	20.50	31.91	0.65	51.01	1243.0	1323.5
2/2/2018	27:48	4.18	22.98	1.69	0.41	1.27	63.4	70.2
2/4/2018	76:50	17.41	14.63	11.41	0.66	3.16	444.6	473.2
2/7/2018	64:52	6.10	8.13	3.62	0.59	1.53	140.1	150.1
2/10/2018	30:16	2.45	3.67	0.88	0.36	0.57	32.5	36.5
2/12/2018	38:58	3.36	3.67	1.49	0.44	0.97	56.3	61.9
2/19/2018	43:46	7.34	7.34	3.18	0.43	1.68	119.7	131.8
2/28/2018	69:06	13.99	12.59	9.61	0.69	2.67	375.8	398.8
3/6/2018	55:58	12.19	6.97	5.38	0.44	2.10	203.2	223.2
3/11/2018	18:58	5.67	5.10	1.06	0.19	1.24	34.8	44.1
3/12/2018	31:38	17.87	8.51	8.35	0.47	11.24	317.2	346.5
3/13/2018	58:42	5.39	6.81	8.35	1.55	2.86	337.6	346.4
3/17/2018	15:26	2.51	15.04	0.70	0.28	1.04	24.9	29.0
3/19/2018	24:44	23.40	16.71	9.07	0.39	35.52	337.8	376.2
3/20/2018	88:42	5.09	15.94	11.84	2.32	3.20	482.8	491.1
3/24/2018	95:48	20.59	9.30	19.03	0.92	23.55	755.4	789.2
4/7/2018	83:08	25.62	11.45	13.19	0.51	32.01	504.8	546.9
4/15/2018	82:50	93.36	83.47	81.66	0.87	295.49	3233.9	3387.4
4/23/2018	73:06	20.57	11.08	8.34	0.41	2.89	312.2	346.0
4/26/2018	33:02	8.38	22.86	3.00	0.36	2.01	110.7	124.5
5/10/2018	14:08	3.56	32.00	0.13	0.04	0.93	-0.6	5.2
5/16/2018	23:28	17.78	54.16	2.40	0.14	2.39	70.4	99.7
5/17/2018	17:30	19.56	42.67	4.94	0.25	2.81	172.7	204.9
5/18/2018	9:28	4.30	29.26	1.82	0.42	2.40	68.2	75.3
5/18/2018	67:14	37.57	46.47	25.46	0.68	62.84	994.4	1056.1
5/21/2018	65:28	27.43	93.59	16.00	0.58	43.30	618.4	663.5
5/28/2018	60:22	25.15	16.23	6.81	0.27	2.90	241.2	282.5

5/30/2018	39:38	7.62	22.86	2.03	0.27	1.73	71.7	84.2
6/11/2018	5:50	7.11	30.48	0.68	0.10	1.25	16.5	28.2
6/26/2018	40:30	20.70	49.37	3.35	0.16	2.29	104.8	138.8
7/5/2018	21:32	52.99	41.03	18.88	0.36	207.75	696.2	783.3
7/6/2018	14:04	47.72	28.80	28.14	0.59	209.07	1089.0	1167.4
7/7/2018	57:46	12.99	289.69	10.33	0.80	12.27	407.2	428.5
7/22/2018	18:58	12.24	22.03	0.82	0.07	1.17	13.9	34.1
7/22/2018	44:04	13.36	7.34	3.38	0.25	2.09	118.1	140.1

Table A-22: MOV2 treatment period hydrology.

Start Date	Duration (hr:min)	Precipitation (mm)	5-min Peak Intensity (mm/hr)	Runoff Depth (mm)	Runoff Fraction	Peak Outflow (L/s)	Runoff Volume (m^3)
10/7/2017	2:46	3.44	10.32	0.00	0.00	0.00	0.0
10/9/2017	3:42	3.44	12.04	0.00	0.00	0.00	0.0
10/11/2017	13:58	47.02	0.00	12.20	0.26	12.43	306.1
10/16/2017	3:48	7.37	8.84	0.10	0.01	0.78	2.5
10/23/2017	6:10	30.23	113.14	7.22	0.24	12.49	181.2
10/28/2017	5:20	21.73	40.24	4.97	0.23	10.95	124.7
11/8/2017	24:16	11.54	9.11	N/A	N/A	N/A	N/A
11/12/2017	7:06	18.03	39.67	N/A	N/A	N/A	N/A
12/6/2017	5:26	6.10	13.72	0.09	0.02	1.57	2.4
12/8/2017	38:10	43.12	8.51	12.93	0.30	9.65	324.4
12/20/2017	11:06	11.62	4.86	1.14	0.10	4.52	28.7
1/12/2018	4:08	13.92	53.01	3.36	0.24	10.82	84.4
1/23/2018	11:32	25.91	37.17	12.06	0.47	13.63	302.6
1/28/2018	41:44	48.96	20.50	21.27	0.43	12.30	533.8
2/2/2018	4:52	4.18	22.98	0.34	0.08	2.21	8.6
2/4/2018	14:08	17.41	14.63	5.54	0.32	9.62	139.1
2/7/2018	9:46	6.10	8.13	0.65	0.11	4.63	16.4
2/10/2018	14:12	2.45	3.67	0.00	0.00	0.00	0.0
2/12/2018	13:12	3.36	5.50	0.02	0.01	0.27	0.4
2/19/2018	5:38	7.34	7.34	0.68	0.09	3.14	17.0
2/28/2018	27:08	13.99	12.59	4.15	0.30	5.48	104.1
3/6/2018	14:00	12.19	6.97	1.81	0.15	4.85	45.5
3/11/2018	18:58	5.67	5.10	0.32	0.06	1.10	8.0
3/12/2018	27:44	17.87	8.51	9.50	0.53	10.31	238.3
3/13/2018	4:52	5.39	6.81	0.08	0.02	0.39	2.1
3/17/2018	1:20	2.51	15.04	0.10	0.04	2.21	2.5
3/19/2018	17:14	23.40	16.71	9.00	0.38	11.77	225.9
3/20/2018	13:16	5.09	15.94	1.82	0.36	7.44	45.8
3/24/2018	23:08	20.59	9.30	11.51	0.56	10.51	288.8
4/7/2018	23:06	25.62	11.45	4.92	0.19	9.00	123.3
4/15/2018	18:00	93.36	83.47	32.22	0.35	18.31	808.4
4/23/2018	18:58	20.57	11.08	3.95	0.19	7.36	99.2
4/26/2018	5:46	8.38	22.86	1.17	0.14	8.07	29.4
5/10/2018	0:06	3.56	32.00	0.00	0.00	0.00	0.0
5/16/2018	14:44	17.78	54.16	1.09	0.06	9.17	27.4
5/17/2018	13:08	19.56	42.67	2.93	0.15	9.62	73.4
5/18/2018	2:10	4.30	29.26	0.44	0.10	4.91	11.0
5/18/2018	23:00	37.57	46.47	11.16	0.30	10.81	280.1
5/21/2018	14:00	27.43	93.59	7.20	0.26	10.27	180.5
5/28/2018	34:28	25.15	16.23	2.80	0.11	8.90	70.3
5/30/2018	9:12	7.62	22.86	0.89	0.12	6.03	22.4

6/11/2018	3:30	7.11	30.48	0.10	0.01	1.57	2.5
6/26/2018	13:44	20.70	49.37	0.63	0.03	2.72	15.8
7/5/2018	8:24	52.99	41.03	13.95	0.26	15.49	350.1
7/6/2018	9:56	47.72	20430.07	16.92	0.35	16.21	424.6
7/7/2018	8:18	12.99	6.78	3.08	0.24	9.03	77.2
7/22/2018	6:22	12.24	22.03	0.03	0.00	0.33	0.7
7/22/2018	25:10	13.36	7.34	0.80	0.06	5.53	20.1

Table A-23: WS treatment period hydrology.

Start Date	**Duration** *(hr:min)*	**Precipitation** *(mm)*	**5-min Peak Intensity** *(mm/hr)*	**Runoff Depth** *(mm)*	**Peak Outflow** *(L/s)*	**Outflow Volume** *(m^3)*	**Inflow Volume** *(m^3)*
3/24/2018	62:00	19.60	7.54	0.61	1.11	5.7	10.6
4/4/2018	1:50	4.02	13.57	0.00	0.00	0.0	0.0
4/6/2018	25:46	22.36	10.55	0.00	0.00	0.0	0.0
4/15/2018	42:02	29.97	48.53	1.46	0.23	13.6	16.0
4/23/2018	70:52	64.01	21.77	7.93	0.24	73.8	84.4
4/26/2018	83:40	12.19	9.14	0.97	0.07	9.0	9.0
5/7/2018	1:08	3.05	6.10	0.00	0.00	0.0	0.0
5/10/2018	0:38	4.32	10.67	0.00	0.00	0.0	0.0
5/11/2018	1:34	8.38	9.14	0.00	0.00	0.0	0.0
5/16/2018	15:04	23.16	3.20	0.00	0.00	0.0	0.0
5/17/2018	6:14	8.79	22.37	0.00	0.00	0.0	0.0
5/18/2018	42:06	35.68	55.92	2.35	0.31	21.8	33.4
5/19/2018	22:44	6.66	217.27	0.01	0.00	0.1	0.1
5/20/2018	20:24	2.93	43.13	0.00	0.00	0.0	0.0
5/21/2018	7:56	2.53	20.61	0.00	0.00	0.0	0.0
5/26/2018	19:12	10.11	18.20	0.00	0.00	0.0	0.0
5/29/2018	10:34	6.07	27.30	0.00	0.00	0.0	0.0
5/29/2018	5:44	2.53	27.30	0.00	0.00	0.0	0.0
6/1/2018	2:44	10.36	24.26	0.00	0.00	0.0	0.0
6/9/2018	5:30	13.90	9.10	0.00	0.00	0.0	3.7
6/21/2018	13:22	14.22	6.10	0.00	0.00	0.0	0.0
6/25/2018	17:02	5.08	3.05	0.00	0.00	0.0	0.0
7/6/2018	35:42	32.77	99.06	1.83	0.62	17.0	21.0
7/17/2018	0:26	4.57	24.38	0.00	0.00	0.0	0.0
7/22/2018	117:36	15.24	51.82	3.96	0.25	36.9	45.3
7/31/2018	16:14	0.00	0.00	0.98	0.49	9.1	13.8

Appendix L: Water Quality during the Calibration Phase

Table A-24: MOV1_IN1 water quality during the treatment phase.

Start Date	TKN (mg/L)	$NO_{2,3}$-N (mg/L)	TAN (mg/L)	TN (mg/L)	ON (mg/L)	TP (mg/L)	OPO_4^{3-} (mg/L)	TSS (mg/L)
10/23/2017	1.25[a]	0.29	0.11	1.54	1.14	0.26	0.17	34
10/28/2017	0.78	0.35	0.12	1.13	0.66	0.22	-	7
11/8/2017	0.80	0.54	0.18	1.33	0.62	0.15	0.12	3
1/12/2018	1.60	0.47	0.16	2.07	1.45	0.51	0.19	216
1/23/2018	1.52	0.54	0.20	2.06	1.32	0.59	0.29	175
2/4/2018	0.85	0.47	0.14	1.32	0.71	0.25	0.16	25
2/19/2018	0.83	0.64	0.23	1.47	0.60	0.15	0.10	18
2/28/2018	1.48	0.86	0.20	2.34	1.28	0.26	0.16	38
3/6/2018	1.03	0.75	0.25	1.78	0.79	0.18	0.13	13
3/11/2018	2.78[a]	0.70	0.25	3.48	2.53	0.31	0.18	25
3/19/2018	3.25	0.66	1.35	3.90	1.90	0.28	0.14	55
3/24/2018	1.37	0.80	0.55	2.16	0.82	0.19	0.14	14
4/7/2018	5.29	2.71	1.25	8.00	4.03	0.17	0.10	19
4/23/2018	1.44	0.87	0.30	2.31	1.13	0.17	0.09	19
5/16/2018	2.01	1.54	0.70	3.55	1.31	0.24	0.08	56
5/17/2018	1.67	2.10	0.55	3.77	1.11	0.18	0.09	12
5/18/2018	1.70	1.33	0.55	3.03	1.15	0.26	0.21[b]	13
5/21/2018	1.49	0.96	0.25	2.45	1.24	0.25	0.10	55[b]
5/28/2018	0.80	0.67	0.17	1.47	0.64	0.12	0.05	12
5/30/2018	1.24	0.33	0.08	1.56	1.16	0.25	0.04	142
6/11/2018	2.10	0.60	0.83	2.70	1.26	0.17	0.05	43
6/26/2018	0.90	0.60	0.30	1.50	0.60	0.10	0.04	9
7/5/2018[c]	1.18	0.44	0.29	1.62	0.90	0.23	0.10	83
7/6/2018	0.95	0.81	0.17	1.76	0.78	0.26	0.18	20
7/22/2018	1.41	1.07	0.26	2.48	1.15		0.13	25

[a]*Accuracy standard not met*
[b]*Holding time exceeded*
[c]*Did not capture entire storm. Storm not included in analysis*

Table A-25: MOV1_IN2 water quality during the treatment phase.

Start Date	TKN	$NO_{2,3}$-N	TAN	TN	ON	TP	O-PO_4^{3-}	TSS

	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)
10/23/2017	0.95[a]	0.56	0.14	1.52	0.81	0.39	0.20	97
10/28/2017	1.08	0.54	0.13	1.62	0.95	0.40	-	33
11/8/2017	0.78	0.67	0.07	1.45	0.71	0.20	0.11	51
1/12/2018	1.89	0.65	0.24	2.54	1.65	0.55	0.23	228
1/23/2018	1.75	0.90	0.22	2.65	1.53	0.66	0.35	140
2/4/2018	2.39	0.52	0.38	2.91	2.01	0.58	0.22	100
2/19/2018	1.76	0.91	0.28	2.67	1.48	0.29	0.14	51
2/28/2018	1.75	1.42	0.23	3.17	1.53	0.32	0.20	60
3/6/2018	1.22	1.12	0.21	2.35	1.02	0.25	0.17	40
3/11/2018	1.72[a]	0.76	0.28	2.48	1.44	0.35	0.15	84
3/19/2018	2.40	0.63	0.29	3.03	2.12	0.44	0.20	130
3/24/2018	-	-	-	-	-	-	-	-
4/7/2018	3.67	1.15	0.94	4.82	2.73	0.52	0.15	74
4/23/2018	16.27	0.02	5.95	16.28	10.31	1.49	0.52	127
5/16/2018	2.65	1.06	0.51	3.71	2.14	0.63	0.08	253
5/17/2018	1.33	1.53	0.24	2.86	1.09	0.23	0.09	12
5/18/2018	3.89	0.55	1.59	4.43	2.30	0.50	0.26[b]	78
5/21/2018	2.42	0.96	0.24	3.38	2.18	0.48	0.10	260[b]
5/28/2018	1.14	0.80	0.23	1.94	0.91	0.20	0.11	55
5/30/2018	0.83	0.46	0.04[c]	1.30	0.79	0.13	0.05	24
6/11/2018	2.15	0.87	0.54	3.02	1.61	0.28	0.08	98
6/26/2018	1.56	0.83	0.34	2.38	1.22	0.28	0.07	65
7/5/2018[d]	6.53	0.73	2.75	7.26	3.78	0.96	0.40	172
7/6/2018	7.02	0.95	3.53	7.97	3.49	1.42	0.29	334
7/22/2018[e]	0.84	0.57	0.26	1.41	0.58		0.04	7

[a]*Accuracy standard not met*
[b]*Holding time exceeded*
[c]*Used H_2SO_4 instead of sample to pH adjust*
[d]*Did not capture entire storm. Storm not included in analysis*
[e]*Sample contaminated. Excluded from analysis*

Table A-26: MOV1_OUT water quality during treatment monitoring.

Start Date	**TKN**	**$NO_{2,3}$-N**	**TAN**	**TN**	**ON**	**TP**	**O-PO_4^{3-}**	**TSS**
	(mg/L)	*(mg/L)*	*(mg/L)*	*(mg/L)*	*(mg/L)*	*(mg/L)*	*(mg/L)*	*(mg/L)*

10/23/2017	1.16[a]	0.36	0.10	1.52	1.06	0.47	0.24	255
10/28/2017	0.95	0.32	0.08	1.27	0.87	0.28	-	14
11/8/2017	1.07	0.17	0.18	1.23	0.89	0.16	0.09	11
1/12/2018	1.29	0.90	0.14	2.18	1.15	0.43	0.30	27
1/23/2018	1.16	0.70	0.14	1.86	1.02	0.45	0.25	74
2/4/2018	0.75	0.56	0.09	1.32	0.67	0.18	0.12	15
2/19/2018	1.03	0.49	0.04	1.52	1.00	0.19	0.05	77
2/28/2018	1.17	0.76	0.09	1.93	1.08	0.19	0.10	21
3/6/2018	2.03	0.71	0.13	2.74	1.89	0.44	0.11	235
3/11/2018	2.77	0.64	0.17	3.41	2.60	0.69	0.19	582
3/19/2018	2.44	0.72	0.90	3.16	1.55	0.29	0.17	31
3/24/2018	0.99	0.62	0.24	1.61	0.75	0.17	0.10	19
4/7/2018	1.75	0.95	0.14	2.70	1.60	0.32	0.10	121
4/23/2018	1.33	0.90	0.20	2.23	1.13	0.23	0.09	25
5/16/2018	1.65	0.67	0.25	2.32	1.40	0.29	0.08	48
5/17/2018	1.54	0.96	0.45	2.51	1.09	0.22	0.12[b]	20
5/18/2018	1.63	0.93	0.39	2.56	1.24	0.21	0.14[b]	14
5/21/2018	1.56	0.71	0.14	2.27	1.42	0.24	0.09	44
5/28/2018	1.13	0.41	0.15	1.55	0.98	0.20	0.07	37
5/30/2018	1.06	0.16	0.04	1.22	1.02	0.14	0.03	27
6/11/2018	2.65	0.25	0.06	2.91	2.60	0.52	0.08	-
6/26/2018	1.33	0.27	0.08	1.60	1.25	0.36	0.24	21.01
7/5/2018[c]	1.28	0.77	0.35	2.06	0.93	0.38	0.24	38.17
7/6/2018	1.39	1.02	0.32	2.41	1.07	0.34	0.24	31.63
7/22/2018	1.08	0.34	0.11	1.42	0.98		0.19	40.34

[a]*Accuracy standard not met*
[b]*Holding time exceeded*
[c]*Did not capture entire storm. Storm not included in analysis*

Table A-27: MOV2_OUT water quality during the treatment phase.

Start Date	**TKN** *(mg/L)*	**$NO_{2,3}$-N** *(mg/L)*	**TAN** *(mg/L)*	**TN** *(mg/L)*	**ON** *(mg/L)*	**TP** *(mg/L)*	**O-PO_4^{3-}** *(mg/L)*	**TSS** *(mg/L)*

10/23/2017	2.00[a]	1.30	0.29	3.29	1.71	0.86	0.75	32
10/28/2017	1.35	1.24	0.13	2.59	1.22	0.70	-	16
12/20/2017	1.87	1.00	0.12	2.87	1.76	0.39	0.36	13
1/12/2018	1.80	1.79	0.22	3.59	1.58	0.88	0.75	44
1/23/2018	1.57	1.24	0.26	2.81	1.31	0.84	0.70	52
2/4/2018	1.11	0.72	0.11	1.83	1.00	0.40	0.29	13
2/19/2018	1.15	0.63	0.09	1.78	1.06	0.23	0.14	17
2/28/2018	2.30	1.47	0.43	3.77	1.86	0.40	0.34	25
3/11/2018	1.68[a]	1.18	0.48	2.86	1.20	0.57	0.50	22
3/24/2018	0.97	0.83	0.15	1.80	0.81	0.35	0.30	14
4/23/2018	1.70	1.61	0.09	3.30	1.60	0.34	0.15	30.83
5/16/2018	2.62	1.86	0.47	4.47	2.15	0.52	0.31	24.35
5/17/2018	1.70	1.68	0.19	3.38	1.51	0.30	0.17	12.61
5/18/2018	1.25	1.07	0.13	2.32	1.13	0.28	0.22[b]	9.12
5/21/2018	1.65	0.97	0.10	2.61	1.55	0.39	0.26	17.96[b]
5/28/2018	1.31	0.67	0.06	1.98	1.25	0.23	0.15	17.17
5/30/2018	1.20	0.42	0.03	1.63	1.18	0.20	0.13	10.66
6/26/2018	2.42	1.63	0.06	4.04	2.36	1.40	1.22	16.46
7/5/2018	1.45	1.20	0.26	2.64	1.19	0.77	0.63	25.36
7/6/2018[c]	1.28	0.83	0.19	2.12	1.10	0.50	0.40	21.05
7/22/2018	1.85	0.44	0.04	2.29	1.81	0.40	0.26	32.60

[a]*Accuracy standard not met*
[b]*Holding time exceeded*
[c]*Power failed during sampling. Sample not included.*

Table A-28: WS_IN water quality during the treatment phase.

Start Date	**TKN** *(mg/L)*	**$NO_{2,3}$-N** *(mg/L)*	**TAN** *(mg/L)*	**TN** *(mg/L)*	**ON** *(mg/L)*	**TP** *(mg/L)*	**O-PO_4^{3-}** *(mg/L)*	**TSS** *(mg/L)*
4/15/2018	2.46	0.29	0.20	2.74	2.26	0.84	0.17	269
4/23/2018	0.90	0.04[a]	0.05	0.94	0.86	0.17	0.07	29
5/18/2018	2.18	0.14	0.32	2.32	1.86	0.54	0.17[a]	148
7/6/2018	2.45	0.50	0.56	2.95	1.88	1.35	1.12	193

[a]*Holding time exceeded*

Table A-29: WS_OUT water quality during the treatment phase.

Start Date	**TKN** *(mg/L)*	**$NO_{2,3}$-N** *(mg/L)*	**TAN** *(mg/L)*	**TN** *(mg/L)*	**ON** *(mg/L)*	**TP** *(mg/L)*	**O-PO_4^{3-}** *(mg/L)*	**TSS** *(mg/L)*

4/15/2018	1.16	0.14	0.16	1.30	1.00	0.36	0.09	102
4/23/2018	0.92	0.04	0.06	0.95	0.86	0.14	0.06	18
5/18/2018	1.43	0.13	0.23	1.56	1.20	0.35	0.13[a]	78
7/6/2018	2.67	0.46	0.36	3.13	2.31	1.17	0.61	205

[a]*Holding time exceeded*

Table A-30: MOV1_IN2 baseflow samples.

Sample	**TKN** *(mg/L)*	**$NO_{2,3}$-N** *(mg/L)*	**TAN** *(mg/L)*	**TN** *(mg/L)*	**ON** *(mg/L)*	**TP** *(mg/L)*	**O-PO_4^{3-}** *(mg/L)*	**TSS** *(mg/L)*
5/11/2018	0.57	1.36	0.16	1.93	0.40	0.17	0.12	16
6/8/2018	0.29	0.98	0.02	1.27	0.26	0.14	0.12	6
6/20/2018	0.14[a]	0.38	0.01[a]	0.52	0.13	0.13	0.13	5
6/29/2018	0.14[a]	1.18	0.02	1.32	0.12	0.14	0.12	9
7/5/2018	0.32	1.17	0.02	1.48	0.30	0.15	0.12	16
7/12/2018	0.14[a]	1.63	0.03	1.77	0.11	0.17	0.13	32
7/20/2018	0.31	1.28	0.02	1.58	0.29	0.17	0.13	41

[a]*Concentration below PQL. 0.5*PQL used for analysis.*

Table A-31: MOV1_OUT baseflow samples.

Sample	**TKN** *(mg/L)*	**$NO_{2,3}$-N** *(mg/L)*	**TAN** *(mg/L)*	**TN** *(mg/L)*	**ON** *(mg/L)*	**TP** *(mg/L)*	**O-PO_4^{3-}** *(mg/L)*	**TSS** *(mg/L)*
5/11/2018	1.73	0.017	0.15	1.75	1.58	0.35	0.13	60
6/8/2018	0.99	0.006[a]	0.05	1.00	0.94	0.20	0.06	22
6/20/2018	1.19	0.006[a]	0.03	1.20	1.17	0.26	0.10	32
6/29/2018	0.84	0.006[a]	0.03	0.85	0.81	0.22	0.13	4
7/5/2018	0.96	0.006[a]	0.05	0.97	0.91	0.16	0.08	24
7/12/2018	1.17	0.014	0.07	1.18	1.10	0.25	0.11	10
7/20/2018	0.93	0.006[a]	0.03	0.94	0.91	0.20	0.11	23

[a]*Concentration below PQL. 0.5*PQL used for analysis.*

Table A-32: Nitrogen deposition samples.

Start Date (mg/L)	**TKN** (mg/L)	**$NO_{2,3}$-N** (mg/L)	**TAN** (mg/L)	**TN** (mg/L)	**ON** (mg/L)
5/18/2018	0.34	0.10	0.19	0.44	0.15
5/21/2018	0.14[a]	0.06	0.14	0.20	0.001
7/6/2018	0.36	0.21	0.31	0.57	0.05
7/8/2018	0.14[a]	0.16	0.12	0.30	0.02

[a]*Concentration below PQL. 0.5*PQL used for analysis.*