

ABSTRACT

WILSON, CORINNE EILEEN. A Comparison of Runoff Quality and Quantity from an Innovative Underground Low Impact Development and a Conventional Development. (Under the direction of Dr. William F Hunt III.)

Urbanization and its associated increased impervious footprint lead to stream impairment through erosion, flooding, and augmented pollutant loads. Low Impact Development (LID) focuses on disconnecting impervious areas, increasing infiltration and evapotranspiration, and harvesting stormwater on site through the use of stormwater control measures (SCMs). In this study, a conventional development (centralized stormwater management) and an adjoining infiltration-based LID commercial site in Raleigh, North Carolina, were compared with respect to hydrology and water quality. The conventional development (2.76 ha, 61% directly connected impervious area (DCIA)) and the LID (2.53 ha, 84% DCIA) have underlying hydrologic soil group B soils Cecil and Appling. A dry detention basin, designed to mitigate peak flow rate, was the conventional development SCM. The LID site consisted of a 44,300-liter aboveground cistern used for indoor toilet flushing, two underground cisterns (57,900 liters and 60,600 liters), used for landscape irrigation, and an underground detention system, which overflowed into a series of infiltration galleries beneath the parking lot of the shopping center. The LID shopping center was designed to mimic predevelopment hydrology for the 10-year return period, 24-hour duration storm and meet nutrient requirements for total nitrogen (TN).

For the 47 hydrologic storms monitored, runoff coefficients of 0.02 at the LID site and 0.49 at the conventional site were calculated, when normalized by DCIA. The conventional

development had an 11 times higher peak flow value, for the median storm, than the LID site when normalized by DCIA. For three storms more intense than the 10-year, 5-minute intensity storm, the conventional site averaged a 7.7 times higher peak flow than the LID, when normalized by DCIA.

Flow proportional, composite water quality samples were analyzed for TN, total phosphorus (TP), total Kjeldahl nitrogen (TKN), total ammoniacal nitrogen (TAN), nitrite-nitrate (NO_2+NO_3), orthophosphate (Ortho-P) and total suspended solids (TSS). For the 20 water quality storms sampled, the LID site pollutant loadings for all species studied were less than 5% of pollutant loadings at the conventional site. Results from this innovative combined detention, stormwater harvesting, and infiltration commercial LID demonstrate highly effective and space-saving solutions for areas where aboveground SCMs, such as constructed stormwater wetlands, are not feasible due to high land costs. Exceptional results from this LID were a result of an “overdesigned” system capable of capturing the 77-mm storm, rather than a typical 25-mm storm, as well as the infiltration capacity of the type B soils.

A Comparison of Runoff Quality and Quantity from an Innovative Underground Low Impact
Development and a Conventional Development

by
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A thesis submitted to the Graduate Faculty of
North Carolina State University
in partial fulfillment of the
requirements for the Degree of
Masters of Science

Biological and Agricultural Engineering

Raleigh, North Carolina

2013

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Biography

Corinne Wilson was born April 4, 1989 to Mr. & Mrs. Bruce and Eileen Dumonceau. Her family, including her younger brother, Adam Dumonceau, grew up in St. James, NY, a town a short drive from the beach on the north shore of Long Island. Although she did not develop much of a “lon-guyland accent” while living in NY, she gained many experiences volunteering and traveling with her church. After graduating from Smithtown High School East in 2007, she and her family decided to uproot to North Carolina, Corinne attending North Carolina State University in Raleigh, and her parents and brother moving into their new home in Mooresville, NC. Though she had many valuable experiences, such as participating in the Park Scholarships, volunteering with Habitat for Humanity, and getting involved with local food systems through Interfaith Food Shuttle, her most valued experience was meeting her charming, handsome, and sometimes goofy husband, Gregory Wilson. Corinne graduated from NC State in 2011, with a Bachelors degree in Civil Engineering, concentration in Structural Engineering, completion of the prestigious Park Scholarships program, and a love for all things Wolfpack. Because of her developing interest in the environment, Corinne decided to continue her studies at NC State in Biological and Agricultural Engineering, concentrating in Stormwater under Dr. Bill Hunt. Upon graduating, Corinne will be working for Hazen and Sawyer in Raleigh, NC.

Acknowledgements

I feel like I have had a very blessed academic career and life, filled with the support of colleagues, friends, and family, all of who played a part in these past two years. The first ‘thank you’ would be to my husband, Greg, for finding Dr. Bill Hunt’s website and encouraging me to meet with him about graduate school. Without Greg’s encouragement and superman-like ability to network, I would not be where I am today.

A host of gratitude goes to Bill for taking me on as a student, believing in me right from the beginning, and for answering my countless questions. Thank you to my committee members, Dr. Greg Jennings and Andy Fox for supporting me and challenging me to think further and deeper, and for volunteering your time in this endeavor. I have to thank Ryan Winston for putting up with probably hundreds of my questions, as well as reading countless paper drafts, and giving advice and knowledge. This project could not have physically happened without the expertise of Shawn Kennedy; a sincere thank you goes to him and Wes Kimbrell for the time and effort in installing monitoring equipment (and answering all my questions on how it works). To the entire stormwater team (alumni included): I could not have imagined a better work atmosphere, support system, or group of teammates to work with over the past two years. I have to especially thank my officemate and great friend Laura Merriman for so many things! But especially listening to my questions and rants, keeping me company sampling, and making me laugh when I need it the most (and really all the time...she’s pretty funny).

I'd like to thank the Clean Water Management Trust Fund for funding this project. It has been a great experience to work in a city I'm passionate about on such an innovative project. Patrick Smith at Soil & Environmental Consultants, the designer of the project I studied, has been more than willing to help throughout this entire process, and I am so happy and thankful that he is so passionate about this design. I'd like to thank Regency for allowing me to work on such an interesting project.

A sincere thank you goes to my parents and my brother for their support and love. Thank you to the Wilson side of the family, for your care and support. And lastly, I'd like to thank my husband, Greg, for the encouragement to pursue my passions and for his love and inspiration every day.

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Chapter 1: A Review of Literature

1.1 Urban Stormwater Impacts and Regulations

As a result of growing concern for controlling water pollution, the United States Environmental Protection Agency (USEPA) introduced the Clean Water Act in 1972. The goal of this legislation was for waterways to be “fishable and swimmable,” or meet their intended use, by reducing both point source and nonpoint source water pollution. Point sources are any “discernable, confined, and discrete conveyance...from which pollutants are or may be discharged.” Nonpoint sources for pollution include rainfall, snowmelt, and groundwater. For example, nonpoint source pollution may include nutrients in fertilizers applied on agricultural land and residential lawns or oils and hydrocarbons from vehicles on roadways that are mobilized by surface water runoff and conveyed toward receiving waters. The USEPA’s *National Water Quality Inventory: 2004 Report to Congress* identified nonpoint source pollution as one of the top sources of impairment in assessed waterways (USEPA, 2009).

Of the pollutant sources described in the *National Water Quality Inventory: 2004 Report to Congress*, urban stormwater runoff is one of the leading sources of impairment in waterways (USEPA, 2009). Dietz and Clausen (2008) monitored runoff volume in CT during development to determine the effect of increasing the percentage of impervious area. As the impervious area increased from 1% to 32% during construction, an increase in runoff of 4900% was reported at a conventional development (Dietz and Clausen, 2008). The

increased impervious area due to urbanization, even if limited to 10% of the total watershed, increases flood frequency, increases discharge volume, and disturbs channel geomorphology (Moscrip and Montgomery, 1997). Streambed alterations such as incision and stream bank widening occur to accommodate the increased runoff volume, causing stream erosion and impairment to aquatic ecosystems (Hammer, 1973; Wang and Lyons, 2003).

Another important metric for evaluating impairment to receiving waterways is pollutant loads, a product of runoff volumes and pollutant concentrations. Line and White (2007) monitored a residential development in NC for runoff quality and quantity, comparing site hydrology and pollutant loads to an undeveloped property with a similar drainage area and location. Compared to the undeveloped property, the developed lot had a 68% greater runoff, 95% greater total suspended solids (TSS), and 66-88% higher nutrient loadings for nitrogen and phosphorus. Line et al. (2002) sampled water quality from various land uses, and determined pollutant export rates for total nitrogen (TN), total phosphorus (TP), and TSS for undeveloped and developed conditions (Table 1.1). Concentrations and loadings for each pollutant increased from undeveloped conditions to developed conditions in both studies by Line et al. (2002) and Line and White (2007), with TN, TP, and TSS pollutant export rates increasing threefold.

Table 1.1: Pollutant export rates of various residential land uses (Line et al., 2002)

Development Type	Pollutant Export Rate (kg/ha/yr)		
	TN	TP	TSS
Undeveloped	11.4	1.0	986
Developed	30.5	3.0	2535

Several states, such as North Carolina and Maryland, have implemented stormwater control design requirements addressing postdevelopment stormwater quantity and quality issues. Requirements include peak flow attenuation, stormwater runoff volume control, and nutrient loading reduction (NCDENR, 2009a; MDE, 2009). The U.S. has termed waterways and watersheds with a history of high nutrient loading “Nutrient Sensitive Waters”, or NSWs. The Tar-Pamlico River Basin was the first watershed to be added to North Carolina’s NSW list in 1991 (NCDENR, 1990), followed by the Neuse River Basin in 1997 (NCDENR, 1999), the Jordan Lake Watershed in 2009 (NCDENR, 2009b), and most recently, the Falls Lake Watershed in 2010 (NCDENR, 2012).

North Carolina’s 16,000-km² Neuse River Basin, encompassing a large expanse of agricultural land (approximately one-third of the watershed) and the Raleigh-Durham metropolitan area, has a history of poor water quality and fish kills due to nitrogen loading. Consequently, the Neuse River Basin is listed on the state’s 303(d) list of impaired watersheds, a list of threatened waters compiled by the North Carolina Department of Natural and Environmental Resources (NCDENR) and submitted to the USEPA, per requirement of the Clean Water Act. As a result, the Neuse Stormwater Rule was enacted in 1997 for new development within the watershed to reduce nitrogen loadings 30% across the watershed (NCDENR, 1999). To develop in the Neuse River Basin, a stormwater management plan must be implemented addressing stormwater runoff quality and quantity of the proposed development. To achieve this goal, nitrogen loadings from new development must not exceed 4.0 kg/ha/year (3.6 lb/ac/year; NCDENR, 1999). The Neuse Stormwater Rule also

requires that the peak flow rate must not increase from predevelopment conditions for the 1-year return period, 24-hour duration storm. Further stormwater rules for the city of Raleigh include peak flow attenuation of the 2- and 10-year storm events (City of Raleigh, 2002).

Nutrient and hydrological reduction is met either through source reduction, which reduces influent pollutant loadings, or the use of stormwater control measures (SCMs) to capture and treat stormwater runoff (Stanley, 1996). Source reduction through decreased impervious areas, public education of stormwater issues, and routine maintenance of streets and storm drains are effective non-structural measures to reduce stormwater issues (NCDENR, 2009a). SCMs mitigate effluent water quality and quantity through peak flow mitigation, runoff volume control, and nutrient removal mechanisms of particle settling, nitrification, and denitrification.

1.2 Conventional Stormwater Runoff Control

A conventional practice to mitigate stormwater runoff quantity and quality on site is the installation of large-scale SCMs, such as dry detention basins (dry ponds) and wet retention basins (wet ponds). These SCMs are designed to mitigate peak flows and allow for a small amount of runoff volume reduction through evaporation, as well as nutrient mitigation through particle settling and biological activity. Large-scale SCMs do not, however, mimic predevelopment hydrology, and their impact on water quality is variable, due to infrequent maintenance and basin design issues. Issues such as erosion and scour, insufficient detention or retention time, and inadequate basin sizing cause diminished performance through

resuspension of TSS particles and insufficient detention time (Fortunato et al., 2005; Jonathan et al., 1993; Stanley, 1996; Tillinghast et al., 2011).



Figure 1.1: Dry detention basin in Raleigh, NC

Dry detention basins, illustrated in Figure 1.1, are designed to mitigate the peak runoff during a storm and release the water at a slower rate to a storm drainage network (Roesner et al., 2001). Dry detention basins are designed to only hold water intra-event, staying dry otherwise. A major advantage of this SCM is its ability to effectively reduce peak flows from both small and large watersheds, typically up to 25 acres (NCDENR, 2009a). Dry detention basins successfully remove TSS and particle-bound nutrients through sedimentation (Middleton and Barrett, 2011; Hossein et al., 2005; Stanley, 1996). TSS removal efficiencies of 68% - 99% were observed at a dry detention basin in Washington

(Hossain et al., 2005). Along with TSS, particle-bound nitrogen and phosphorus species settle and were generally removed, while dissolved nutrients have not been removed through dry detention (Pettersson, 1998). In a study of a dry detention basin in Greenville, NC, removal efficiencies of 71% for TSS, 45% for particulate nitrogen, and 33% for particulate phosphorus were shown. Dissolved nutrient concentrations did not statistically improve in the basin, with the exception of a 25% removal of dissolved phosphorus (Stanley, 1996). Middleton and Barrett (2008) studied a dry detention pond in Austin, TX. They found statistically significant removals of TSS (91%), TP (52%), total Kjeldahl nitrogen (TKN) (35%), and dissolved phosphorus (orthophosphate, Ortho-P) (7%).

Wet retention basins differ from dry detention basins only in that they hold a permanent pool of water throughout the year, allowing sufficient retention time for nitrification, denitrification, and/or particle settling to occur. Wet retention basins are not designed to infiltrate runoff water, but to capture and release stormwater runoff slowly and mitigate peak flow rates. With the exception of evapotranspiration, runoff volume is not mitigated (NCDENR, 2009a). A wet retention basin in Virginia outperformed a nearby dry detention basin in terms of mitigating TSS, nitrate+nitrite (NO_2+NO_3), ammonium (NH_4^+), total particulate phosphorus, and TP (Fortunato et al., 2005). Though removal of some species of nitrogen is shown under certain conditions (Winston et al., 2013), calculation of removal efficiencies for retention basins is difficult to quantify due to varying land uses and nutrient removal performance (Chen and Adams, 2006). In Durham, North Carolina, two wet retention basins treating I-85 highway runoff and North Carolina Museum of Life and

Science parking lot runoff were studied. The former reduced TN, TP, and TSS by 36%, 36%, and 92%, respectively, while the latter had 59%, 57%, and 89% reductions, respectively (Winston et al., 2013). A retention basin in New Jersey showed seasonal variation as a strong covariate in determining pollutant reduction (Rosenzweig et al., 2011). For example, “reductions” of NO_3^- , NH_4^+ , and TN were found to be -38%, 60%, and -10% during the winter and 19%, -10%, and 45% during the spring, respectively (Rozenweig et al., 2011). Seasonal variation in nutrient loads may be a result of fertilization patterns and temperature and climate variation affecting nutrient cycling.

Conventional SCMs successfully mitigate peak runoff and account for some particulate settling; however, drawbacks of conventional SCMs include a relative lack of nutrient removal and stormwater infiltration (Fortunato et al., 2005; Stanley, 1996; Jonathan et al., 1993). In addition, dry detention and retention basins may be required to occupy up to 8% of a property without consideration to parking or landscape benefit. Because of the land requirement and lack of multiple uses, conventional SCMs may not be ideal in areas of high land cost or land scarcity (NCDENR, 2009a).

1.3 Low Impact Development

Low Impact Development (LID), a term coined by Prince George’s County, MD (1999), describes an alternative approach to stormwater management, focusing on mitigating stormwater impacts onsite and mimicking predevelopment hydrology entirely. Restoring the predevelopment hydrology of a site requires on increasing evapotranspiration and/or

infiltration, decreasing runoff volumes and peak flows, and addressing water reuse (Perrin et al., 2009). Rather than a sole focus on peak flow mitigation, LID SCMs generally infiltrate stormwater, capturing stormwater and its associated pollutants onsite and allowing for groundwater recharge, runoff volume mitigation, and nutrient removal mechanisms (Dietz, 2007). Individual LID SCMs such as bioretention, swales, permeable pavement, and rainwater harvesting have been shown to be effective, sometimes more effective than their storage-based conventional SCMs, in capturing and detaining runoff volumes from the small (<50 mm, or 2 in.) storm events, but decrease in effectiveness as the storm size increases (Damodaram et al., 2010).



Figure 1.2: A grassed bioretention cell in Raleigh, NC

Bioretention is an infiltration-based SCM used for its benefits in runoff quantity and quality improvement (Davis et al., 2009; Li et al, 2009; Hunt et al., 2012). This SCM features a

depressed area for storage of stormwater, a layer of permeable soil media for filtration of water, vegetation for biological nutrient removal mechanisms and gross-filtration, and in some cases an underdrain with an upturned elbow for increased N and P removal (Brown and Hunt, 2011; Davis et al., 2009; Hunt et al., 2012; NCDENR, 2009a). Bioretention systems have been proven to reduce peak flows up to 99%, reduce runoff volumes by 90%, and reduce TSS, TN, and TP loadings up to 99%, especially during storms smaller than the design event (typically 25 mm; Li et al., 2009; Hunt et al., 2008; Davis et al., 2006; USEPA, 2006). Figure 1.2 pictures the bioretention cell installed at the studied site in Raleigh, NC.

Grassed swales are conveyance channels for stormwater runoff, usually vegetated for gross-filtration of pollutants (Figure 1.3). This SCM has a small land requirement, as grassed swales typically run linearly between residential houses, or along roadways and highways (NCDENR, 2009a). Water quality improvements have been shown in roadside swales when compared to untreated roadways (Winston et al., 2012; Deletic and Fletcher, 2006; Barrett et al., 1998). Fifty percent nitrate ($\text{NO}_3\text{-N}$), 50% total ammoniacal nitrogen (TAN), 50% TKN, 50% TP and >90% TSS were found in roadside swales in NC and Texas (Winston et al., 2012; Barrett et al., 1998).



Figure 1.3: A grassed swale in Raleigh, NC

Another infiltration-based SCM is permeable pavement (Figure 1.4). This SCM consists of a permeable hardscape with a gravel underground storage layer for storing and infiltrating stormwater. Three types of permeable pavements: permeable interlocking concrete pavers, porous concrete, and concrete grid pavers, reduced runoff volumes and peak discharges when compared against impervious asphalt, with all pervious pavements reducing runoff volume greater than 98% (Collins et al., 2008; Wardynski et al., 2012). In addition to hydrologic impacts, permeable pavements were also shown to produce significantly smaller pollutant concentrations than asphalt paving. Because of their ability to mitigate runoff, pervious pavements can reduce nutrient loading to below a detectable level, or to zero in cases of no outflow (Bean et al., 2007). Permeable pavements can also be driven on, making them a favorite of LID “designers” looking to reduce site impact.



Figure 1.4: Permeable interlocking concrete pavers in Boone, NC

Infiltration structures, including basins and trenches, are designed to collect stormwater runoff, allowing the water to infiltrate into the underlying soil on site rather than run off into a storm drainage system, reducing runoff volumes and peak flow rates (Akan, 2002; NCDENR, 2009). Infiltration structures are given credit by NCDENR (2009a) to remove pollutant loads at rates of 85% TSS, 30% TN, and 35% TP due to their potential for particle settling and infiltration. Infiltration basins are constructed similarly to dry detention basins, except they lack an outflow structure, because they rely completely on infiltration. Birch et al. (2005) found a 51% removal of TP and a 65% removal rate for TKN in an infiltration basin in Australia, exceeding NCDENR credits. Infiltration trenches, usually lined with coarse gravel, and can be constructed either underground or aboveground, depending on land availability. When constructed along a roadside in Korea, removal rates of total suspended solids and associated sediment-bound pollutants including TN and TP were observed at 80-

90% (Maniquiz et al., 2010). In New Hampshire, TSS removal rates of 95% were found for an underground infiltration trench system (Roseen et al., 2006).

LID emphasizes water use onsite through the use of rainwater harvesting (RWH) systems (Figure 1.5). The primary objective for RWH systems is to provide an alternative water source for non-potable uses such as irrigation, toilet flushing, car washing, and laundry rather than using potable water. Up to a 70% reduction in the demand for potable water can be expected for a LID incorporating RWH systems versus a development without the RWH (van Roon, 2007). This reduced demand for potable water in communities with municipal water supply reduces dependence on large-scale water treatment facilities.



Figure 1.5: RWH cisterns in Raleigh, NC, from left: (a) aboveground, and (b) underground

Although streets and parking lots are the majority of impervious surfaces, rooftops encompass approximately 20% of impervious surfaces (Bannerman et al., 1993). Roof runoff consists of various pollutants and nutrients, dependent upon rooftop material and length of antecedent dry periods (Egodawatta et al, 2009). Historically implemented in arid or semi-arid regions, RWH has recently surged in popularity in more humid regions, such as the southeastern U.S., due to increased interest in water conservation, severe drought conditions, and restrictions on lawn irrigation (Jones and Hunt, 2010). Varying the size and usage of RWH systems can affect the efficiency, water usage, and pollutant removal of the SCM (DeBusk et al., 2013; Jones and Hunt, 2010; Khastagir and Jayasuriya, 2010). The RWH system monitored at Craven County Animal Shelter in New Bern, N.C. was sized for a dedicated, year-round water usage inside the shelter, and therefore demonstrated the highest water usage and efficiency (DeBusk et al. 2013). In contrast, a system designed for irrigation showed increased reliance on rainfall frequency because of the smaller tank size and lack of usage outside of the growing season (Jones and Hunt, 2010).

If a RWH system is designed and used properly, regulatory credit is given by NCDENR (2009a) for mitigation of runoff volume and peak flow rates. Rooftop runoff captured by RWH systems contains pollutants such as metals, organics, sediment, and microbiological contaminants (Abbasi and Abbasi, 2011). Though no regulatory credit is currently given to RWH for nutrient removal, particle-bound TN (81%) and TP (90%), as well as TSS (97%) reductions have been observed due to sedimentation in the cisterns (Khastagir and Jayasuriya, 2010, NCDENR, 2009a). Rainwater harvesting not only provides for a safe and reliable

water supply when designed properly, but it also has the potential for mitigation of stormwater runoff as well as sediment-bound pollutants.

Through the use of individual LID SCMs, runoff quality and quantity improvements are observed. Vegetated SCMs such as bioretention cells and swales successfully mitigated pollutant loadings into waterways, while pervious pavement and infiltration structures reduced peak flow rates and runoff volumes. Research is still lacking on combining multiple SCMs with a large scale commercial LID.

1.4 Impact of Development Type on Site Hydrology

Since LID focuses on disconnecting impervious areas such as streets and rooftops, and managing this water on a local scale through the installation of various infiltration and harvesting-based SCMs, runoff volume exiting a conventional development is much greater than the volume exiting a LID (Bedan and Clausen, 2009; Dietz and Clausen, 2008; Hood et al., 2007; Line et al., 2012). Brander et al. (2004) modeled the effect of adding infiltration practices, such as bioretention and swales, to various types of developments, resulting in a reduction in runoff of up to 85% in all development types. The Jordan Cove Urban Watershed Project, which monitored a LID and conventional site, compared postdevelopment peak flow and runoff volume from both the LID and conventional site to an undeveloped control site. Runoff volume from the conventional development increased in magnitude 600 times from its predevelopment conditions; whereas, LID runoff volume was reduced by 42% (Bedan and Clausen, 2009).

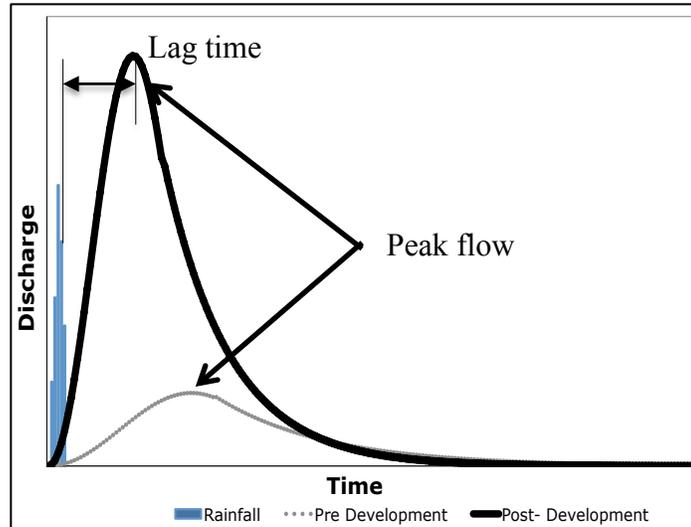


Figure 1.6: A single-event flow hydrograph

Peak flow, or peak discharge, is defined as the largest discharge measured during a storm event (Figure 1.6). In a study by Hood et al. (2007), the peak discharge was eleven times greater in a conventional development with 1.5 times more impervious area than a LID, indicating that rather than the percentage imperviousness, disconnecting impervious areas impacts peak discharge. LID peak flows were significantly less than the control watershed as well. At the Jordan Cove Urban Watershed Project, peak discharge increased 2800% for conventional development, but decreased 26% for the LID (Bedan and Clausen, 2009).

Lag time, one important indicator of watershed health (Leopold, 1991), is measured as the time from the center of mass of rainfall until the peak flow is reached (Figure 1.1). Many factors contribute to the lag time of a watershed, including watershed size, soils, geology, slope, and land use (Dingman, 2002). A small lag time denotes an urbanized watershed with a higher amount of directly connected impervious surfaces, allowing stormwater to move

more quickly than a less urbanized watershed (Kang et al., 1998). For small storms (<1”) of short duration (<4h), the lag time of a LID was shown to be three times greater than that of a conventional development (Hood et al., 2007), indicating the LID exhibited more similar characteristics to a natural, undeveloped watershed than the conventional development.

To determine the percentage of runoff generated from a precipitation event, a runoff coefficient for a storm is generated from a ratio of the runoff generated to precipitation (Equation 1.1).

$$\text{Runoff coefficient} = \frac{\text{Runoff Depth}}{\text{Rainfall Depth}} \quad (1.1)$$

Hood et al. (2007) found that a LID produced a significantly smaller runoff coefficient (0.067) than both the conventional development (0.239) and the control (undeveloped) (0.193) scenarios. At the Jordan Cove Urban Watershed Project, though the LID received a higher amount of rainfall, LID runoff coefficients did not change from predevelopment conditions through development with 21% impervious surfaces. However, an exponential relationship was found between the runoff coefficient and impervious area at the conventional development (Dietz and Clausen, 2007). Line et al. (2012) observed a slightly higher runoff coefficient at a commercial LID site (0.58) than a conventional commercial site treated with a wet retention basin (0.44). The high LID runoff coefficient reported by Line et al. (2012) was speculated to be a result of malfunctioning and undersized LID SCMs. Another study by Rushton (2001) incorporated swales and basins into various types of pavement in a large parking lot in Florida and found that an asphalt parking lot with a swale

and detention basin had average runoff coefficients of 0.16 and 0.35 for the two parking lots monitored.

Infiltration-based LID combined with land preservation resulted in a hydrology similar to the predevelopment conditions in a study by Williams and Wise (2006). Studies by Bedan and Clausen (2009), Dietz and Clausen (2007), Hood et al. (2007), and Line et al. (2012) confirm that LID can outperform both conventional developments and, in some cases, undeveloped conditions based on common hydrologic metrics.

1.5 Impact of Development Type on Site Water Quality

Because individual LID SCMs mitigate nutrients, the potential for water quality improvement of a LID is presumed to be at least as high. Event mean concentrations (EMCs), a metric of evaluating water quality, are average effluent flow-weighted concentrations of a storm event. EMCs of LID and conventional commercial developments, as well as EMCs from parking lot runoff and from commercial sites in the National Stormwater Quality Database (NSQD), are listed in Table 1.2 (Passeport and Hunt, 2009; Pitt et al., 2004). As shown, parking lot runoff as well as runoff from the conventional and LID sites studied had lower EMCs than data reported in the NSQD (Line et al., 2012; Passeport and Hunt, 2009). With the exception of NO_2+NO_3 , the LID site had lower mean effluent EMCs than the conventional development.

Table 1.2: Summary of commercial EMCs.

Development type	TKN (mg/L)	TAN (mg/L)	NO ₂ +NO ₃ (mg/L)	TP (mg/L)	Ortho-P (mg/L)	TSS (mg/L)
NSQD ^a	1.6	0.50	0.6	0.22	0.11	42
Parking lot runoff ^b	1.19	0.32	0.36	0.19	0.07	-
Commercial: Conventional ^c	0.87	0.13	0.13	0.09	-	31.1
Commercial: LID ^c	0.69	0.06	0.56	0.06	0.01	18.4
Residential: Conventional ^d	1	0.15	0.3	0.185	-	24
Residential: LID ^d	1.3	0.03	0.4	0.291	-	11

^aNational Stormwater Quality Database, median commercial development (Pitt et al., 2004)

^bEight parking lots in NC (Passeport and Hunt, 2009)

^cCommercial Development in NC (Line et al, 2012)

^dJordan Cove Residential Development (Bedan and Clausen, 2009)

The commercial LID site (76% impervious) studied by Line et al. (2012) exhibited a reduction in all pollutant concentrations between in the inlet and the outlet, showing pollutant treatment by bioretention cells, pervious concrete, and constructed stormwater wetlands used onsite. As compared to average parking lot runoff from eight parking lots in central North Carolina, EMCs of runoff leaving the LID site were lower for TKN, TAN, TP, Ortho-P, and TSS, and higher for NO₂+NO₃ (Line et al., 2012). The difference in NO₂+NO₃ concentrations between the parking lots and the LID outlet may have been attributed to nitrogen cycle processes (Table 1.2).

The residential conventional development and LID studied in the Jordan Cove reported a different trend in water quality between development type than the commercial developments. In the residential conventional development (32% impervious), no significant change in NO₂+NO₃ and TAN concentrations were found between inflow and outflow, but a significant decrease in TKN, TP and TSS concentrations were observed (Bedan and Clausen,

2009). In the residential LID (22% impervious), three of the five pollutants studied *increased* from inflow to outflow, likely due to organic nitrogen and fertilization inputs from the grassed swales (Bedan and Clausen, 2009).

Due to a combination of nutrient removal mechanisms and runoff volume reduction, LIDs significantly mitigate mass exports of pollutants, including nutrients, metals, bacteria, and sediment (Bedan and Clausen, 2009; Dietz et al., 2008; Hunt et al., 2008; Line et al., 2012). Dietz et al. (2008) sampled a LID and a conventional site, revealing that as impervious land cover increased during construction, NO_2+NO_3 , TN, and TP export increased exponentially in conventional development, but no change in pollutant export was found from the LID. For TAN in particular, a decrease in export was observed in response to increased impervious areas in LID while TAN loads continued to increase in the conventional development. A similar trend was shown in a study of the same watershed post-construction by Bedan and Clausen (2009): as stormwater volume increased from the conventional watershed, so did nutrients (NO_2+NO_3 , TAN, TKN, and TP) and TSS from predevelopment to postdevelopment; whereas, the LID increased mass exports of TSS and TP and decreased TKN and TAN. The conventional development increased TP mass loadings by more than 76,000%, while the LID increased TP 939%. Bedan and Clausen (2009) speculated the increase in TP loading at the LID site was likely due to fertilization of the grassed swales located onsite.

1.6 Conclusions

Although conventional SCMs such as dry detention and wet retention basins both mitigate peak flows and remove sediment and (to some extent) nutrients, LID was introduced to mimic predevelopment hydrology, thereby mitigating pollutant loads. Though limited literature exists comparing LID to conventional development, research on individual infiltration-based SCMs, such as bioretention, swales, and infiltration trenches/basins, show high potential for mitigation of runoff volume, peak flow rates, and nutrient loadings. Existing literature suggests that LID reduces peak flow and runoff volumes, sometimes beyond predevelopment conditions, while showing a longer lag time and runoff coefficient than a conventional development (Bedan and Clausen, 2009; Brander et al., 2004; Dietz and Clausen, 2007; Hood et al., 2007; Williams and Wise, 2006). LID also proves capable of reducing pollutant concentrations, and moreover pollutant loadings, of nitrogen species, phosphorus species, and TSS (Bedan and Clausen, 2009; Dietz and Clausen, 2007; Line et al., 2012).

Commercial development, comprised of approximately 80% impervious surfaces on average (Pitt et al., 2004), presents a challenge to stormwater designers to maximize land use for buildings and parking, while maintaining space for required stormwater control. One space-saving technique is the installation of permeable pavement; however, limited research exists on the hydrologic performance of permeable pavement low-permeability soils, such as the N.C. piedmont. Another technique is the installation of underground SCMs to detain and infiltrate stormwater for hydrologic mitigation. In this study, two commercial sites: a

conventional development (centralized stormwater management) and an infiltration-based LID in Raleigh, N.C., were compared with respect to hydrology and water quality performance.

1.7 References

Abbasi, T. and Abbasi, S.A. (2011). "Sources of pollution in rooftop rainwater harvesting systems and their control." *Critical Reviews in Environmental Science and Technology*, 41(23), 2097-2167.

Akan, A. (2002). "Modified rational method for sizing infiltration structures." *Canadian Journal of Civil Engineering*, 29(4), 539-542.

Bannerman, R.T., Owens, D.W., Dodds, R.B. and Hornewer, N.J. (1993). "Sources of pollutants in Wisconsin stormwater." *Wat. Sci. Tech*, 28(3-5), 241-259.

Barrett, M.E., Walsh, P.M., Malina, J.F. and Charbeneau, R.J. (1998). "Performance of vegetative controls for treating highway runoff." *J. Environ. Eng.*, 124(11), 1121-1128.

Bean, E.Z., Hunt, W.F. and Bidelspach, D.A. (2007). "Evaluation of four permeable pavement sites in Eastern North Carolina for runoff reduction and water quality impacts." *Journal of Irrigation and Drainage Engineering*, 133(6), 583-592.

Bedan, E.S. and Clausen, J.C. (2009). "Stormwater runoff quality and quantity from traditional and low impact development watersheds." *J.Am. Water Resour.Assoc.*, 45(4), 998-1008.

Birch, G.F., Fazeli, M.S. and Matthai, C. (2005). "Efficiency of an infiltration basin in removing contaminants from urban stormwater." *Environmental Monitoring and Assessment*, 101(1-3), 23-38.

Brander, K., Owen, K. and Potter, K. (2004). "Modeled impacts of development type on runoff volume and infiltration performance." *J.Am. Water Resour.Assoc.*, 40(4), 961-969.

Brown, R.A. and Hunt, W.F. (2011). "Underdrain configuration to enhance bioretention exfiltration to reduce pollutant loads." *J.Environ.Eng.*, 137(11), 1082-1091.

Chen, J. and Adams, B.J. (2006). "Urban stormwater quality control analysis with detention ponds." *Water Environment Research*, 78(7), 744-753.

City of Raleigh (1999). "Stormwater Management Design Manual."

Clean Water Act of 1972, 33 U.S.C. § 1251 et seq.

- Collins, K.A., Hunt, W.F. and Hathaway, J.M. (2008). "Hydrologic comparison of four types of permeable pavement and standard asphalt in Eastern North Carolina." *J. Hydrol. Eng.*, 13(12), 1146-1157.
- Damodaram, C., Giacomoni, M.H., Khedun, C.P., Holmes, H., Ryan, A., Saour, W. and Zechman, E. M. (2010). "Simulation of combined best management practices and low impact development for sustainable stormwater management." *J. Am. Water Resour. Assoc.*, 46(5), 907-918.
- Davis, A.P., Hunt, W.F., Traver, R.G. and Clar, M. (2009). "Bioretention technology: Overview of current practice and future needs." *J. Environ. Eng.*, 135(3), 109-117.
- Davis, A.P., Shokouhian, M., Sharma, H. and Minami, C. (2006). "Water quality improvement through bioretention media: Nitrogen and phosphorus removal." *Water Environ. Res.*, 78(3), 284-293.
- Deletic, A. and Fletcher, T.D. (2006). "Performance of grassed filters used for stormwater treatment – a field and monitoring study." *J. Hydrol. (Amsterdam)*, 317(3-4), 261-275.
- DeBusk, K.M., Hunt, W.F. and Wright, J.D. (2013). "Characterization of rainwater harvesting performance in humid regions of the United States." *J.Am.Water Resour.Assoc.*, (accepted February 2013).
- Dietz, M.E. (2007). "Low impact development practices: A review of current research and recommendations for future directions." *Water Air and Soil Pollution*, 186(1-4), 351-363.
- Dietz, M.E. and Clausen, J.C. (2008). "Stormwater runoff and export changes with development in a traditional and low impact subdivision." *J.Environ.Manage.*, 87(4), 560-566.
- Dingman, S.L. (2002). *Physical Hydrology* (2nd Edition), Macmillan, New York.
- Egodawatta, P., Thomas, E. and Goonetilleke, A. (2009). "Understanding the physical processes of pollutant build-up and wash-off on roof surfaces." *Sci.Total Environ.*, 407(6), 1834-1841.
- Fortunato, C., McDonough, O. and Chambers, R. (2005). "The effectiveness of dry and wet stormwater detention basins as sediment and nutrient processors." In *ASCE Conference Proceedings*, 178(98), 1-12.
- Hood, M.J., Clausen, J.C. and Warner, G.S. (2007). "Comparison of stormwater lag times for low impact and traditional residential development." *J.Am.Water Resour.Assoc.*, 43(4), 1036-1046.

- Hossain, M.A., Alam, M., Yonge, D.R. and Dutta, P. (2005). "Efficiency and flow regime of a highway stormwater detention pond in Washington, USA" *Water, Air, and Soil Pollution*, 164(1), 79-89.
- Hunt, W.F., Smith, J.T., Jadlocki, S.J., Hathaway, J.M. and Eubanks, P.R. (2008). "Pollutant removal and peak flow mitigation by a bioretention cell in urban Charlotte, NC." *J. Environ. Eng.*, 134(5), 403-408.
- Hunt, W.F., Davis, A.P. and Traver, R.G. (2012). "Meeting hydrologic and water quality goals through targeted bioretention design." *J. Environ. Eng.*, 138(6), 698-707.
- Jonathan, C., Bryant, G.J. and Andrews, D.J. (1993). "Ontario Stormwater Best Management Practices (BMP) Planning and Design Manual." In *Proceedings of the 6th International Conference on Urban Storm Drainage, Niagara Falls, Ontario, Canada, Volume II*; Marsalek, J., Tornom H.C. (Eds.) IAHR/IAWQ Joint Committee on Urban Drainage, Seapoint Publishing: Victoria, British Columbia, Canada, 1055-1061.
- Jones M.P. and Hunt, W.F. (2010). "Performance of rainwater harvesting systems in the Southeastern United States." *Resources Conservation and Recycling*, 54, 623-629.
- Kang, S., Park, J. and Singh, V. (1998). "Effect of urbanization on runoff characteristics of the On-Cheon stream watershed in Pusan, Korea." *Hydrological Processes*, 12, 351-363.
- Khastagir, A. and Jayasuriya, L.N.N. (2010). "Impacts of using rainwater tanks on stormwater harvesting and runoff quality." *Water Sci. Technol.*, 62(2), 324-329.
- Leopold, L. (1991). "Lag Time for Small Drainage Basins." *Catena*, 18(2), 157-171.
- Li, H., Sharkey, L.J., Hunt, W.F. and Davis, A.P. (2009). "Mitigation of impervious surface hydrology using bioretention in North Carolina and Maryland." *J. Hydrol. Eng.*, 14(4), 407-415.
- Line, D.E., Brown, R.A., Hunt, W.F. and Lord, W.G. (2012). "Effectiveness of LID for commercial development in North Carolina." *J. Environ. Eng.*, 138(6), 680-688.
- Line, D.E. and White, N.M. (2007). "Effects of development on runoff and pollutant export." *Water Environ. Res.*, 79(2), 185-190.
- Line, D.E., White, N.M., Osmond, D.L., Jennings, G.D. and Mojonier, C.B. (2002). "Pollutant export from various land uses in the upper Neuse River Basin." *Water Environ. Res.*, 74(1), 100-108.

Maniquiz, M.C., Lee, S. and Kim, L. (2010). "Long-term monitoring of infiltration trench for nonpoint source pollution control." *Water Air and Soil Pollution*, 212(1-4), 13-26.

MDE (Maryland Department of the Environment). (2009). "Maryland Stormwater Design Manual."

Middleton, J.R. and Barrett, M.E. (2008). "Water quality performance of a batch-type stormwater detention pond." *Water Environ. Res.*, 80(2), 172-178.

NCDENR (North Carolina Department of Natural Resources). (2010). "Falls reservoir water supply nutrient strategy: stormwater management for new development."

NCDENR (North Carolina Department of Natural Resources). (2009a). "Jordan water supply nutrient strategy rules."

NCDENR (North Carolina Department of Environmental and Natural Resources). (2009b). "Stormwater Best Management Practices Manual."

NCDENR (North Carolina Department of Natural Resources). (2001). "Tar-Pamlico river basin – nutrient sensitive waters management strategy."

NCDENR (North Carolina Department of Natural Resources). (1999). "Neuse River Basin: Model Stormwater Program for Nitrogen Control."

Passeport, E. and Hunt, W.F. (2009). "Asphalt parking lot nutrient characterization for eight sites in North Carolina, USA." *J. of Hydrol. Eng.*, 14(4), 352-361.

Perrin, C., Milburn, L., Szpir, L., Hunt, W.F., Bruce, S., McLendon, R., Job, S., Line, D., Lindbo, D., Smutko, S., Fisher, H., Tucker, R., Calabria, J., DeBusk, K., Cone, K.C., Smith-Gordon, M., Spooner, J., Blue, T., Deal, N., Lynn, J., Rashash, D., Rubin, R., Senior, M., White, N., Jones, D. and Eaker, W. (2009). "Low Impact Development: A Guidebook for North Carolina (AG-716)." NC Cooperative Extension Service, NC State University. website: <http://www.ncsu.edu/WECO/LID>

Pettersson, T.J.R. (1998). "Water quality improvement in a small stormwater detention pond." *Wat. Sci. Tech.*, 38(10), 115-122.

Pitt, R., Maestre A. and Morquecho R. (2004) "Evaluation of NPDES phase I Municipal Stormwater Monitoring Data" In: National Conference on Urban Stormwater: Enhancing the Programs at the Local Level. *EPA/625/R-03/003*.

Prince George's County. (1999). "Low-Impact Development design strategies. An integrated design approach."

- Roesner, L., Bledsoe, B. and Brashear, R. (2001). "Are best-management-practice criteria really environmentally friendly?" *Journal of Water Resources Planning and Management*, 127(3), 150-154.
- Roseen, R.M., Ballestro, T.P., Houle, J.J., Avelleneda, P., Wildey, R. and Briggs, J. (2006). "Storm water low-impact development, conventional structural, and manufactured treatment strategies for parking lot runoff." *Transportation Research Record: Journal of the Transportation Research Board*, No. 1984, 135-147.
- Rozenweig, B.R., Smith, J.A., Baeck, M.L. and Jaffé, P.R. (2011). "Monitoring nitrogen loading and retention in an urban stormwater detention pond." *J. Environ. Qual.*, 40(2), 598-609.
- Stanley, D.W. (1996). "Pollutant removal by a stormwater dry detention pond." *Water Environment Research*, 68(6), 1076-1083.
- Tillinghast, E.D., Hunt, W.F. and Jennings, G.D. (2011). "Stormwater Control Measure (SCM) Design Standards to Limit Stream Erosion for Piedmont North Carolina." *Journal of Hydrology*, 411(3-4), 185-196.
- USEPA. (2009). "National Water Quality Inventory: 2004 Report to Congress." *EPA-841-R-08-001*, Washington, D.C.
- van Roon, M. (2007). "Water localisation and reclamation: Steps towards low impact urban design and development." *J. Environ. Manage.*, 83(4), 437-447.
- Wardynski, B.J., Winston, R.J. and Hunt, W.F. (2012). "Internal water storage enhances exfiltration and thermal load reduction from permeable pavement in the North Carolina mountains." *Journal of Environmental Engineering*, 139(2), 187-195.
- Williams, E. and Wise, W. (2006). "Hydrologic impacts of alternative approaches to storm water management and land development." *J. Am. Water Resour. Assoc.*, 42(2), 443-455.
- Winston, R.J., Hunt, W.F., Kennedy, S.G., Wright, J.D. and Lauffer, M.S. (2012). "Field evaluation of storm-water control measures for highway runoff treatment." *J. Environ. Eng.*, 138(1), 101-111.
- Winston, R.J., Hunt, W.F., Kennedy, S.G., Merriman, L.S., Chandler, J. and Brown, D. (2013). "Evaluation of Floating Treatment Wetlands as Retrofits to Existing Stormwater Retention Ponds" *Ecological Engineering* (accepted – January 2013).

Chapter 2: Hydrologic Comparison of an Urban Commercial Low Impact Development and a Conventional Commercial Development

2.1 Abstract

Urbanization and its associated increased impervious footprint lead to stream impairment through erosion, flooding, and augmented pollutant loads. Low Impact Development (LID) focuses on disconnecting impervious areas, increasing infiltration and evapotranspiration, and reusing stormwater on site through the use of stormwater control measures (SCMs). In this study, a conventional development (centralized stormwater management) and an adjoining infiltration-based LID commercial site in Raleigh, North Carolina, were compared with respect to hydrology. The conventional development (2.76 ha, 61% directly connected impervious area (DCIA)) and the LID (2.53 ha, 84% DCIA) both had underlying hydrologic soil group B soils, Cecil and Appling, respectively. A dry detention basin, designed to mitigate peak flow rate, was the conventional development SCM. The LID site consisted of a 44,300-liter aboveground cistern used for indoor toilet flushing, two underground cisterns (57,900 liters and 60,600 liters, used for landscape irrigation), and an underground detention system, which overflowed into a series of infiltration galleries beneath the parking lot of the shopping center. The LID shopping center was designed to mimic predevelopment hydrology for the 10-year return period, 24-hour duration storm. For the 47 events monitored, runoff coefficients of 0.02 at the LID site and 0.49 at the conventional site were observed, when normalized by DCIA. The conventional development had an 11 times higher peak flow value for the median storm, than the LID site when normalized by DCIA. For three storms

more intense than the 10-year storm, the conventional site averaged a 7.7 times higher peak flow than the LID, when normalized by DCIA. Results from this innovative combined detention, stormwater harvesting, and infiltration LID system demonstrate highly effective and space-saving solutions for areas where aboveground SCMs, such as bioretention and constructed stormwater wetlands, are not feasible due to high land costs.

Keywords. Low impact development, L.I.D., stormwater management, urban commercial infill, infiltration, dry detention basin, stormwater runoff, peak flow, volume reduction, runoff coefficient, North Carolina

2.2 Introduction

Urban stormwater runoff is one of the leading sources of impairment in waterways (USEPA, 2009). Increased impervious area due to urbanization, even if limited to 10% of the total watershed, increases flood frequency, increases discharge volume, and disturbs channel geomorphology (Moscrip and Montgomery, 1997). In CT, Dietz and Clausen (2008) found as the impervious area increased from 1% to 32% during construction, an increase in runoff of 4900% resulted. Due to increased flow rates, stream impacts such as incision and stream bank widening occur, causing stream erosion and impairment to aquatic ecosystems (Hammer, 1972; Wang and Lyons, 2003).

Low Impact Development (LID), a term coined by Prince George's County, MD (1999), describes an alternative approach to stormwater management, focusing on mitigating predevelopment hydrology entirely. Rather than a sole goal of on peak flow mitigation, LID stormwater control measures (SCMs) generally infiltrate stormwater, capturing stormwater and its associated pollutants onsite and allowing for evapotranspiration, groundwater recharge, runoff volume mitigation, and pollutant removal mechanisms (Bean et al., 2007; Collins et al., 2008; Davis et al., 2009; Dietz, 2007; Hunt et al., 2008; Winston et al., 2012). Because LID focuses on disconnecting impervious areas, runoff volume leaving a LID is substantially less than that from a conventionally developed site, and in some cases less than that of an undeveloped site (Bedan et al., 2009; Dietz and Clausen, 2008; Hood et al., 2007; Line et al., 2012). In a study by Hood et al. (2007), the peak discharge from a conventional residential development was 1100% of that from a residential LID, and when compared to a

control, or undeveloped, watershed, the LID peak discharge was only 20% of the undeveloped peak discharge. Hood et al. (2007) demonstrated that with residential LID it is possible to not only mimic predevelopment hydrology, but to reduce runoff volumes and peak discharges to below predevelopment conditions. The same residential study produced a smaller runoff coefficient (ratio of runoff to rainfall) (0.067) for residential LID than for both the conventional residential development (0.239) and the control (undeveloped) (0.193) watersheds (Hood et al., 2007).

Peer-reviewed literature on LID for commercial sites is somewhat limited. Most studies focus on a single SCM, one exception being Line et al. (2012) incorporating bioretention, permeable pavement, and constructed stormwater wetlands at a commercial development in North Carolina. This LID site was more effective at reducing pollutant loadings compared to a conventional wet detention basin in North Carolina; however, this LID site did not reduce outflow volumes more than the conventional wet pond due to an unexpectedly high infiltration rate at the wet detention basin, performing as more of a LID SCM than a typical wet detention basin (Line et al., 2012). Another study by Rushton (2001) incorporated swales and basins into various types of pavement (including permeable) in parking lots in Florida and found that an asphalt parking lot with a swale and detention basin had average runoff coefficients of 0.16 and 0.35 for the two parking lots. Runoff coefficients are typically 0.95 for impervious areas, such as parking lots, roadways, and rooftops (USDA, 2004). When swales and detention basins were incorporated into a parking lot design, this

number was reduced by a factor of 2.7 – 5.9 due to their infiltration capabilities (Rushton, 2001).

In areas of high land value or low land availability, large-scale commercial options for stormwater control may include incorporating underground SCMs to detain and infiltrate stormwater, but the performance of predominantly underground LID infrastructure has yet to be demonstrated. Moreover, the performance of a system combining large detention and infiltration-based SCMs with smaller LID SCMs has yet to be demonstrated for hydrologic mitigation.

This study compared the hydrology of two side-by-side commercial developments: (1) a conventional development treated with a simple swale and dry pond and (2) an innovative commercial LID consisting of aboveground and underground stormwater treatment. The LID site contained cisterns for water harvesting, aboveground swales and bioretention for water quality mitigation, and a large-scale underground detention and infiltration system.

2.3 Methodology

A conventionally developed commercial site (61% directly connected impervious area (DCIA)) and a water harvesting and infiltration-based LID commercial site (84% DCIA) were monitored post-construction from January 2012 – December 2012 for runoff quantity in Raleigh, North Carolina. Located in the piedmont region, Raleigh, the capital city, has a population of approximately 400,000 (U.S. Census Bureau, 2013). Normal average monthly temperatures range from 4.2°C in January – 25.9°C in August (SCO, 2012). Normal

precipitation for Raleigh is 1179 mm/year (SCO, 2012). The sites were adjacent to one another, so they were exposed to the same climatic conditions.

2.3.1 Site Descriptions

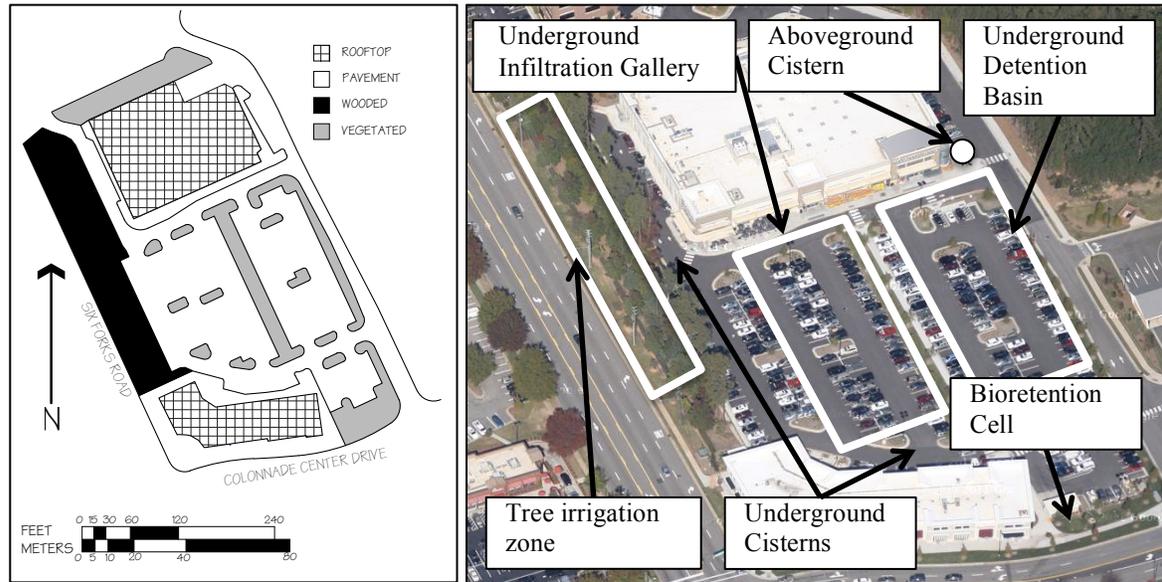


Figure 2.1: Commercial LID diagrams, from left: (a) watershed land use and characteristics, and (b) major underground SCMs and tree irrigation zone

The 2.5-ha commercial LID site, consisting of a mix of commercial businesses and restaurants, an asphalt parking lot, several vegetated parking islands, and a preserved natural wooded area, drained into a series of underground and aboveground SCMs (Figures 2.1a, 2.1b). The drainage area and land use characteristics of the site are found in Table 2.1. The LID site was intended to (1) mimic predevelopment peak discharges for the Type II, 2-year and 10-year return period storms (142 mm/hour and 181 mm/hr, respectively), and (2) capture and infiltrate runoff from the 25-mm water quality event.

Table 2.1: Characteristics of study sites examined in North Carolina

	LID	Conventional Development
Location	Intersection of Six Forks & Strickland Roads, Raleigh, NC (35.900°N, -78.651°W)	Intersection of Six Forks & Strickland Roads, Raleigh, NC (35.897°N, -78.649°W)
Year built	2011	2007
Watershed	Neuse River Basin	Neuse River Basin
Catchment composition	Commercial businesses, grocery store, and restaurant	Commercial businesses
Underlying soil	Appling (Hydr. Soil Group B)	Cecil (Hydr. Soil Group B)
Catchment area, ha	2.53	2.76
Grassed	0.13	1.09
Wooded	0.29	-
Rooftop	0.54	0.24
Parking	1.57	1.43
% of catchment as DCIA	84%	61%

Three cisterns, totaling approximately 162,800 L of storage, captured runoff from the rooftop surfaces and recycled the water onsite for indoor and outdoor uses. A 44,300-liter aboveground cistern and a 57,900-liter underground cistern each captured runoff from the northern rooftop (3800 m²). Water from the aboveground cistern (drainage area 1700m²) was used for indoor toilet flushing, while that in the underground cistern (drainage area 2100m²) was used for irrigation of the preserved wooded area. A drip irrigation system was connected to the underground cistern and scheduled to cycle daily, and contained a 10-mm diameter drawdown orifice that emptied the cistern within five days after a storm event. A third cistern (60,600 liters), also located underground, captured rooftop runoff from the southern building (1600 m²) for use in the irrigation (drip and spray) of site landscaped areas. Parking lot runoff and rooftop runoff exceeding the capacity of the cisterns drained to an underground detention and infiltration system beneath the parking lot (Figure 2.1).



Figure 2.2: LID SCMs, clockwise from top left: (a) grassed swales, (b) pretreatment bioretention cell, (c) underground detention basin, and (d) underground infiltration trenches

Stormwater from 0.25 ha of the parking lot drained into approximately 140 m of grassed bioswales (Figure 2.2a, Figure 2.3), and the 60-m² bioretention cell captured stormwater from 0.08 ha of the parking lot (Figure 2.2b, Figure 2.3). Media in the bioretention cell and grassed bioswale was comprised of 85-88% loamy sand, 8-12% silt and clay (fines), and 3-5% organic material, with a P-index of 10-30, following local specifications (NCDENR, 2009). Overflow from the two pretreatment SCMs entered the 1325-m³ concrete detention chamber (Figure 2.2c). Stormwater from the 0.29 ha wooded area drained to a depression in the wooded area for infiltration.

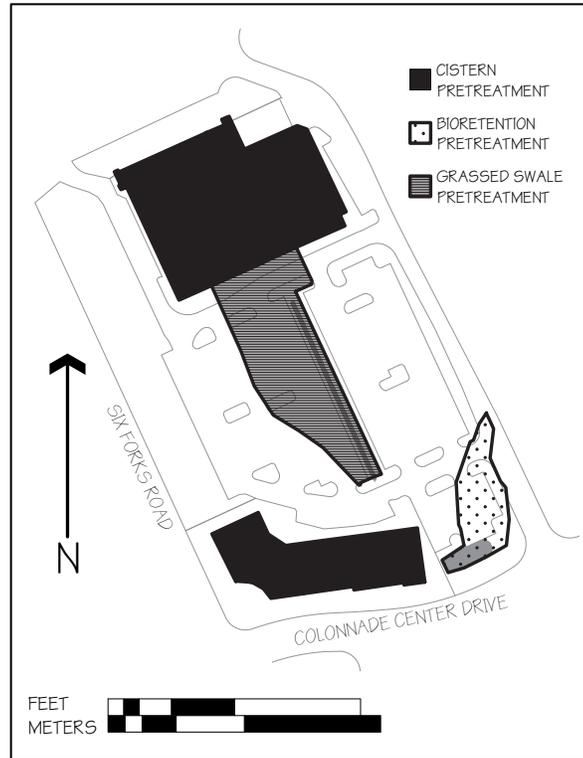


Figure 2.3: Drainage areas of cisterns, a bioretention cell, and a grassed swale

The remainder of the stormwater (1.37 ha drainage area) was conveyed directly into the detention chamber through drop inlets and curb inlets. Immediately prior to entering the detention chamber, all stormwater was routed to concrete StormTrap™ pretreatment units, including an underflow weir and baffles for oil/water separation and sediment and debris removal. The detention basin included a 38-mm diameter drawdown orifice, designed to dewater the detention basin in five days or less. Overflow from the detention chamber drained into an underground infiltration gallery, a 760 m gravel filled trench system (435 m³ of storage) overlying undisturbed soil (Figure 2.2d). The infiltration gallery was designed to have an infiltration rate of 3 mm/hr, infiltrating the 25-mm design storm in 4 days. Overflow

from the infiltration gallery exited the site through a 380-mm diameter concrete pipe into the municipal storm drainage system.

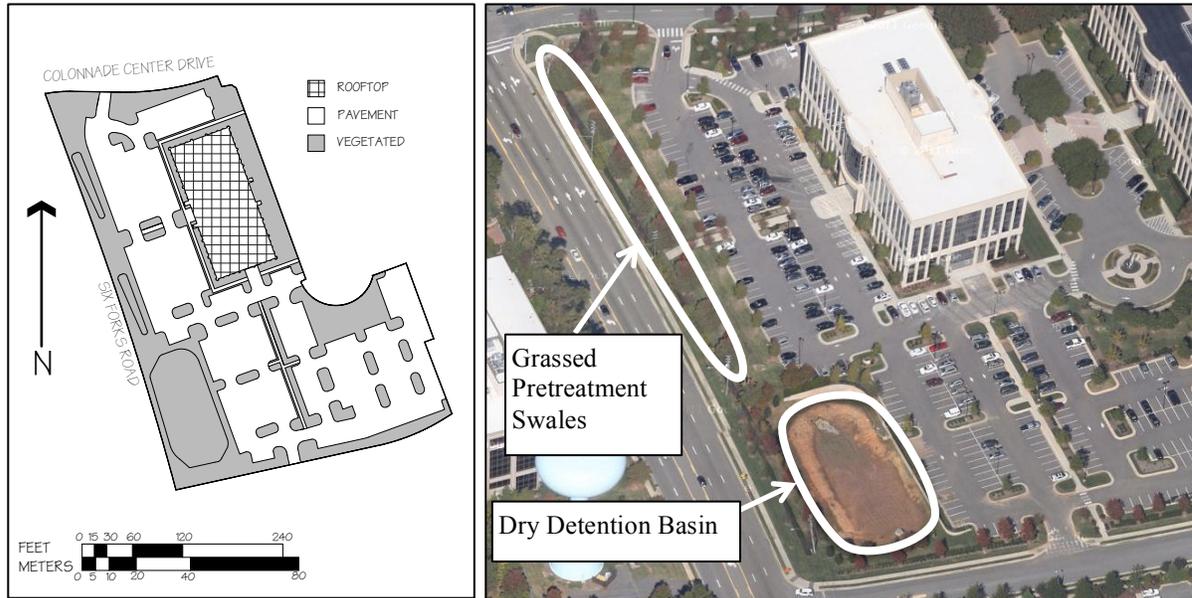


Figure 2.4: Conventional site diagrams, from left: (a) watershed land use and characteristics, and (b) major SCMs

Site characteristics and land use for the conventional site, a 2.8 ha commercial watershed, are found in Table 2.1 and Figure 2.4a. SCMs treating stormwater at this site included three grassed pretreatment swales (33 m and 29 m linear swales, and a 90-m perimeter swale surrounding the dry detention basin) and a 0.14 ha dry detention basin (Figure 2.4b).

Stormwater was routed to one of the three swales for pretreatment before entering the dry detention pond through concrete culverts (Figures 2.5a, 2.5b). The dry detention basin detained and slowly released this stormwater runoff through a concrete outlet structure into the existing storm drainage network (Figure 2.5a). The dry detention pond was designed to

(1) control runoff volumes from the 1-year, 24-hour storm (72.4 mm in Raleigh, NC; NOAA, 2006), releasing this runoff volume over a minimum duration of 48 hours and (2) control peak flows from the 2-year and 10-year storm events (142 mm/hour and 181 mm/hr, respectively; NOAA, 2006).



Figure 2.5: Conventional development design features, from left: (a) dry detention basin with grassed perimeter pretreatment swales and concrete outlet structure, and (b) pretreatment swales

2.3.2 Site Cost

The site cost breakdown of the LID site is found in Table 2.2. As a result of its large-scale underground system, the LID site was more expensive than typical SCMs. However, the high land cost in the area provides reasoning for the high SCM cost. As a result of placing SCMs underground, the increased available land use aboveground allows for more revenue for developers from larger retail lease space. Therefore, it is important to assess the costs and benefits of installing underground SCMs.

Table 2.2: LID site cost breakdown

Pretreatment		\$59,700.00
Pre-treatment Units	\$40,700.00	
Bioswales	\$8,400.00	
Bioretention Area	\$10,600.00	
Underground System		\$541,900.00
Underground Detention & Settling Chamber	\$338,950.00	
Infiltration System	\$132,200.00	
Pipe Network	\$70,750.00	
Cisterns & Irrigation		\$124,500.00
Cisterns	\$111,250.00	
Spray Irrigation & Infiltration System	\$13,250.00	
Total		\$726,100.00

2.3.3 *Monitoring*

This study compared two watersheds after the implementation of conventional and LID SCMs. Aspects of two monitoring designs were used in this study: upstream/downstream and paired watershed (Clausen and Spooner, 1993). Measured discharge at the outflow (downstream) station was compared to precipitation-based estimated inflow (upstream) values for peak flow and runoff volumes at each site using the upstream/downstream monitoring design approach. However, since upstream monitoring stations were not in place at each site throughout the entirety of the monitoring period, the monitoring scheme was not truly a paired watershed design.



Figure 2.6: Monitoring equipment, from left: (a) ISCO 674 tipping bucket rain gage, (b) v-notch weir at outlet of LID site, and (c) compound v-notch weir at outlet of dry detention basin

Monitoring equipment used in this study is found on Table 2.3. An ISCO 674 tipping-bucket rain gage (Figure 2.6a) located at the conventional development site recorded precipitation depth and intensity of storm events. To check the accuracy of the automatic rain gage, a manual rain gage measured precipitation depth and was used to calibrate the tipping-bucket rain gauge. On average, the automatic rain gage data was adjusted by a correction coefficient of 1.15; the adjustment amount did not depend on rainfall intensity. Precipitation values enabled calculation of inflow volumes to estimate inflow pollutant loading.

Table 2.3: Monitoring equipment used in study

	LID	Conventional Development
Precipitation	-	ISCO 674 Automatic Rain Gage ¹
Water level	ISCO 730 Bubbler Module	ISCO 730 Bubbler Module
Stage-Discharge Relationship	30°, 150-mm V-notch weir	30°, 100-mm V-notch weir, 410-mm rectangular end restrictions
Data Storage	ISCO 6712 Sampler	ISCO 6712 Sampler

¹Rain gage was 0.25 km from the LID watershed.

Monitoring equipment was installed at the outlet to the municipal storm drainage network at each site: the 610-mm diameter outlet pipe at the conventional development was fitted with a steel compound weir (Figure 2.5b), and the 380-mm diameter outlet pipe at the LID was fitted with a steel v-notch weir (Figure 2.5c). Water levels at each site outlet were measured at 2-minute intervals throughout each storm event by ISCO 730 Bubbler Modules installed at the invert of the pipe at the LID site and inside the concrete outlet structure at the conventional site. ISCO 6712 Samplers fitted with the bubbler modules at each site recorded and stored water level data throughout each storm event. Recorded water levels were converted to storm discharge through stage-discharge weir equations. Equations and calculations are found in Appendix A.

Power failure due to insufficient battery life, holes cut in the bubbler tubing, and air in the bubbler tubing caused occasional data collection malfunction. This was not an issue at the conventional development, with the exception of one incident where the bubbler tubing was accidentally cut due to plant maintenance in the dry pond. A few occurrences of insufficient battery life caused some data to be omitted, but overall data collection malfunction was low at both sites (4 of 24 storm events). Incorrect data were not included in analyses. To ensure that data collected were accurate, a manual level was measured after each storm, and recorded 2-minute increment data were adjusted accordingly. With the exception of three inaccurate water level readings incidences at the LID site (average error 0.047 m) and three such at the conventional site (average error 0.036 m), water level readings were accurate throughout the monitoring period.

Table 2.4: Curve number and initial abstraction values for land uses and antecedent moisture conditions (AMC; USDA, 1972)

	AMC I		AMC II		AMC III	
	CN	I _a (mm)	CN	I _a (mm)	CN	I _a (mm)
Grassed	63	29.8	80	12.7	91	5.0
Wooded	40	76.2	60	33.9	78	14.3
Rooftop (concrete)	94	3.2	98	1.0	99	0.5
Parking (asphalt)	94	3.2	98	1.0	99	0.5

Theoretical inflow runoff volumes were calculated by assigning curve numbers (CN) to each land type based on land use, hydrologic soil group, and land slope (USDA, 1972). Storage (*S*, mm) and initial abstraction (*I_a*, mm) were calculated from the curve numbers (Equations 2.1, 2.2) using the discrete method to separate the DCIA from pervious areas, and are shown in Table 2.3 (USDA, 1972). Subtracting *I_a* from each precipitation depth produced an estimated runoff depth for each monitored storm (Brown and Hunt, 2011; Pandit and Heck, 2009). Runoff depths were multiplied by respective land areas to estimate composite inflow volumes for each storm.

$$S = \frac{1000}{CN} - 10 \quad (2.1)$$

$$I_a = 0.2S \quad (2.2)$$

$$Q_P = \frac{CiA}{360} \quad (2.3)$$

Estimated peak inflow rates associated with each storm were determined using the Rational Method (Equation 2.3; USDA, 2004). To determine peak inflow, runoff coefficients (*C*) were composited for each watershed from values of 0.95 for rooftops and parking lots, 0.3

for grassed areas, and 0.15 for wooded natural conditions (USDA, 2004). Using measured rainfall intensities (i , mm/hr) and known areas (A , hectares) for each land use, estimated peak flows (Q_P , m^3/s) were calculated for each storm. Runoff coefficients were calculated from a ratio of observed runoff depths to rainfall depths. Runoff depths for each storm were calculated by normalizing the runoff volume at each site by their respective watershed area. To relate runoff coefficients observed to those expected, typical runoff coefficients reported by ASCE (1972) were used for comparison.

2.3.4 Data Analysis

Data analysis was performed with SAS® 9.3 software (SAS Institute, Inc. 2012) to statistically compare hydrologic differences between the two watersheds. Differences of paired data sets were first tested for normality with three goodness-of-fit tests: Kolmogorov-Smirnov, Cramer-von Mises, and Anderson-Darling. Resulting normally distributed data were tested for statistical significance using the student's t test; non-normal data were log-transformed and reanalyzed for a normal distribution. Resulting log-transformed data fitting a normal distribution were analyzed using the student's t test; non-normal data were analyzed for statistical significance using nonparametric tests (Wilcoxon signed rank or sign test). Due to the assumption of symmetry in the Wilcoxon signed rank test, outliers of nonparametric data were analyzed for symmetry. If an approximately equal number of outliers existed on each tail of the data, symmetry was assumed, and the Wilcoxon signed rank test was utilized. However, excessive outliers on only one tail suggest asymmetry, and so the less statistically powerful sign test was used to determine differences. Analysis of

Covariance (ANCOVA) determined the effect of seasonality on results. Data were analyzed for significance at the $\alpha=0.05$ level. SAS® code is included in Appendix D.

2.4 Results

2.4.1 Precipitation Characteristics

Site hydrology and precipitation were monitored from January 11 – December 18, 2012.

During 2012, annual precipitation of 1285 mm was 9% above the climate normal of 1179 mm. Annual rainfall was not compiled at the site; however, these annual precipitation values were observed in Raleigh, NC (Station #317079: Raleigh State Univ, 35.794°N, 78.699°W), approximately 15 km from the study site.

The 47 storms monitored produced approximately 777 mm of rainfall, much less than that of the annual rainfall observed in Raleigh, NC. Reasoning behind a lack of annual recorded rainfall onsite was attributed to two factors: (1) manual rain gage readings were not recorded during non-sampled events for the first six months of the study period, and (2) storm events that did not produce runoff (generally precipitation depths less than 5 mm) were not included in the monitored 777 mm. Therefore, it is important to emphasize that this 777 mm of recorded data was not the annual precipitation depth onsite.

The 47 storm events monitored ranged in depth from 4.6 – 79.5 mm, and in 5-minute intensity from 5.8 – 216.2 mm/hour. Average rainfall intensity during the summer storms

(115.6 mm/hour) was significantly higher than in winter (17.3 mm/hour; $p=0.0077$), spring (46.5 mm/hour; $p=0.0002$), and fall (26.7 mm/hour; $p=0.0001$).

2.4.2 Runoff Volume

Both the conventional development and the LID sites were designed for peak flow mitigation; the LID site was also designed for runoff volume mitigation. Runoff volume reduction was observed at both sites. The median values of runoff volume, reduction, and coefficients were computed for each site, and are shown in Table 2.5.

Table 2.5: Median hydrologic values of the conventional development and the LID.

	Conventional Development ($n=47$)	LID ($n=47$)	Ratio: Conventional / LID
Inflow volume (m^3) ^a	175.12	219.30	0.80
Inflow depth (mm) ^{a b}	10.49	10.39	1.01
Outflow volume (m^3)	94.56	3.40	27.83
Outflow depth (mm) ^b	5.66	0.16	35.16
Runoff reduction (%)	51.4%	98.3%	0.52
Runoff coefficient	0.49	0.02	34.85

^aEstimated from Curve Number method

^bNormalized by DCIA

Inflow calculations are found in Appendix B. Median inflow runoff volumes for the conventional development and LID were $175 m^3$ and $219 m^3$, respectively, while cumulatively, inflow volumes of $12930 m^3$ and $14252 m^3$ were calculated for the conventional development and the LID, respectively, throughout the monitoring period. Though the LID received a 1.1 times higher inflow volume throughout the monitoring

period, the median outflow volume at the conventional site is 35 times higher than at the LID, showing the higher potential of the LID site to mitigate runoff volumes through detention and infiltration.

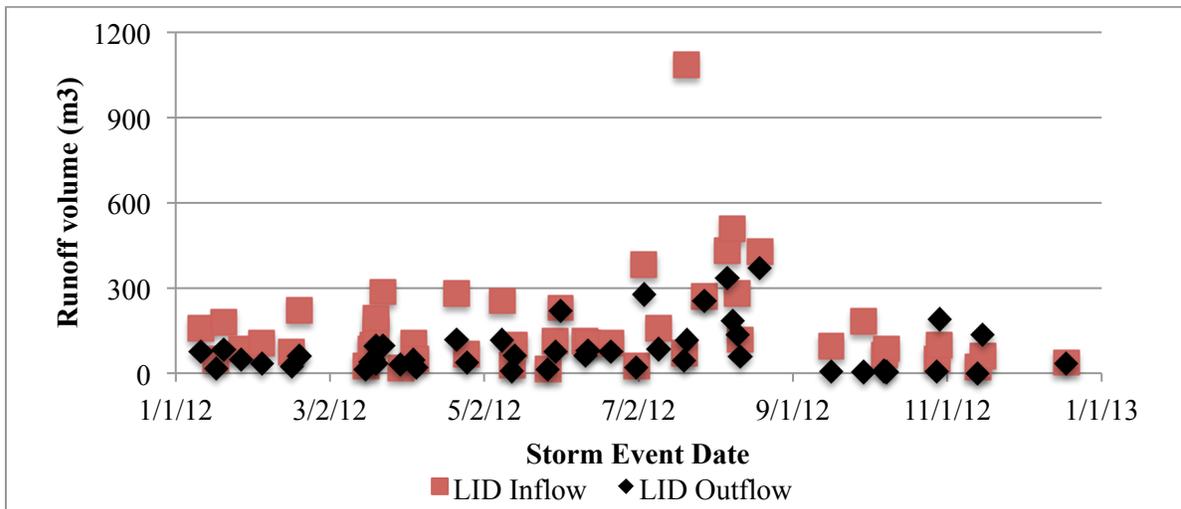


Figure 2.7: LID estimated inflow and measured outflow

Figure 2.7 illustrates the estimated inflow and observed outflow volumes at the LID site. A median runoff reduction of 98.3% was observed at the LID site, meaning that all but 1.7% of the stormwater was detained onsite and infiltrated into the underlying soils or harvested. This high runoff volume reduction was due to the detention and infiltration capacity of the underground stormwater detention chamber and the infiltration gallery, as well as the high infiltration capacity of the underlying soils. Prior to construction, infiltration rates of 6.4 mm/hour were measured at the invert of the infiltration gallery (Patrick Smith, personal communication, January 4, 2013), falling at the upper end of the infiltration range of 3.8 – 7.6 mm/hour for hydrologic soil group B soils (Musgrave, 1955). The LID was designed to

capture all stormwater from the 25-mm event for infiltration; however, minimal outflow is observed for 46 of the 47 storms (38 of which were less than 25 mm). This unexpected outflow volume may be attributed to a suspected cross-connection in the underground system due to construction errors. Although the average storm depth and intensity were smaller in autumn, LID outflow was 6-7 times higher in autumn than the spring ($p=0.0029$) and winter ($p=0.0048$), which was unexplained in the data.

Table 2.6: Statistical analysis for paired differences of LID runoff volume

Paired Difference:	LID inflow - outflow
Distribution:	Non-normal
Test used:	Sign test
Test statistic:	M = 23.5
p	<0.0001

LID site outflow volume was significantly lower than inflow ($p < 0.0001$), ranging from an 89% reduction in runoff for a 79.5-mm storm event to a 100% runoff reduction for a 16.5-mm storm event. Cumulatively, the LID site reduced runoff during the monitoring period by 97%, infiltrating approximately 13,800 m³ into underlying soils. Table 2.6 shows the statistical analysis used.

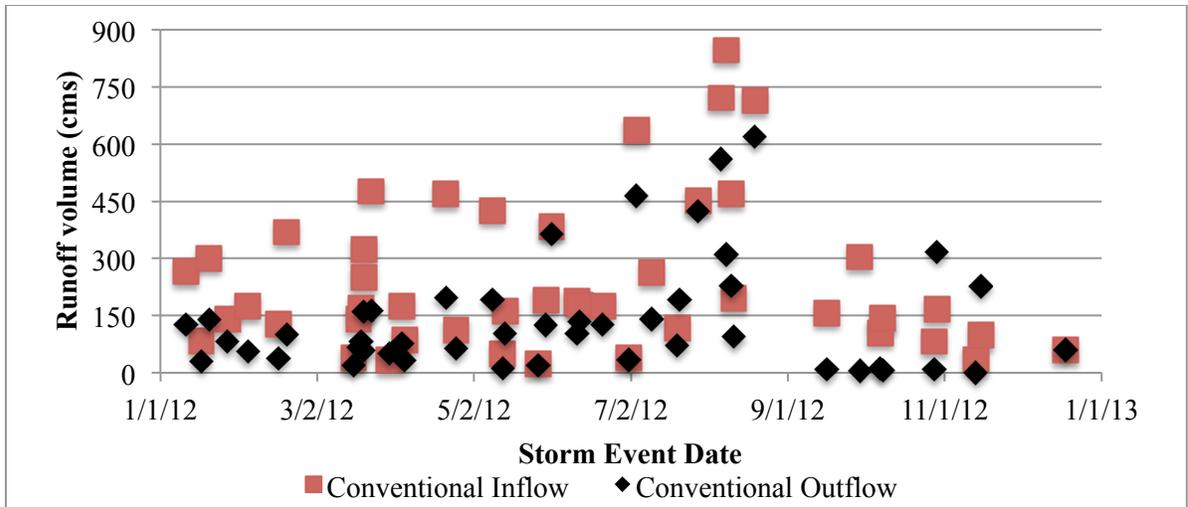


Figure 2.8: Conventional development estimated inflow and measured outflow

Figure 2.8 illustrates the inflow and outflow runoff volumes calculated and measured, respectively, at the conventional development. The conventional development had a median runoff reduction of 51.4%, approximately one-half of the reduction measured at the LID site. Because dry detention basins are not designed to infiltrate stormwater, but to mitigate peak flow, this runoff reduction value is higher than expected for the conventional site (Roesner et al., 2001). This may be due to error in approximating the inflow runoff volumes for each storm, a higher-than-expected infiltration capacity of the underlying soil, or a leak in the outlet structure. However, a trio of double ring infiltrometer tests in the dry detention basin yielded an average infiltration rate of approximately 0.17 mm/h, which is much lower than the expected value for hydrologic soil group B soils (3.8 – 7.6 mm/hour; Musgrave, 1955). Likely due to smaller storm depths and intensities during the fall season, conventional development outflow in the spring, summer, and winter seasons are 4-5 times higher than that in the fall season ($p=0.007$, $p=0.0256$, and $p=0.0390$, respectively).

Table 2.7: Statistical analysis for paired differences of conventional development runoff volume

Paired Difference:	Conv. inflow - outflow
Distribution:	Non-normal
Test used:	Sign test
Test statistic:	M = 20.5
<i>p</i>	<0.0001

The conventional development had significantly lower outflow than inflow ($p < 0.0001$), ranging from no net reduction for 5-mm, 13-mm, and 9-mm storms to a 100% runoff reduction for a 5-mm storm event. Three events had essentially no runoff reduction; in fact, measured outflow was on average 1.9 times higher than the calculated inflow for these three storms, which may have been due to existing stormwater in the dry detention basin prior to the storm event contributing to the outflow volume. For two of these three storms, the antecedent dry period was less than two days. Cumulatively, the conventional development detained and infiltrated or evaporated 49.7% of stormwater, allowing for evaporation and infiltration of 6,400 m³ of water throughout the monitoring period. Table 2.7 summarizes site statistics.

Table 2.8: Statistical analysis for paired differences of conventional – LID inflow and outflow

Paired Difference:	Inflow: Conventional - LID	Outflow: Conventional - LID
Distribution:	Non-normal	Non-normal
Test used:	Wilcoxon signed rank	Sign test
Test statistic:	S = -400	M = 20.5
<i>p</i>	<0.0001	<0.0001

Due to the smaller fraction of impervious area onsite, estimated median inflow values at the conventional development were approximately 80% of those at the LID site (Table 2.8). However, the conventional development outflow was 28 times higher than that of the LID ($p < 0.0001$). After normalizing by DCIA, the conventional site discharged 35 times more runoff volume than the LID. These results were more extreme than those found by both Bedan and Clausen (2009) and Line et al. (2012). Bedan and Clausen (2009) calculated the total conventional development runoff volume to be 4.3 times the total runoff volume exiting a residential LID, and the commercial LID studied by Line et al. (2012) produced 1.3 times more runoff volume than the conventional development when normalized by rainfall and drainage area. The considerably greater runoff reduction of the LID compared to similar studies may be attributed to the high volume capacity LID design. Between the cisterns, detention chamber, and infiltration gallery at the LID site, approximately 1950 m³ of storage was incorporated onsite, allowing the capture of a 77-mm storm event over the entirety of the catchment area, rather than the design storm of 25 mm; only one storm event monitored was larger than the 77-mm storm. Rather than detain this runoff, underlying high infiltration type B soils infiltrated 98% of the stormwater, allowing for the low outflow volumes observed.

In this study, the conventional development was not designed for runoff volume reduction, but both sites mitigated runoff volumes significantly ($p < 0.0001$). Figure 2.10 displays the cumulative rainfall depth monitored throughout the monitoring period, as well as the runoff depth discharged at the inflow and outflow of each site. Runoff depth was computed by dividing cumulative runoff volumes by each site's respective DCIA. Of the 777 mm of rainfall recorded at the site, the conventional development and the LID had cumulative inflow runoff depths of 468 and 516 mm, respectively. After treatment from SCMs at both sites, the conventional development and the LID exported 236 and 15 mm of cumulative runoff depth, respectively, and had cumulative runoff depth reductions of 49.7% and 97.0%, respectively.

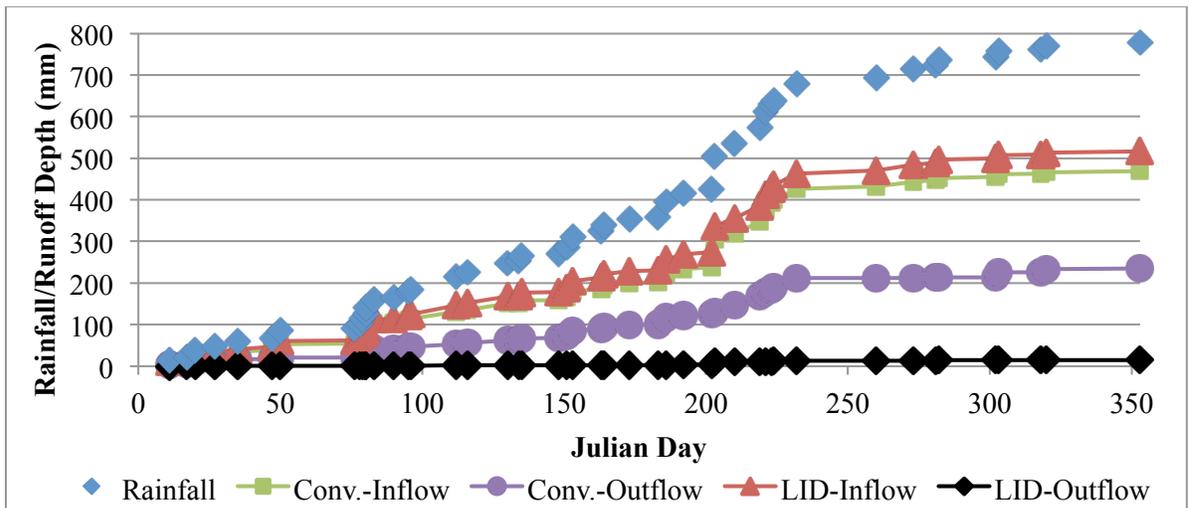


Figure 2.10: Cumulative rainfall and runoff depths for the conventional development and the LID for inflow and outflow

2.4.3 Runoff Coefficients

Typically, as percentage imperviousness of a site increases, the runoff coefficients increase (Dietz and Clausen, 2007; Schueler, 1994). In a study of a 1.7 ha conventional residential development and a residential LID both pre- and post-construction, an exponential relationship existed between the percentage imperviousness of the conventional development and its runoff coefficient; the runoff coefficient at the LID site did not change as percent imperviousness increased (Dietz and Clausen, 2007).

Runoff coefficients were computed at each site for each storm events monitored, and the median values are found in Table 2.5. Compared to the median runoff coefficient observed at the LID site (0.02), Line et al. (2012) observed a 14 times higher cumulative runoff coefficient (0.58) at a commercial LID, while Hood et al. (2007) observed a 3.4 times higher mean runoff coefficient (0.067) at a residential LID. Rushton (2001) studied various low impact parking lot developments and observed runoff coefficients between 0.10 - 0.35 for parking lots incorporating a mixture of swales and various pavement types, including pervious pavement. The LID reduced runoff volumes more substantially than previous studies, and less than runoff coefficient ranges for all land uses (ASCE, 1992). The low runoff coefficient at the LID site was likely due to the large detention volume onsite and the high infiltration-capacity type B soils at the LID site. Line et al. (2012) cites poorly constructed bioretention cells and stormwater wetlands at the commercial LID as the reasoning behind their high runoff coefficient. Due to undersized SCMs, storm capture was less than the design storm of 25 mm.

At the conventional development, the runoff coefficient (0.49) is slightly higher than that found at a wet detention basin (0.44) studied by Line et al. (2012), and was 2.1 times the conventional residential development runoff coefficient (0.24) observed by Hood et al. (2007). Rushton (2001) observed runoff coefficients of 0.51 – 0.58 and 0.16 – 0.35 for asphalt parking lots without and with a swale, respectively. Asphalt pavement is assigned runoff coefficients of 0.70 – 0.95, so the addition of SCMs had an important effect, leaving the conventional commercial development with a runoff coefficient reflective of one for residential single family homes without SCMs (ASCE, 1992). During the fall, the runoff coefficient at the LID site was 6-7 times higher than during the spring ($p=0.0048$) and winter ($p=0.0061$) seasons, due to the significantly higher LID outflow during the fall season. However, there was no effect of seasonality on the conventional development runoff coefficient.

2.4.4 Peak Discharge

A major goal of both sites was to mitigate peak flows of the 2-year and 10-year storm events (142 mm/hour and 181 mm/hr, respectively; NOAA, 2006). Due to automatic rain gage malfunction, 40, rather than 47, storms were monitored for peak discharge. Median calculated peak inflow, observed peak discharge, and peak reduction values are found in Table 2.9. Peak inflow values for three storms with 5-minute intensities larger than that of the 10-year storm are discussed in this section as well.

Table 2.9: Peak flow values of the conventional development and the LID.

	Conventional Development (<i>n</i> =40)	LID (<i>n</i> =40)	Ratio: Conventional / LID
Peak inflow (L/s)	231	252	0.92
Peak inflow (mm/hr) ^a	49.8	43.0	1.16
Peak outflow (L/s)	2.85	0.33	8.66
Peak outflow (mm/hr) ^a	0.61	0.06	10.94
Peak reduction (%)	98.7%	99.8%	0.99
Peak inflow > 10yr (L/s) ^b	1087	1186	0.92
Peak outflow > 10yr (L/s) ^b	75.6	12.4	6.10

^a Normalized by DCIA

^b Peak inflow rates associated with three storms more intense than the 10-year, 5-minute storm

Estimated peak inflows were calculated from the Rational Method (USDA, 2004); calculations are shown in Appendix B. Median peak inflow rates for the conventional development and the LID were 231 L/s and 252 L/s, respectively, while three storms larger than the 10-year storm were calculated to have an average peak inflow rate of 1087 L/s and 1186 L/s for the conventional development and the LID, respectively (Table 2.9). Though the conventional development had a 9 times higher median peak flow ($p < 0.0001$) than the LID, both sites reduced peak flows by more than 98%, because large-scale detention SCMs (dry detention and underground detention basin) were principal elements of each site's design.

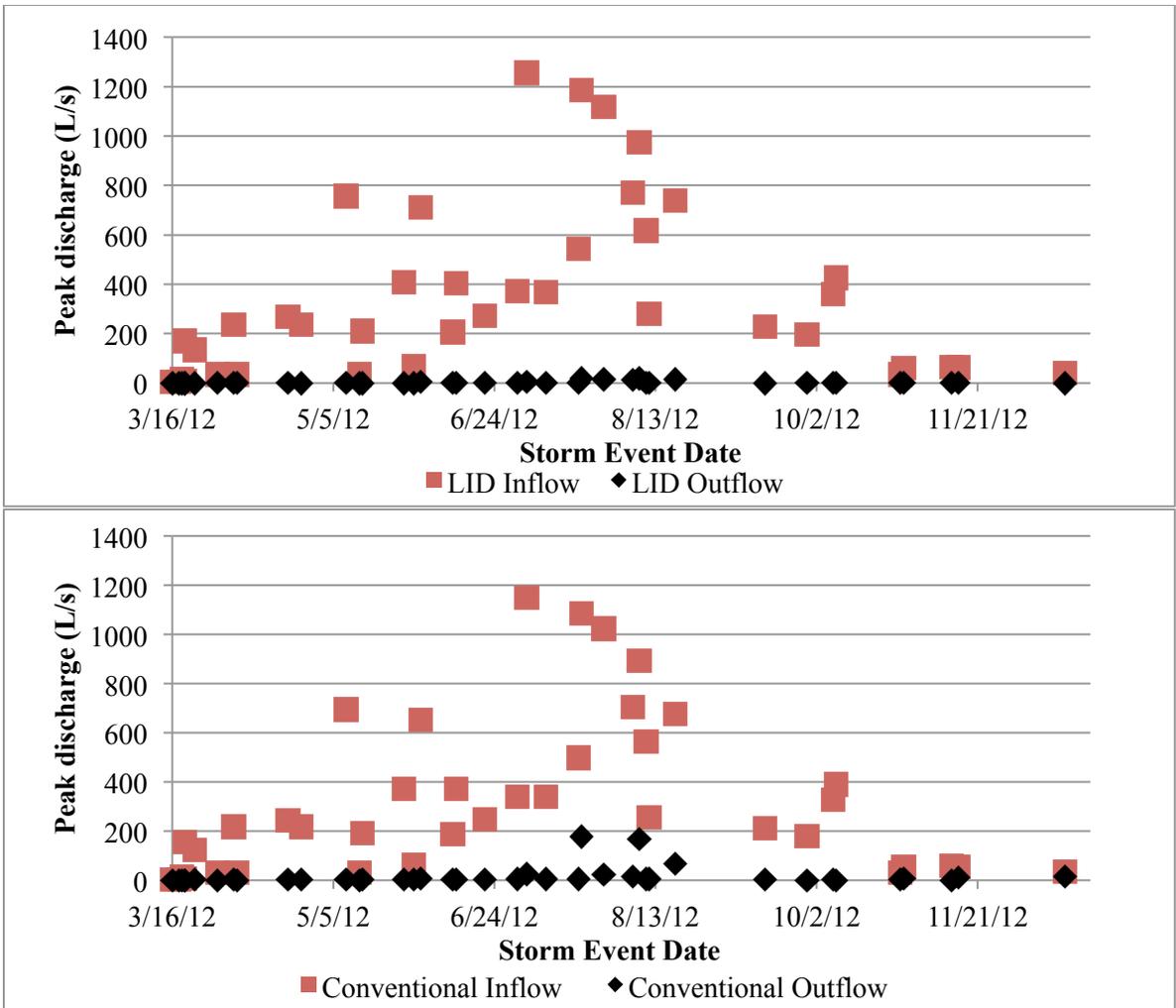


Figure 2.11: From top: calculated inflow and measured outflow from the (a) LID, and (b) conventional development.

Figure 2.11 illustrates the estimated peak inflow and measured outflow discharges at the LID and conventional development sites. Median peak flow reductions of 98.7% and 99.8% were observed at the conventional development and the LID, respectively. Tabulated raw data are found in Appendix A. LID peak discharge was significantly lower than inflow for all storms sampled ($p < 0.0001$), ranging from 98% - 100% reduction. At the conventional development, peak discharge was significantly lower than inflow as well ($p < 0.0001$), ranging

from a 56% reduction for a 6.9-mm storm to a 100% reduction for an 11-mm storm. Table 2.10 shows the statistical analysis used.

Table 2.10: Statistical analysis for paired differences of inflow – outflow for the conventional development and the LID

Paired Difference:	Conventional: Inflow-Outflow	LID: Inflow - Outflow
Distribution:	Non-normal	Non-normal
Test used:	Sign test	Sign test
Test statistic:	M = 20	M = 20
<i>p</i>	<0.0001	<0.0001

Median peak discharge rates for the conventional development and LID are 2.85 L/s and 0.33 L/s, respectively (Figure 2.12). Though the LID had a significantly higher estimated peak inflow due to its larger DCIA, the conventional site had larger peak discharge values for 39 of 40 storms. Due to its small size, the one remaining storm (5 mm) produced no discharge at the conventional site.

Table 2.11: Statistical analysis for paired differences of conventional development and LID peak discharges

Paired Difference:	Outflow: Conventional - LID
Distribution:	Non-normal
Test used:	Sign test
Test statistic:	M = 19.5
<i>p</i>	<0.0001

Measured hydrographs from a 9.7-mm storm event occurring on October 7, 2012 are illustrated in Figure 2.7. The peak discharge rates from this storm event were 3.28 L/s and 0.88 L/s for the conventional site and the LID, respectively. As shown, the peak discharge from the LID site coincided with the peak rainfall intensity (within twenty minutes), as compared to the one-hour lapse between peak rainfall intensity and conventional peak discharge. A reason for this difference is the suspected cross-connection inside the LID underground system, which allowed stormwater to bypass the detention chambers and infiltration gallery, thus exiting the site untreated.

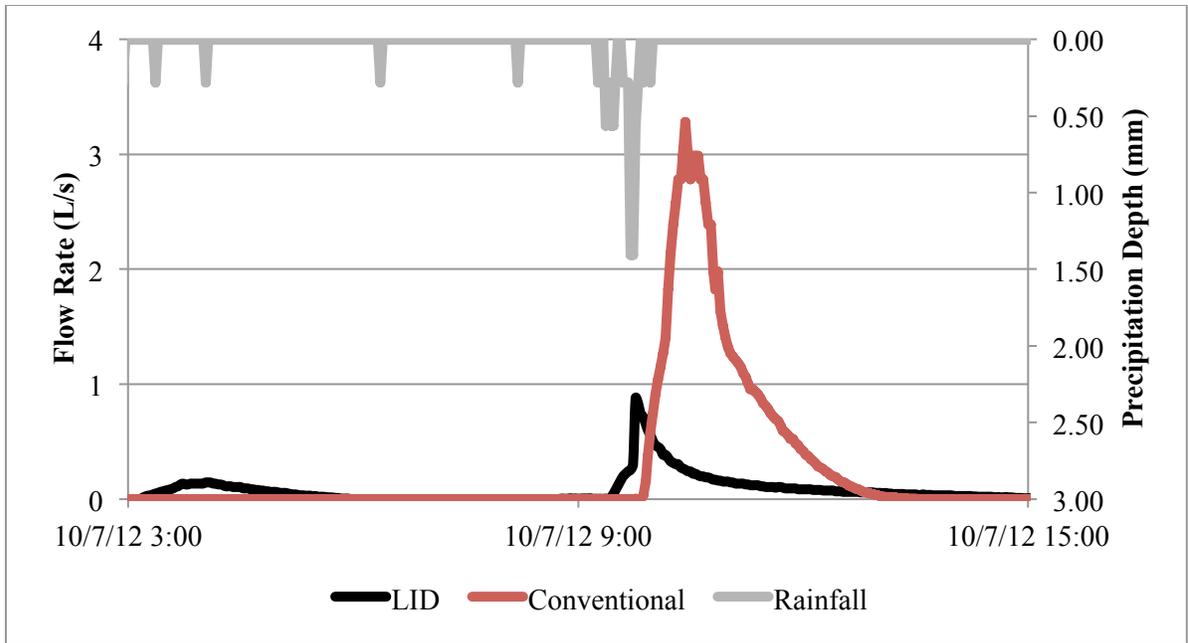


Figure 2.13: Hydrograph of rainfall, LID outflow, and conventional development outflow from a 9.7-mm storm event October 7, 2012

At both sites, mitigation of peak discharges of the 2- and 10-year, 5-minute storm events (142 mm/hour and 181 mm/hour, respectively) was required to that of predevelopment conditions. To calculate predevelopment peak discharges for both sites, predevelopment land uses were analyzed from historical maps, and assigned runoff coefficients (Table 2.12). Composite C values of 0.19 and 0.21 were calculated for predevelopment conditions of the LID and the conventional development, respectively; a C value of 0.2 was used to calculate predevelopment peak discharges at both sites.

Table 2.12: Predevelopment land use characteristics

Land use	Conventional Development area (ha)	LID area (ha)	C ¹
Watershed area	2.76	2.53	-
Grassed	1.43	0.66	0.30
Wooded	1.03	1.88	0.15

¹ USDA, 2004

Three monitored storms had greater rainfall intensities than both the 2- and 10-year, 5-minute storm (7/4/2012, 7/21/2012, and 7/28/2012), and peak discharges from all three storms, as well as all 40 storms sampled, are less than calculated predevelopment peak discharges at both sites, meeting the goal of predevelopment peak discharge mitigation (Figure 2.14). A composite runoff coefficient of 0.75, determined from land use characteristics (USDA, 2004) and averaged between the two developments, was used to calculate peak discharges if no SCMs were installed (untreated watershed). As illustrated in Figure 2.14, the peak discharge for the three most intense storms of the untreated development was 380% of predevelopment conditions, which is consistent with previous studies on untreated development; Line and White (2007) found development to cause an 420% increase in peak discharge.

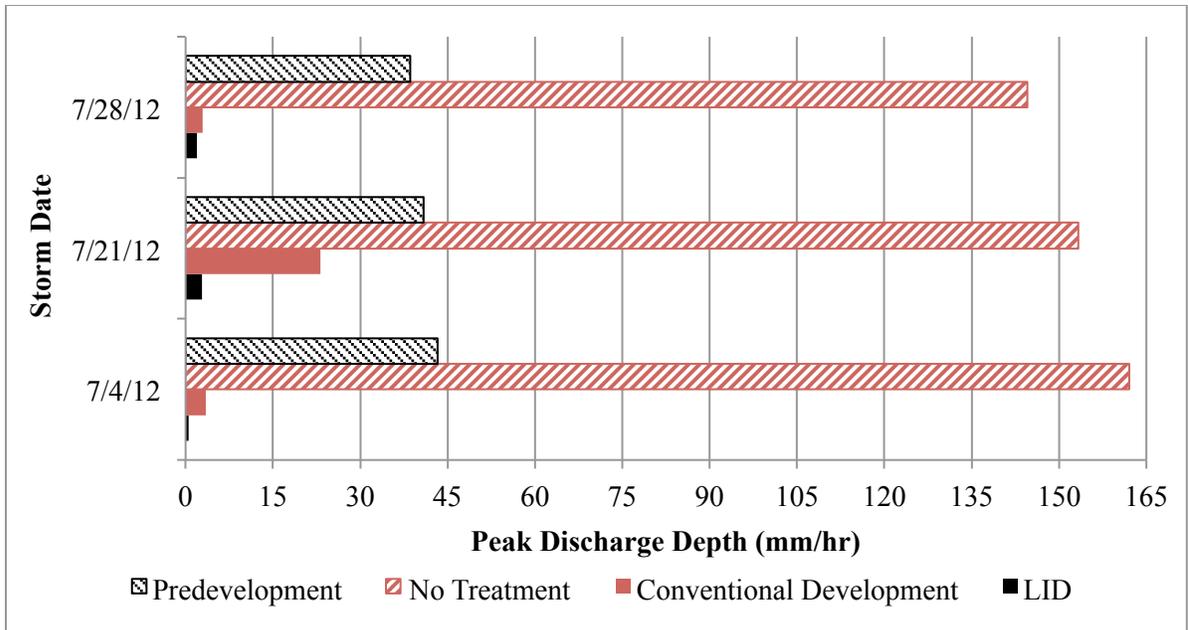


Figure 2.14: Peak discharges for predevelopment, postdevelopment with no stormwater treatment, postdevelopment with conventional SCM treatment, and postdevelopment with LID treatment for three monitored storm events larger than the 10-year storm in Raleigh, NC

The conventional development and the LID peak discharges were 24% and 4%, respectively, of predevelopment peak discharge for these three events. Though both the conventional development and the LID were required to *meet* predevelopment hydrology for the 2- and 10-year storms, both sites dramatically reduced peak discharges *below* that of predevelopment, likely due to higher than expected detention capabilities of SCMs on site. The unexpected peak discharge mitigation of the conventional development may be due to its 76-mm outlet orifice limiting large discharge rates, or the higher than expected infiltration, as shown by a 51% runoff reduction. This was not shown in the results of the double ring infiltrometer tests, but other parts of the dry detention basin may have had better infiltration.

The conventional residential development and the residential LID studied by Bedan and Clausen (2009) discharged 160% and 11%, respectively, of predevelopment peak discharge. Hood et al. (2007) found conventional development and LID peak discharges of 224% and 20%, respectively, of that of predevelopment conditions, higher than values observed in this study. When compared to predevelopment conditions, the LID site peak flow mitigation likely outperformed similar studies due to “overdesigning” the LID site, allowing for a large detention volume capable of capturing 77 mm of runoff, rather than the 25.4 mm design storm as in Bedan and Clausen (2009) and Hood et al. (2007).

2.5 Conclusions

Runoff and peak discharge reductions were observed at both the LID and conventional development throughout the monitoring period. Due to the detention and infiltration SCMs at both developments, peak flow mitigation of over 98% was observed, though an 11 times higher peak flow rate was observed at the conventional development than the LID. In addition, the LID site retained 98% of stormwater volume onsite for indoor use, irrigation, and infiltration, outperforming similar LID studies (Bedan and Clausen, 2009; Hood et al., 2007; Line et al., 2012; Rushton, 2001). Runoff coefficients of 0.02 and 0.49 were observed for the LID and conventional developments, with the LID reducing runoff volumes more substantially than any similar LID studies (Hood et al., 2007; Line et al., 2012; Rushton, 2001). Due to the infiltration and detention of the LID SCMs, a large fraction of runoff was transformed from surface runoff into groundwater; this reduces erosive impacts caused by

surface runoff in receiving waterways (Moscrip and Montgomery, 1997) while providing benefits of a recharged groundwater flow.

The high performance of the LID in mitigation of runoff volumes and peak discharges was likely due to an “overdesign” of the LID, providing enough detention capability to capture the 77-mm storm event, rather than a typical 25.4-mm design event. Underlying hydrologic soil group B soils (Appling and Cecil) were paramount in the high runoff and peak flow reduction observed at both the LID and the conventional development, mitigating peak flow rates to a fraction of that of predevelopment conditions, and infiltrating 97% and 50% of the 777 mm of rainfall recorded, respectively. Due to its innovative design combining large-scale detention and infiltration trenches with rainwater harvesting, swales, and bioretention, not only did LID reduce runoff volumes, peak discharges, and runoff coefficients below previous LID studies, but it will provide space-saving solutions for areas where aboveground SCMs, such as bioretention and dry detention, are not feasible due to high land costs and constricted spaces.

2.6 References

- American Society of Civil Engineers (ASCE). (1992). *Design and construction of urban stormwater management systems*. ASCE Manuals and Reports of Engineering Practice No. 77.
- Bean, E.Z., Hunt, W.F. and Bidelspach, D.A. (2007). "Evaluation of four permeable pavement sites in Eastern North Carolina for runoff reduction and water quality impacts." *Journal of Irrigation and Drainage Engineering*, 133(6), 583-592.
- Bedan, E.S. and Clausen, J.C. (2009). "Stormwater runoff quality and quantity from conventional and low impact development watersheds." *J.Am.Water Resour.Assoc.*, 45(4), 998-1008.
- Brown, R.A. and Hunt, W.F. (2011). "Underdrain configuration to enhance bioretention exfiltration to reduce pollutant loads." *J.Environ.Eng.*, 137(11), 1082-1091.
- Clausen, J.C., and Spooner, J.. (1993). "Paired watershed study design." EPA 841-F-93-009. U.S. Environmental Protection Agency, Office of Water, Washington, DC.
- Collins, K.A., Hunt, W.F. and Hathaway, J.M. (2008). "Hydrologic comparison of four types of permeable pavement and standard asphalt in Eastern North Carolina." *J. Hydrol. Eng.*, 13(12), 1146-1157.
- Davis, A.P., Hunt, W.F., Traver, R.G. and Clar, M. (2009). "Bioretention technology: Overview of current practice and future needs." *J. Environ. Eng.*, 135(3), 109-117.
- Dietz, M.E. (2007). "Low impact development practices: A review of current research and recommendations for future directions." *Water Air and Soil Pollution*, 186(1-4), 351-363.
- Dietz, M.E. and Clausen, J.C. (2008). "Stormwater runoff and export changes with development in a conventional and low impact subdivision." *J.Environ.Manage.*, 87(4), 560-566.
- Hammer, T.R. (1972). "Stream channel enlargement due to urbanization." *Water Resour. Res.*, 8(6), 1530 – 1540.
- Hood, M. J., Clausen, J.C. and Warner, G.S. (2007). "Comparison of stormwater lag times for low impact and conventional residential development." *J.Am.Water Resour.Assoc.*, 43(4), 1036-1046.

- Hunt, W.F., Smith, J.T., Jadlocki, S.J., Hathaway, J.M. and Eubanks, P.R. (2008). "Pollutant removal and peak flow mitigation by a bioretention cell in urban Charlotte, NC." *J. Environ. Eng.*, 134(5), 403-408.
- Line, D.E., Brown, R.A., Hunt, W.F. and Lord, W.G. (2012). "Effectiveness of LID for commercial development in North Carolina" *J. Environ. Eng.*, 138(6), 680-688.
- Line, D.E. and White, N.M. (2007). "Effects of development on runoff and pollutant export." *Water Environ. Res.*, 79(2), 185-190.
- Moscip, A.L. and Montgomery, D.R. (1997). "Urbanization, flood frequency, and salmon abundance in Puget lowland streams." *J. Am. Water Resour. Assoc.*, 33(6), 1289-1297.
- Musgrave, G.W. (1955). "How much of the rain enters the soil?" *Water: U.S. Department of Agriculture Yearbook of Agriculture*, Washington, DC. 151-160.
- NCDENR (North Carolina Department of Environmental and Natural Resources). (2009). "Stormwater Best Management Practices Manual."
- NOAA (National Ocean and Atmospheric Administration). (2006). "NOAA Atlas 14. Vol. 2. Ver. 3." *Precipitation data frequency server*, National Weather Service, Silver Spring, MD., website: <http://hdsc.nws.noaa.gov/hdsc/pfds>
- Pandit, A. and Heck, H.H. (2009). "Estimations of soil conservation service curve numbers for concrete and asphalt." *J. Hydrol. Eng.*, 14(4), 335-345.
- Prince George's County. (1999). "Low-impact development design strategies. An integrated design approach."
- Roesner, L., Bledsoe, B. and Brashear, R. (2001). "Are best-management-practice criteria really environmentally friendly?" *Journal of Water Resources Planning and Management*, 127(3), 150-154.
- Rushton, B.T. (2001). "Low-impact parking lot design reduces runoff and pollutant loads." *J. Water Resour. Plann. Manage.* 127(3), 172-179.
- Schueler, T. (1994). "The importance of imperviousness." *Watershed Protection Techniques*, 1(3), 100-111.
- State Climate Office of North Carolina (SCO). (2012). State Climate Office of North Carolina. North Carolina State University. website: <http://www.nc-climate.ncsu.edu>

U.S. Department of Agriculture (USDA), Natural Resources Conservation Service. (2004). Chapter 10: Estimation of direct runoff from storm rainfall. Part 630 – Hydrology: National Engineering Handbook, Washington D.C.

U.S. Census Bureau. (2013). *State & county Quickfacts: Wake County, N.C.* website: <http://quickfacts.census.gov>

U.S. Department of Agriculture (USDA), Soil Conservation Service. (1972). *National engineering handbook, Section 4: Hydrology*, Chapters 4–10, Washington, DC. 15-7–15-11.

USEPA (United States Environmental Protection Agency). (2009). "National water quality inventory: 2004 report to Congress."

Wang, L. and J. Lyons. (2003). "Fish and benthic macroinvertebrate assemblages as indicators of stream degradation in urbanizing watersheds." Pp. 228–232, In T.P. Simon (Ed.). *Biological Response Signatures: Indicator Patterns Using Aquatic Communities*. CRC Press, Boca Raton, FL.

Wardynski, B.J., Winston, R.J. and Hunt, W.F. (2013). "Internal water storage enhances exfiltration and thermal load reduction from permeable pavement in the North Carolina mountains." *Journal of Environmental Engineering*, 139(2), 187-195.

Winston, R.J., Hunt, W.F., Kennedy, S.G., Wright, J.D. and Lauffer, M.S. (2012). "Field evaluation of storm-water control measures for highway runoff treatment." *J. Environ. Eng.*, 138(1), 101-111.

Chapter 3: A Comparison of Water Quality from a Urban Commercial Low Impact Development and a Conventional Development

3.1 Abstract

Urbanization and its associated increased impervious footprint lead to stream impairment through erosion, flooding, and augmented pollutant loads. Low Impact Development (LID) focuses on disconnecting impervious areas, increasing infiltration and evapotranspiration, and harvesting stormwater for use on site. LID stormwater control measures (SCMs) mitigate pollutant loadings by reducing runoff volumes and allowing for pollutant mitigation. In this study, a conventional development (centralized stormwater management) and an adjoining infiltration-based LID commercial site in Raleigh, North Carolina, were compared with respect to stormwater quality. The conventional development (2.76 ha, 61% directly connected impervious area (DCIA)) and the LID (2.53 ha, 84% DCIA) have underlying hydrologic soil group B soils, Appling and Cecil. A dry detention basin, designed to mitigate peak flow rate, was the conventional development SCM. The LID site consisted of a 44,300-liter aboveground cistern used for indoor toilet flushing, two underground cisterns (57,900 liters and 60,600 liters, used for landscape irrigation), and an underground detention system, which overflowed into 760 linear meters of infiltration galleries beneath the parking lot of the shopping center. Flow proportional, composite water quality samples were analyzed for total nitrogen (TN), total phosphorus (TP), total Kjeldahl nitrogen (TKN), total ammoniacal nitrogen (TAN), nitrite-nitrate (NO_2+NO_3), orthophosphate (Ortho-P) and total suspended solids (TSS). For the 20 events sampled (January – December 2012), the LID site pollutant

loadings for all species studied were less than 5% of pollutant loadings at the conventional site. Results from this hybrid treatment system that combined detention, stormwater harvesting, and infiltration demonstrated highly effective and space-saving solutions for areas where aboveground SCMs are not feasible due to high land costs.

Keywords. Low impact development, stormwater management, urban commercial infill, nutrient management, dry detention basin, stormwater runoff, nitrogen, phosphorus, total suspended solids, L.I.D., North Carolina

3.2 Introduction

Urban stormwater runoff is one of the leading sources of impairment in waterways (USEPA, 2009). The increased impervious area due to urbanization and development results in increased runoff volumes and pollutant loads into waterways. Nonpoint-source pollution, including urban stormwater runoff, causes export of nutrients, sediment, metals, bacteria, and other pollutants in unsustainable quantities (Makepeace et al., 1995). Of the pollutant sources described in the *National Water Quality Inventory: 2004 Report to Congress*, urban stormwater runoff is one of the leading sources of impairment in waterways in the United States (USEPA, 2009). Line et al. (2002) sampled water quality from various land uses in NC, and determined total nitrogen (TN), total phosphorus (TP), and total suspended solids (TSS) export rates increased threefold from undeveloped to developed conditions (Table 3.1). When in excess, nutrient loads cause algal blooms and eutrophication, depleting oxygen supply and reducing stream and biological health in receiving waterways (Anderson et al., 2002).

Table 3.1: Pollutant export rates of various residential land uses (Line et al., 2002)

Development Type	Pollutant Export Rate (kg/ha/yr)		
	TN	TP	TSS
Undeveloped	11.4	1.0	986
Developed	30.5	3.0	2535

Low Impact Development (LID), a term coined by Prince George's County (1999), focuses on treating stormwater onsite and mimicking predevelopment hydrology. Rather than a sole

focus on peak flow mitigation, LID stormwater control measures (SCMs) generally detain and infiltrate stormwater, providing for evapotranspiration, groundwater recharge, runoff volume mitigation, and pollutant removal mechanisms (Bean et al., 2007; Collins et al., 2008; Davis et al., 2009; Dietz, 2007; Hunt et al., 2008; Winston et al., 2012). Because LID focuses on disconnecting impervious areas, runoff volume leaving a LID is substantially less than that from a conventionally developed site, and in some cases less than that of an undeveloped site (Bedan et al., 2009; Dietz and Clausen, 2008; Hood et al., 2007; Line et al., 2012).

Studies have shown lower nutrient and TSS loading from LID compared to that of conventional development (Bedan and Clausen, 2009; Dietz and Clausen, 2008; Line et al., 2012). Dietz and Clausen (2008) sampled a residential LID and a residential conventional development and found that as impervious land cover increased from construction, nitrate-nitrite (NO_2+NO_3), total nitrogen (TN), and total phosphorus (TP) loads increased exponentially in conventional development, but no change in pollutant export was found in the LID. Total ammoniacal nitrogen (TAN) loads actually decreased in response to increased impervious areas in LID, while TAN loads continued to increase in the conventional development.

Peer-referred literature on LID for commercial sites is somewhat limited. Most studies focus on a single SCM, with Line et al. (2012) incorporating bioretention, permeable pavement, and constructed stormwater wetlands. This LID was more effective at reducing TKN, TAN, TP, and TSS pollutant loads studied compared to a conventional wet detention basin. As

compared to average runoff concentrations from eight parking lots in central North Carolina, event mean concentrations (EMCs) of runoff leaving the LID site were lower for TKN, TAN, TP, Ortho-P, and TSS (Line et al., 2012; Passeport and Hunt, 2009).

In areas of high land value or low land availability, large-scale commercial options for stormwater control may include building underground SCMs to detain and infiltrate stormwater, but the performance of LID based on underground infiltration has yet to be reported. Moreover, the performance of a hybrid system combining large-scale detention and infiltration-based SCMs with smaller LID SCMs has yet to be documented for water quality mitigation. This case study compares the water quality of a conventional commercial development to a commercial LID consisting of cisterns, aboveground swales and bioretention for water quality mitigation and a large-scale underground detention and infiltration system to capture stormwater onsite for infiltration into underlying soils.

3.3 Methodology

A conventionally-treated commercial site (61% directly connected impervious area (DCIA)) and a water harvesting and infiltration-based LID commercial site (84% DCIA) were monitored post-construction from January 2012 – December 2012 for runoff quantity and quality in Raleigh, North Carolina. Located in the piedmont region, Raleigh, the capital city, has a population of approximately 400,000 (U.S. Census Bureau, 2013). Normal average monthly temperatures range from 4.2°C in January – 25.9°C in August (SCO, 2012).

Normal precipitation for Raleigh is 1179 mm/year (SCO, 2012). The sites were adjacent to one another, so they were exposed to the same climatic conditions.

3.3.1 Site Descriptions

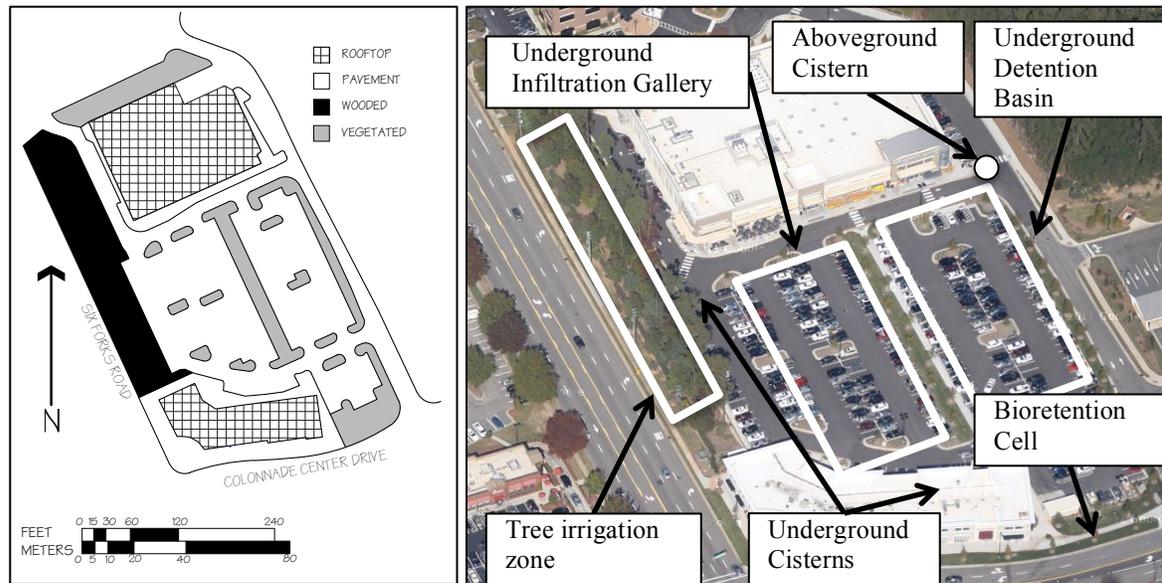


Figure 3.1: Commercial LID diagrams, from left: (a) watershed land use and characteristics, and (b) major underground SCMs and tree irrigation zone

The 2.5-ha LID site, consisting of a mix of commercial businesses and restaurants, an asphalt parking lot, several vegetated parking islands, and a preserved natural wooded area, drained into a series of aboveground and underground SCMs (Figures 3.1a, 3.1b). The drainage area and land use characteristics of the site are found in Table 3.2. The LID site was intended to (1) mimic predevelopment peak discharges for the Type II, 2-year and 10-year return period storms (142 mm/hour and 181 mm/hr, respectively; NOAA, 2006), and (2) capture and infiltrate runoff from the 25-mm water quality event.

Table 3.2: Characteristics of study sites examined in North Carolina

	LID	Conventional
Location	Intersection of Six Forks & Strickland Roads, Raleigh, NC (35.900°N, -78.651°W)	Intersection of Six Forks & Strickland Roads, Raleigh, NC (35.897°N, -78.649°W)
Year built	2011	2007
Watershed	Neuse River Basin	Neuse River Basin
Watershed composition	Commercial businesses, grocery store, and restaurant	Commercial businesses
Underlying soil	Appling (Hydr. Soil Group B)	Cecil (Hydr. Soil Group B)
Watershed area, ha	2.53	2.76
Grassed	0.13	1.09
Wooded	0.29	-
Rooftop	0.54	0.24
Parking	1.57	1.43
% of watershed as DCIA	84%	61%

Three cisterns, totaling approximately 162,800 L of storage, captured runoff from the rooftop surfaces for onsite indoor and outdoor uses. A 44,300-liter aboveground cistern and a 57,900-liter underground cistern each captured runoff from the northern rooftop (3800 m²). Water was withdrawn from the aboveground cistern (drainage area 1700m²) for indoor toilet flushing, while water in the underground cistern (drainage area 2100m²) was used for irrigation of the preserved wooded area. A drip irrigation system scheduled to cycle daily. The tank also contained a 10-mm diameter drawdown orifice, designed to empty the cistern within five days after a storm event. A third cistern (60,600 liters), located underground, captured rooftop runoff from the southern building (1600 m²) to irrigate (drip and spray) onsite landscaped areas. Parking lot runoff and rooftop runoff exceeding the capacity of the cisterns drained to an underground detention and infiltration system beneath the parking lot (Figure 3.2).



Figure 3.2: LID SCMs, clockwise from top left: (a) grassed swales, (b) pretreatment bioretention cell, (c) underground detention basin, and (d) underground infiltration trenches

Stormwater from 0.25 ha of the parking lot drained into approximately 140 m of grassed bioswales (Figure 3.2a, Figure 3.3), and the 60-m² bioretention cell captured stormwater from 0.08 ha of the parking lot (Figure 3.2b, Figure 3.3). Media in the bioretention cell and grassed bioswale was comprised of 85-88% loamy sand, 8-12% silt and clay (fines), and 3-5% organic material, with a P-index of 10-30, following NC DENR specifications (NCDENR, 2009). Overflow from these SCMs entered the 1325-m³ concrete detention chamber (Figure 3.2c). Stormwater from the 0.29 ha of wooded area drained to a depression in the wooded area for infiltration; a retaining wall separated the wooded area from draining

to the parking lot. The remainder of the stormwater (1.37 ha of parking lot drainage area) was conveyed directly into the detention chamber through drop inlets and curb inlets.

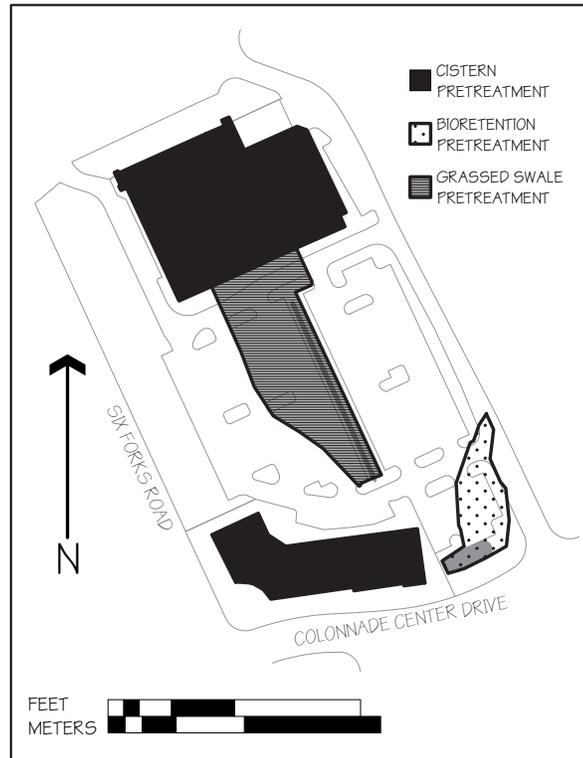


Figure 3.3: Drainage areas of cisterns, a bioretention cell, and a grassed swale.

Immediately prior to entering the detention chamber, all stormwater was routed to concrete StormTrap™ pretreatment units, including an underflow weir and baffles for oil/water separation and sediment and debris removal. The detention basin included a 38-mm diameter drawdown orifice, designed to dewater the detention basin in less than five days. Overflow from the detention chamber drained into an underground infiltration gallery, a 760 m gravel filled trench system (435 m³ of storage) overlying in-situ soil (Figure 3.2d). The infiltration

gallery was designed to have an infiltration rate of 3 mm/hr, infiltrating the 25-mm design storm in 4 days. Overflow from the infiltration gallery discharged into the municipal storm drainage system via a 380-mm diameter concrete pipe.

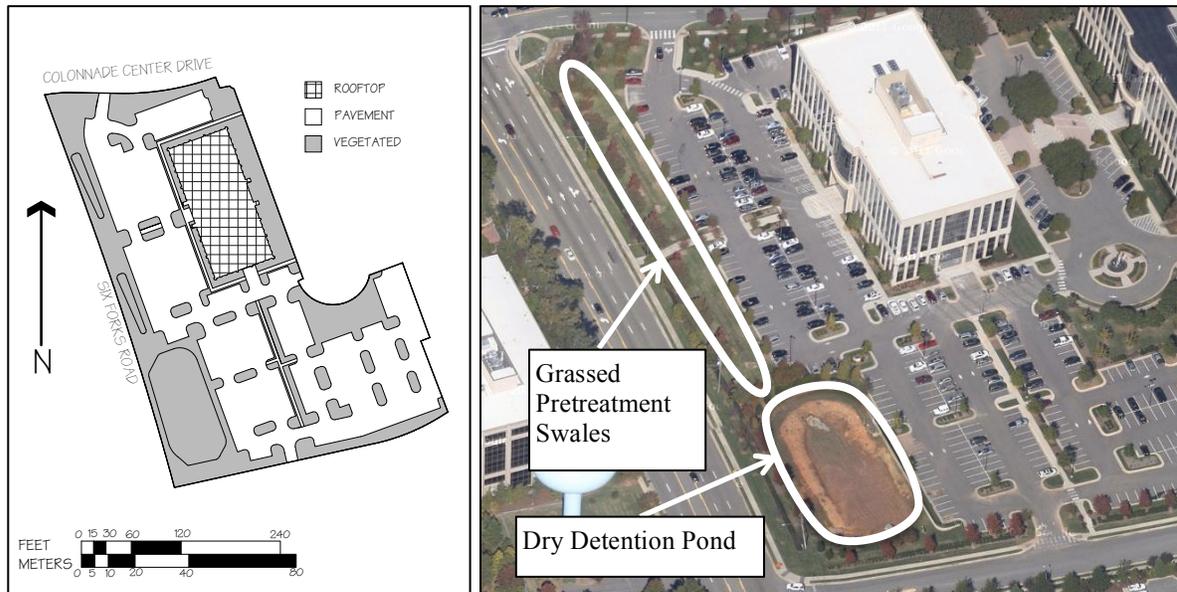


Figure 3.4: Conventional site diagrams, from left: (a) watershed land use and characteristics, and (b) major SCMs

Site characteristics and land use for the conventional site, a 2.8 ha commercial watershed, are found in Table 3.1 and Figure 3.4a. SCMs treating stormwater at this site included three grassed pretreatment swales (33 m and 29 m linear swales, and a 90 m pond perimeter swale) and a 0.14 ha dry detention basin (Figure 3.4b). Stormwater was routed to one of the three swales for pretreatment before entering the dry detention pond (Figures 3.5a, 3.5b). The dry detention pond detained and slowly released this stormwater runoff through a concrete outlet structure into the existing storm drainage network (Figure 3.5a). The dry detention pond was

designed to (1) control runoff volumes from the 1-year, 24-hour storm (72.4 mm in Raleigh, NC; NOAA, 2006), releasing this runoff volume over a minimum duration of 48 hours and (2) control peak flows from the 2 and 10-year storm events (142 mm/hour and 181 mm/hr, respectively; NOAA, 2006).



Figure 3.5: Conventional development design features, from left: (a) dry detention basin with grassed perimeter pretreatment swales and concrete outlet structure, and (b) pretreatment swales

3.3.2 *Site Cost*

The site cost breakdown of the LID site is found in Table 3.3. As a result of its large-scale underground system, the LID site was more expensive than typical SCMs. However, the high land cost in the area provides reasoning for the high SCM cost. As a result of placing SCMs underground, the increased available land use aboveground allows for more revenue for developers from larger retail lease space. Therefore, it is important to assess the costs and benefits of installing underground SCMs.

Table 3.3: LID Site Cost Breakdown

Pretreatment		\$59,700.00
Pre-treatment Units	\$40,700.00	
Bioswales	\$8,400.00	
Bioretention Area	\$10,600.00	
Underground System		\$541,900.00
Underground Detention & Settling Chamber	\$338,950.00	
Infiltration System	\$132,200.00	
Pipe Network	\$70,750.00	
Cisterns & Irrigation		\$124,500.00
Cisterns	\$111,250.00	
Spray Irrigation & Infiltration System	\$13,250.00	
Total		\$726,100.00

3.3.3 *Monitoring*

This study compared two watersheds after the implementation of conventional and LID SCMs. Aspects of two monitoring designs were used in this study: upstream/downstream and paired watershed (Clausen and Spooner, 1993). Measured discharge at the outflow (downstream) station was compared to precipitation-based estimated inflow (upstream) values for peak flow and runoff volumes at each site using the upstream/downstream monitoring design approach. However, since upstream monitoring stations were not in place at each site throughout the entirety of the monitoring period, the monitoring scheme was not truly a paired watershed design.



Figure 3.6: Monitoring equipment, from left: (a) ISCO 674 tipping bucket rain gage, (b) v-notch weir at outlet of LID site, and (c) compound v-notch weir at outlet of dry detention basin

Monitoring equipment used in this study is found on Table 3.4. An ISCO 674 tipping-bucket rain gage (Figure 3.6a) located at the conventional development site recorded precipitation depth and intensity of storm events. To check the accuracy of the automatic rain gage, a manual rain gage measured precipitation depth and was used to calibrate the tipping-bucket rain gauge. On average, the automatic rain gage data was adjusted by a correction coefficient of 1.15; the adjustment amount did not depend on rainfall intensity. Precipitation values enabled calculation of inflow volumes to estimate inflow pollutant loading.

Table 3.4: Monitoring equipment used in study

	LID	Conventional Development
Precipitation	-	ISCO 674 Automatic Rain Gage ¹
Water level	ISCO 730 Bubbler Module	ISCO 730 Bubbler Module
Stage-Discharge Relationship	30°, 150-mm V-notch weir	30°, 100-mm V-notch weir, 410-mm rectangular end restrictions
Data Storage	ISCO 6712 Sampler	ISCO 6712 Sampler

¹Rain gage was 0.25 km from the LID watershed.

Monitoring equipment was installed at the outlet to the municipal storm drainage network at each site: the 610-mm diameter outlet pipe at the conventional development was fitted with a steel compound weir (Figure 3.5b), and the 380-mm diameter outlet pipe at the LID was fitted with a steel v-notch weir (Figure 3.5c). Water levels at each site outlet were measured at 2-minute intervals throughout each storm event by ISCO 730 Bubbler Modules installed at the invert of the pipe at the LID site and inside the concrete outlet structure at the conventional site. ISCO 6712 Samplers fitted with the bubbler modules at each site recorded and stored water level data throughout each storm event. Recorded water levels were converted to storm discharge through stage-discharge weir equations. Equations and calculations are found in Appendix A.

Flow-weighted, composite water quality samples were collected during each runoff-producing storm events at each site with an ISCO 6712 automated sampler. Samples were collected within 24 hours of the cessation of rainfall, and samples were chilled to $<4^{\circ}\text{C}$ until analyzed. Composite stormwater samples were split into three bottles: (1) 20 ml was filtered through a 0.45-micron filter into a glass bottle to measure orthophosphate (Ortho-P), (2) a 250 mL plastic bottle pre-acidified with H_2SO_4 for all other nutrient forms, and (3) a non-acidified 1-liter plastic bottle for TSS.

Power failure due to insufficient battery life, holes cut in the bubbler tubing, and air in the bubbler tubing caused occasional data collection malfunction. This was not an issue at the conventional development, with the exception of one incident where the bubbler tubing was accidentally cut due to plant maintenance in the dry pond. A few occurrences of insufficient

battery life caused some data to be omitted, but overall data collection malfunction was low at both sites (4 of 24 storm events). Incorrect data were not included in analyses. To ensure that data collected were accurate, a manual level was measured after each storm, and recorded 2-minute increment data were adjusted accordingly. With the exception of three inaccurate water level readings incidences at the LID site (average error 0.047 m) and three such at the conventional site (average error 0.036 m), water level readings were accurate throughout the monitoring period.

3.3.4 Data Analysis

Nitrogen species [TKN, TAN, NO₂+NO₃], phosphorus species [TP, Ortho-P], and TSS were analyzed at the North Carolina State University Center for Applied Aquatic Ecology (CAAE) laboratory, approximately 18 km from site. Samples were analyzed using EPA and standard methods (Eaton et al., 1995; EPA Method 351.1). TN was calculated by summing TKN and NO₂+NO₃. EMCs below the practical quantitation limit (PQL) determined by the CAAE were assigned a value of one-half of the PQL for statistical purposes.

Data analysis was performed with SAS® 9.3 software (SAS Institute, Inc. 2012) to statistically compare pollutant concentration and loading differences between the two watersheds. Differences of paired data sets were first tested for normality with three goodness-of-fit tests: Kolmogorov-Smirnov, Cramer-von Mises, and Anderson-Darling. Resulting normally distributed data were tested for statistical significance using the student's t test; non-normal data were log-transformed and reanalyzed for a normal distribution. Log-

transformed data fitting a normal distribution were analyzed using the student's t test; non-normal data were analyzed for statistical significance using nonparametric tests (Wilcoxon signed rank or sign test). Due to the assumption of symmetry in the Wilcoxon signed rank test, outliers of nonparametric data were analyzed for symmetry. If the number of outliers on each tail differentiated by one or less, symmetry was assumed, and the Wilcoxon signed rank test was performed. However, excessive outliers on only one side suggested asymmetry, and so the less statistically powerful sign test was used to determine runoff differences between the two sites. Analysis of Covariance (ANCOVA) determined the effect of seasonality on results. Data were analyzed for significance at the $\alpha=0.05$ level. SAS® code is included in Appendix D.

3.4 Results

3.4.1 Precipitation Characteristics

Water quality samples were collected from January 11 – December 18, 2012. During 2012, annual precipitation of 1285 mm was 9% above the climate normal of 1179 mm. Annual rainfall was not compiled at the site; however, these annual precipitation values were observed in Raleigh, NC (Station #317079: Raleigh State Univ, 35.794°N, 78.699°W), approximately 15 km from the study site.

The 47 hydrologic storms monitored produced approximately 777 mm of rainfall, much less than that of the annual rainfall observed in Raleigh, NC. Reasoning behind a lack of annual recorded rainfall onsite was attributed to a number of factors: (1) manual rain gage readings

were not recorded during non-sampled events for the first six months of the study period, and (2) storm events that did not produce runoff (generally precipitation depths less than 5 mm) were not included in the monitored 777 mm. Therefore, it is important to emphasize that this 777 mm of recorded data was not the annual precipitation depth onsite. The 20 storm events sampled for water quality ranged in depth from 4.8 – 39.9 mm, with a mean and median event size of 18.1 and 14.9 mm, respectively.

3.4.2 Pollutant Concentrations

Median values of pollutant concentrations observed at the outlet of each watershed are shown in Table 3.5. Also included on Table 3.5 are EMCs from (1) two studies comparing LID and conventional developments in residential (Bedan and Clausen, 2009) and commercial (Line et al., 2012) watersheds, (2) effluent parking lot EMCs throughout North Carolina (Passeport and Hunt, 2009), and (3) commercial EMCs from the National Stormwater Quality Database (NSQD; Pitt et al., 2004).

Table 3.5: Summary of effluent EMCs from the conventional development and the LID

	TKN (mg/L)	TAN (mg/L)	NO ₂ +NO ₃ (mg/L)	Ortho-P (mg/L)	TSS (mg/L)	TN (mg/L)	TP (mg/L)
Conventional (<i>n</i> = 20)	0.51	0.09	0.18*	0.01	8.6	0.68	0.06
LID (<i>n</i> = 20)	0.57	0.12	0.30*	0.01	9.5	0.87	0.05
NSQD ^a	1.6	0.5	0.6	0.11	42	2.2	0.22
Parking lot runoff ^b	1.19	0.32	0.36	0.07	-	1.55	0.19
Residential – Conventional ^c	1	0.15	0.3	-	24	1.3	0.185
Residential – LID ^c	1.3	0.03	0.4	-	11	1.7	0.291
Commercial – Conventional ^d	0.87	0.13	0.13	-	31.1	1.00	0.09
Commercial – LID ^d	0.69	0.06	0.56	0.01	18.4	1.25	0.06

^aNational Stormwater Quality Database (NSQD; Pitt et al., 2004),

^bEight parking lots in NC (Passeport and Hunt, 2009)

^cJordan Cove Residential Development (Bedan and Clausen, 2009)

^dCommercial Development in NC (Line et al, 2012)

*Significantly different values

Both the LID and conventional development exhibited EMCs lower than the NSQD values (Pitt et al., 2004) by a factor of 2 to 11, indicating the relatively clean effluent stormwater at both sites, compared to a database of commercial runoff. The LID site was designed for no outflow at or below the 25-mm storm. However, outflow observed at and below this storm depth suggests a possible leak in the system, allowing for partial stormwater outflow to pass by the system untreated.

Table 3.6: Summary of median influent and effluent EMCs from the conventional development and the LID during September 9, 2012 – December 18, 2012.

	TKN (mg/L)	TAN (mg/L)	NO ₂ +NO ₃ (mg/L)	Ortho-P (mg/L)	TSS (mg/L)	TN (mg/L)	TP (mg/L)
Conventional, influent (<i>n</i> =4)	0.30	0.12	0.13	0.05	7.84	0.43	0.09
Conventional, effluent (<i>n</i> =4)	0.50	0.04	0.16	0.07	5.20	0.66	0.10
LID, influent (<i>n</i> = 6)	0.67	0.06	0.12	0.01	27.47	0.79	0.08
LID, effluent (<i>n</i> = 6)	0.57	0.20	0.22	0.01	9.13	0.79	0.04

Although inflow data were not collected for parking lot runoff during the entirety of the monitoring period, four and six samples were taken at the conventional development and the LID, respectively, during the period of September 9, 2012 to December 18, 2012, when inflow monitoring was installed (Table 3.6). Because of the low number of storms, statistics were not computed for these values.

3.4.2.1 Nitrogen Species

LID and conventional development EMCs were not significantly different for TKN ($p = 0.2774$), TAN ($p = 0.4980$), and TN ($p = 0.2969$); however, NO_2+NO_3 EMCs were 1.7 times higher in the LID discharge than that of the conventional development ($p=0.0126$). At the conventional development, apparent reduction of TAN and addition of TKN, NO_x , and TN was observed (Table 9). At the LID site, apparent reduction of TKN, addition of TAN and NO_2+NO_3 was observed, while TN appeared to remain constant (Table 3.5).

Observed TKN EMCs were lower than those observed in previous LID and conventional development studies; TAN EMCs were lower than conventional developments studied in both Bedan and Clausen (2009) and Line et al. (2012), but exceeded LID values observed in both studies. For fourteen and eleven of the twenty storms sampled, the LID site had higher TKN and TAN concentrations, respectively, than the conventional development. Large percentages of impervious areas at both the conventional development (61% DCIA) and the LID (84% DCIA) imply similarly low nutrient inputs at each site.

Influent and effluent EMCs at the conventional site suggest addition of TKN and reduction of TAN (Table 3.6), implying inputs of ON, likely due to fertilization of the grassed swales onsite; Rushton (2001) observed similar TKN EMC increases for parking lots with swales, but did not observe this trend in parking lots without swales. Any reduction of TAN at the conventional development was likely due to nitrification (oxidation of ammonium to NO_2+NO_3) in the dry detention basin water column; Rosenzweig et al. (2011) observed a similar trend of TAN reduction and NO_2+NO_3 addition in a dry detention basin. Contrarily, reduction of TKN and addition of TAN was detected at the LID site. Because TAN increased, TKN reduction was likely due to removal of ON in the system, resulting from mineralization (microbial conversion of organic nitrogen to ammonium) and subsequent nitrification likely occurring in the underground detention basin water column.

NO_2+NO_3 concentrations were higher at the LID than the conventional development for 15 of 20 storms. Compared to values from residential runoff, NO_2+NO_3 EMCs at both the conventional development and the LID had lower or equal concentrations relative to those observed in Bedan and Clausen (2009) and the LID studied by Line et al. (2012).

Observed TN EMCs were lower than those observed in previous LID and conventional development studies (Bedan and Clausen, 2009; Line et al., 2012). For fourteen of the twenty storms sampled, the LID site had higher TN concentrations than the conventional development, primarily due to higher NO_x concentrations at the LID site. Parking lot and effluent TN EMCs at the conventional development indicated addition of TN, resulting from the addition of TKN and NO_x . An outside source of nitrogen likely existed at the

conventional development, such as fertilization of the grassed swales. At the LID site, median influent and effluent EMCs were equal because reduced TKN were offset by increases of NO_2+NO_3 . This was likely a result of mineralization and nitrification occurring in the underground detention basin, transforming influent ON to TAN, and subsequently to NO_x within the system, keeping TN constant.

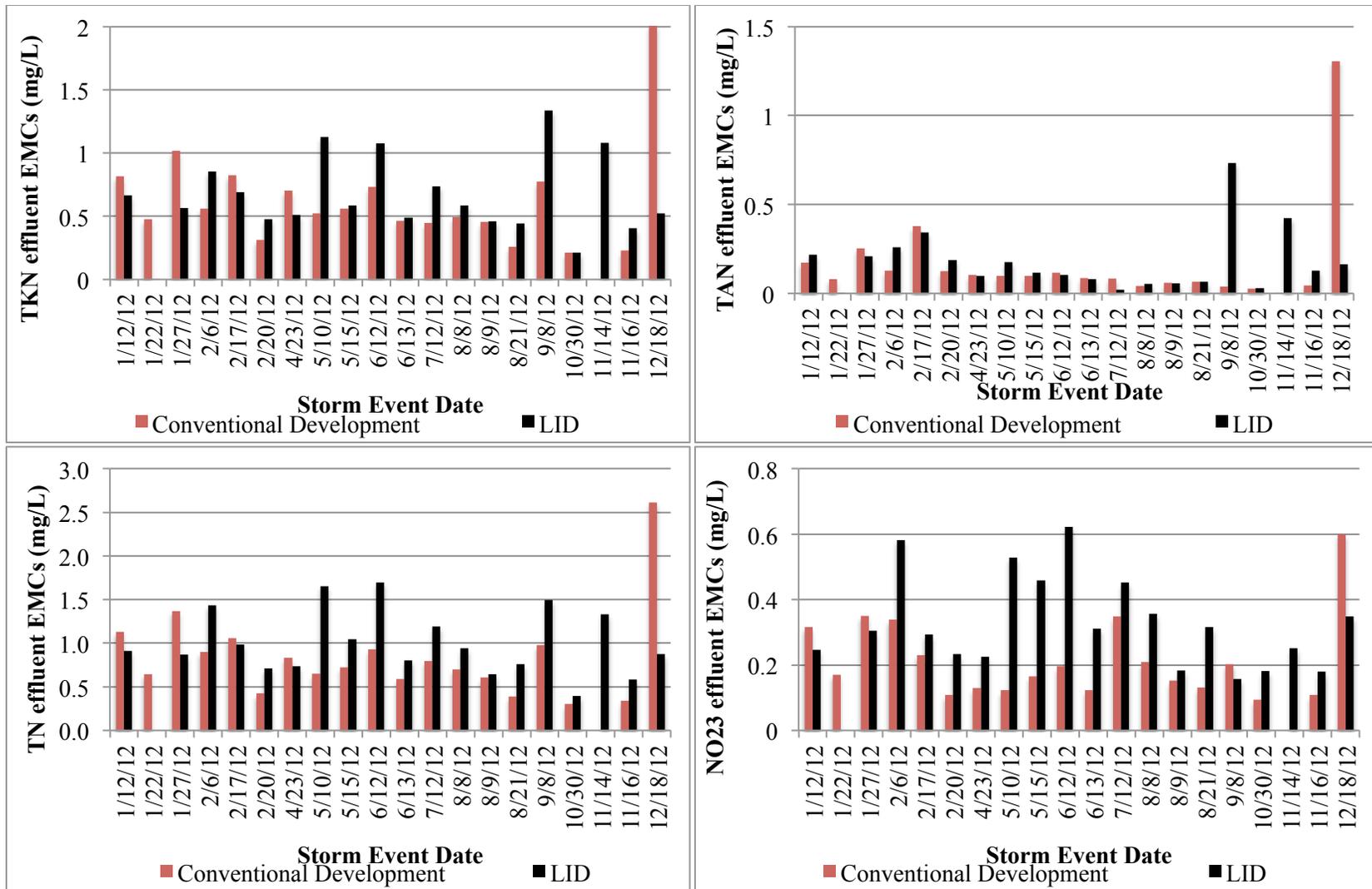


Figure 3.7: Effluent nitrogen species EMCs by storm date, clockwise from top left: (a) TKN, (b) TAN, (c) NO₂+NO₃, and (d) TN

3.4.2.2 Phosphorus Species

LID and conventional development EMCs were not significantly different for TP ($p = 0.1769$) or Ortho-P ($p = 0.0826$). For seven and six of the twenty storms sampled, the LID site had higher TP and Ortho-P concentrations, respectively, than the conventional development (Figure 3.8a, Figure 3.8b).

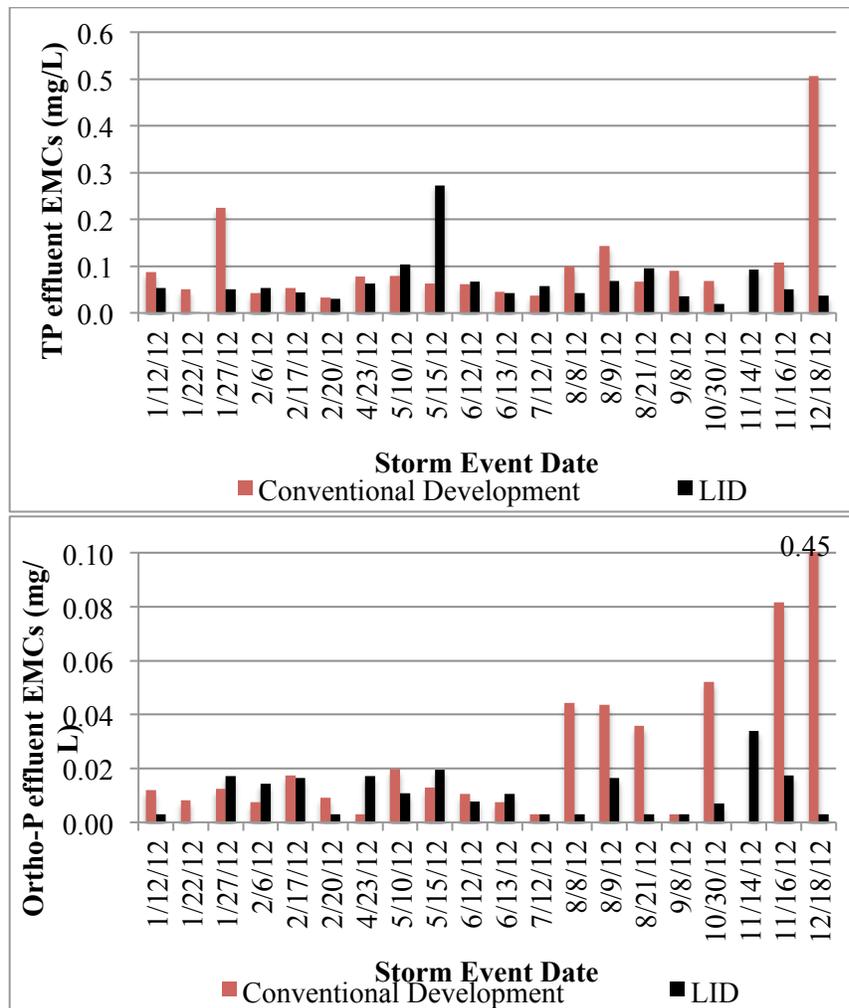


Figure 3.8: Effluent phosphorus species EMCs by storm date, from top: (a) TP, and (b) ortho-P

Observed TP and Ortho-P EMCs were lower than or equal to those observed in previous LID and conventional development studies (Bedan and Clausen, 2009; Line et al., 2012). Similar fertilization patterns at each site may cause the similarity in phosphate concentrations between each site. Large percentages of impervious areas at both the conventional development (61% DCIA) and the LID (84% DCIA) imply similarly low fertilization amounts at each site; the lack of vegetated areas may explain the low inputs of phosphorus at each site.

For six storms sampled at the LID site, apparent reduction of TP was observed, and Ortho-P remained constant (Table 3.6). Influent and effluent EMCs at the conventional site suggest a negligible change in TP and a slight addition of Ortho-P (Table 3.6). TP removal at the LID site may be due settling of particle-bound phosphorus within the underground detention basin. A slight addition of Ortho-P was observed at the conventional development (Table 3.6), suggesting inputs of phosphate, likely due to fertilization and mowing onsite, similar to changes in TN and TP. The LID site did not show a change in Ortho-P.

3.4.2.3 Total Suspended Solids

LID and conventional development EMCs were not significantly different for TSS ($p = 0.5217$). At both the conventional development and the LID, removal of TSS was observed, though only slight removal was suggested at the conventional development. During the spring season, TSS concentrations were approximately 2 times those from the winter season at the LID ($p=0.0452$), likely due to the typically high intensity of the storms during the

spring season (SCO, 2012). Average spring intensity storms (46 mm/hr) were higher than in winter (18 mm/hr), though this difference was not significant.

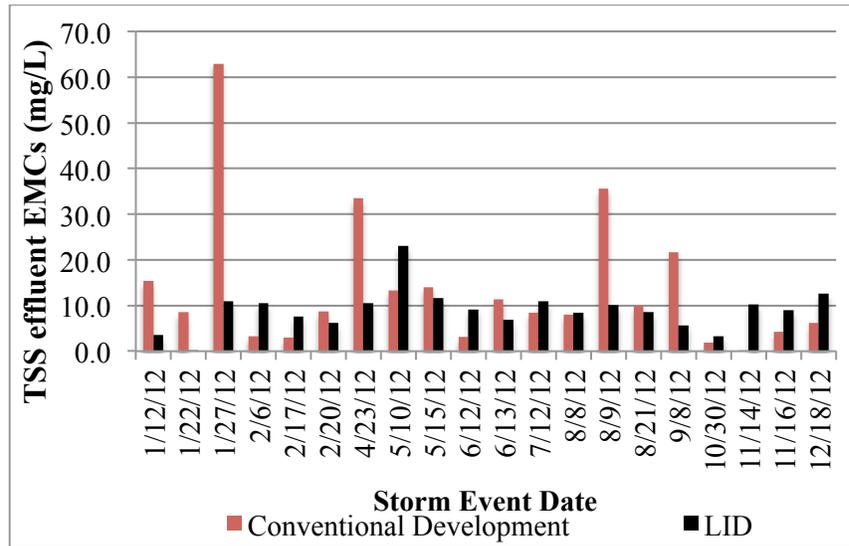


Figure 3.9: TSS effluent EMCs from the conventional development and the LID

Conventional development and LID TSS EMCs were lower than reported by comparable studies (Bedan and Causen, 2009; Line et al., 2012). Of the twenty storms sampled, ten storms exhibited a higher concentration of TSS at the LID site than the conventional site (Figure 3.9). Large percentages of impervious areas at both sites, as well as similar soil types imply comparable sediment concentrations. Also, both sites were designed with a focus on detaining stormwater, allowing for sedimentation to occur within the dry detention basin and the underground detention basin (Hossain et al., 2005). Removal of TSS was observed at the LID, in part due to relatively higher influent LID EMCs than those of the conventional development.

3.4.3 Pollutant Loadings

Though event mean concentrations were not significantly different between the two sites for nearly all pollutants, when runoff volume was taken into account to calculate pollutant loadings, the LID site discharged significantly lower ($p < 0.0001$) loads, by factors ranging from 20 to 90, for all pollutants monitored (Table 3.7).

Table 3.7: Median effluent pollutant loadings at the conventional development and the LID.

	Conventional Development ($n=20$)	LID ($n=20$)	Ratio: Conventional / LID	Test Statistic ¹	p
TKN (kg/ha/yr)	3.43	0.09	37	M = 9	<0.0001
TAN (kg/ha/yr)	0.46	0.02	24	M = 9	<0.0001
NO ₂ +NO ₃ (kg/ha/yr)	1.10	0.05	23	M = 9	<0.0001
Ortho-P (kg/ha/yr)	0.16	0.002	85	t = 7.75	<0.0001
TSS (kg/ha/yr)	96.9	1.56	62	M = 9	<0.0001
TN (kg/ha/yr)	4.52	0.14	32	M = 9	<0.0001
TP (kg/ha/yr)	0.53	0.01	52	S = 104	<0.0001

¹ Test statistics M (Sign test), t (Student's t test), and S (Wilcoxon signed rank test)

Although the TN EMCs at each site were not significantly different, effluent TN loadings were 30 times higher at the conventional development than at the LID, demonstrating the importance of volume reduction for pollutant mitigation. As illustrated in Figure 3.11, the LID outperforms both sites studied in Line et al. (2012) as well as the mean value of two parking lots with swales in NC (Rushton, 2001), while the conventional development exports more TN mass than any of these studies.

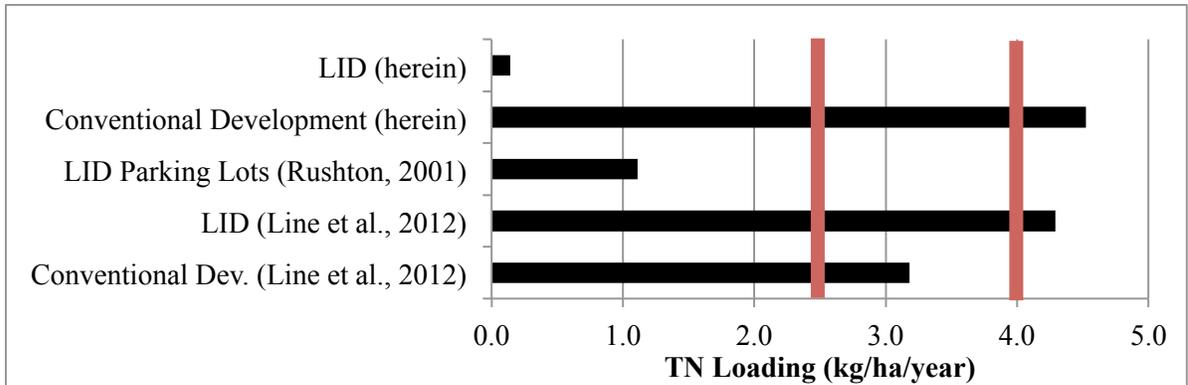


Figure 3.10: TN effluent loading at the conventional development and the LID, compared with similar literature. Vertical lines shown at 2.5 kg/ha/year and 4.0 kg/ha/year represent local regulations adjacent to (Falls Lake Watershed) and within (Neuse River Basin) the site watershed, respectively

The LID discharges substantially less TN than the thresholds established for the Neuse River Basin, the sites' watershed (4.0 kg/ha/year), and the Falls Lake Watershed, a nearby watershed within the Neuse River Basin (2.5 kg/ha/year), by factors of nearly 28 and 18, respectively (NCDENR, 1999; NCDENR, 2010; Figure 3.10). However, the conventional site did not meet either of these regulations, instead exporting 0.52 kg/ha/year more TN than the threshold for new development in the Neuse River Basin.

The conventional development exported 52 times more TP mass than the LID ($p < 0.0001$). This LID discharged less TP mass than other commercial LID sites, by factors of 23 (Line et al., 2012) to 42 (Rushton, 2001), although similar EMCs were observed in Line et al. (2012) (Table 3.4), again emphasizing the importance of runoff volume reduction in reducing pollutant loadings.

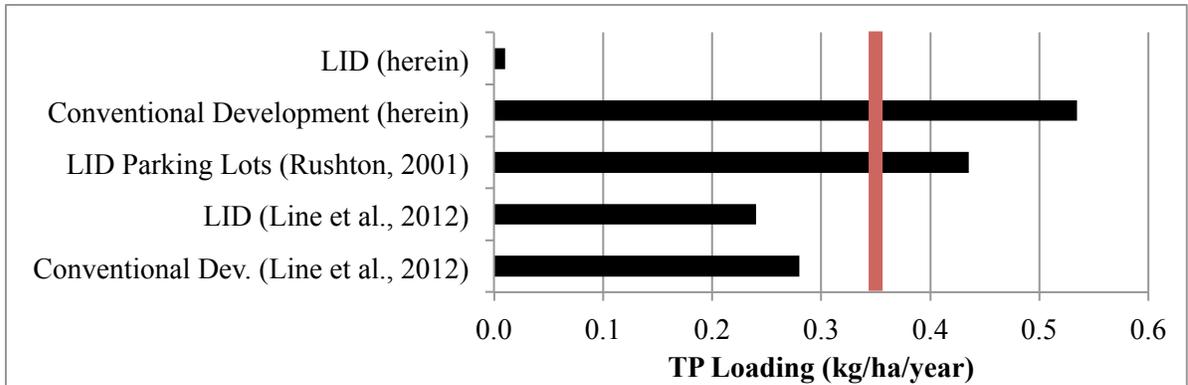


Figure 3.11: TP effluent loading at the conventional development and the LID, compared with similar literature. A vertical line shown at 0.35 kg/ha/year represents local regulations (Falls Lake Watershed) adjacent to site

The LID mitigated mass TP loadings below the Falls Lake Watershed threshold (0.35 kg/ha/year) by a factor of 34. However, the conventional development exported approximately 1.5 times this limit (Figure 3.11). The conventional development exported approximately 63 times more TSS than the LID ($p < 0.0001$). Figure 3.12 illustrates the TSS effluent loadings observed in this study as compared to Line et al. (2012)'s commercial development and LID-type parking lots studied by Rushton (2001).

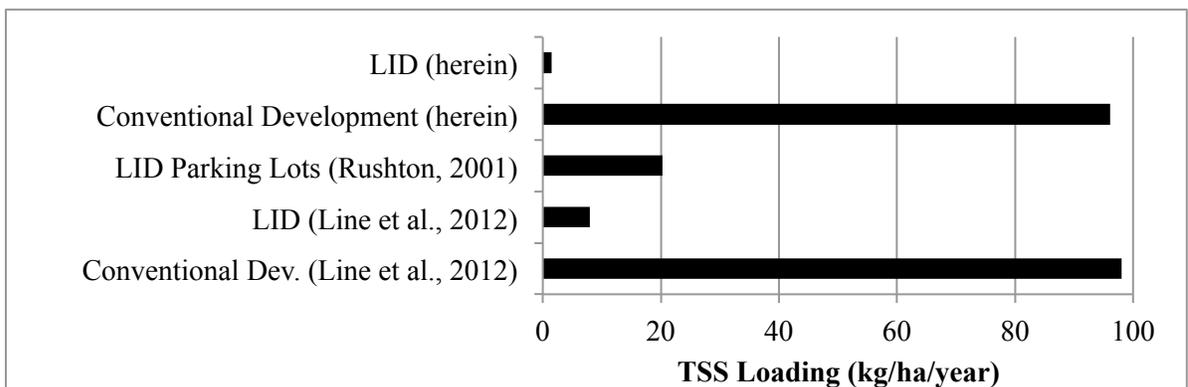


Figure 3.12: TSS effluent loading at the conventional development and the LID, compared with similar literature

Although the conventional development EMCs were approximately 25% of those at the conventional development studied by Line et al. (2012), both conventional developments exported approximately the same mass of TSS, at 96 kg/ha/year and 98 kg/ha/year, respectively. In contrast, all three LID sites exported substantially lower pollutant loads of 8 kg/ha/year (Line et al., 2012), 20 kg/ha/year (Rushton, 2001) and 2 kg/ha/year (herein, Figure 3.13). Line et al. (2012) attributed the TSS load differential between their two sites as a function of higher runoff volume mitigation and EMC reduction at the LID. Although the effluent TSS EMCs at Line et al. (2012)'s LID were approximately 2 times higher than this LID, Line et al. (2012) reported TSS loadings 5.3 times higher than observed herein, emphasizing the importance of runoff volume mitigation for pollutant mitigation.

3.5 Conclusions

EMCs discharged at each site were lower than parking lot effluent in NC (Passeport and Hunt, 2009) and a range of commercial landscapes across the U.S. (Pitt et al., 2004).

Although EMCs were not generally significantly different between the conventional development and the LID, the conventional development exported a factor of 23 – 85 times more pollutants than the LID. This was due to the higher runoff volume reduction from the large-scale detention and infiltration system at the LID site, stressing the importance of both pollutant and hydrologic mitigation to maximize function.

The LID site discharged substantially less pollutant loadings than similar LID commercial studies as well (Line et al., 2012; Rushton, 2001) due to the high infiltration capacity of the

underlying hydrologic type B soils and the proportionally large infiltrative footprint. Such a large SCM footprint would be considered “overdesigning” the SCMs to capture the entire design storm for infiltration, rather than solely meeting peak flow requirements. Pairing large-scale detention and infiltration SCMs with infiltrating LID SCMs as in this design resulted in mitigation of pollutant loadings below similar peer-reviewed LID studies.

Results from this innovative combined detention, stormwater harvesting, and infiltration LID system will provide space-saving solutions for areas where aboveground SCMs, such as dry detention basins and constructed stormwater wetlands, are not feasible due to high land costs and constricted spaces.

3.6 References

- Anderson, D.M., Gilbert, P.M. and Burkholder, J.M. (2002). "Harmful algal blooms and eutrophication: Nutrient sources, composition, and consequences." *Estuaries*. 25(4), 704-726.
- Bean, E.Z., Hunt, W.F. and Bidelspach, D.A. (2007). "Evaluation of four permeable pavement sites in Eastern North Carolina for runoff reduction and water quality impacts." *Journal of Irrigation and Drainage Engineering*, 133(6), 583-592.
- Bedan, E.S. and Clausen, J.C. (2009). "Stormwater Runoff Quality and Quantity From Conventional and Low Impact Development Watersheds." *J.Am.Water Resour.Assoc.*, 45(4), 998-1008.
- Clausen, J.C. and Spooner, J. (1993). "Paired watershed study design." EPA 841-F-93-009. U.S. Environmental Protection Agency, Office of Water, Washington, D.C.
- Collins, K.A., Hunt, W.F. and Hathaway, J.M. (2008). "Hydrologic comparison of four types of permeable pavement and standard asphalt in Eastern North Carolina." *J. Hydrol. Eng.*, 13(12), 1146-1157.
- Davis, A.P., Hunt, W.F., Traver, R.G. and Clar, M. (2009). "Bioretention technology: Overview of current practice and future needs." *J. Environ. Eng.*, 135(3), 109-117.
- Dietz, M.E. (2007). "Low impact development practices: A review of current research and recommendations for future directions." *Water Air and Soil Pollution*, 186(1-4), 351-363.
- Dietz, M.E. and Clausen, J.C. (2008). "Stormwater runoff and export changes with development in a conventional and low impact subdivision." *J.Environ.Manage.*, 87(4), 560-566.
- Eaton, A.D., Clesceri, L.S. and Greenberg, A.R. (1995). *Standard Methods for the Examination of Water and Wastewater*, 19th Edition. American Public Health Association. Washington, D.C.
- Hood, M.J., Clausen, J.C. and Warner, G. S. (2007). "Comparison of stormwater lag times for low impact and conventional residential development." *J.Am.Water Resour.Assoc.*, 43(4), 1036-1046.
- Hossain, M.A., Alam, M., Yonge, D.R. and Dutta, P. (2005). "Efficiency and flow regime of a highway stormwater detention pond in Washington, USA" *Water, Air, and Soil Pollution*, 164(1), 79-89.

Hunt, W.F., Smith, J.T., Jadlocki, S.J., Hathaway, J.M. and Eubanks, P.R. (2008). "Pollutant removal and peak flow mitigation by a bioretention cell in urban Charlotte, NC." *J. Environ. Eng.*, 134(5), 403-408.

Line, D.E., Brown, R.A., Hunt, W.F. and Lord, W.G. (2012). "Effectiveness of LID for Commercial Development in North Carolina" *J. Environ. Eng.* 138(6), 680-688.

Line, D.E., White, N.M., Osmond, D.L., Jennings, G.D. and Mojonnier, C.B. (2002). "Pollutant export from various land uses in the upper Neuse River Basin." *Water Environ. Res.*, 74(1), 100-108.

Makepeace, D.K., Smith, D.W. and Stanley, S. J. (1995). Urban stormwater quality: Summary of contaminant data. *Critical Reviews in Environmental Science and Technology*, 25(2), 93–139.

NCDENR (North Carolina Department of Natural Resources). (2010). "Falls Reservoir Water Supply Nutrient Strategy: Stormwater Management for New Development."

NCDENR (North Carolina Department of Natural Resources). (1999). "Neuse River Basin: Model Stormwater Program for Nitrogen Control."

NOAA (National Ocean and Atmospheric Administration). (2006). "NOAA Atlas 14. Vol. 2. Ver. 3." *Precipitation data frequency server*, National Weather Service, Silver Spring, MD., website: <http://hdsc.nws.noaa.gov/hdsc/pfds/>

Passepport, E. and Hunt, W.F. (2009). "Asphalt parking lot nutrient characterization for eight sites in North Carolina, USA." *J. of Hydrol. Eng.*, 14(4), 352-361.

Prince George's County. (1999). "Low-Impact Development design Strategies. An Integrated Design Approach."

Rushton, B.T. (2001). "Low-impact parking lot design reduces runoff and pollutant loads." *J. Water Resour. Plann. Manage.* 127(3), 172-179.

State Climate Office of North Carolina (SCO). 2012. State Climate Office of North Carolina. North Carolina State University. website: <http://www.nc-climate.ncsu.edu>

U.S. Census Bureau. (2013). *State & county Quickfacts: Wake County, N.C.* website: <http://quickfacts.census.gov>

USEPA (United States Environmental Protection Agency). (2009). "National Water Quality Inventory: 2004 Report to Congress."

USEPA (United States Environmental Protection Agency). (1983). "Results of the nationwide urban runoff program." *NTIS PB84-185552*, Washington, D.C.

Chapter 4: Assessment of a Rainwater Harvesting System as a Component of an Innovative, Underground Low Impact Development

4.1 Abstract

The USEPA's *National Water Quality Inventory: 2004 Report to Congress* identifies urban stormwater runoff as one of the leading sources of impairment in waterways (USEPA, 2009). Low Impact Development (LID) emphasizes water use onsite through the use of rainwater harvesting (RWH) systems. Historically implemented in arid or semi-arid regions, RWH has recently surged in popularity in more humid regions, such as the southeastern U.S., due to increased interest in water conservation and severe drought conditions. A LID commercial site in Raleigh, NC, incorporates RWH with other SCMs to mitigate runoff quantity and quality. The RWH system consists of a 44,300-liter aboveground cistern used for indoor toilet flushing, and two underground cisterns (57,900 liters and 60,600 liters, used for landscape irrigation). The 57,900-liter cistern was monitored to determine the rooftop water quality (influent) and the water quality exiting the cistern via a drip irrigation system (effluent). Samples were analyzed for total nitrogen, total phosphorus, total Kjeldahl nitrogen, total ammoniacal nitrogen (TAN), nitrite-nitrate (NO_2+NO_3), orthophosphate and total suspended solids (TSS). TAN concentrations increased 1.3 times from inflow to outflow, while NO_2+NO_3 concentrations and TSS concentrations decreased significantly. All other pollutants were not significantly different between the inflow and outflow. All pollutants monitored, however, were less than previously published observed rainfall pollutant concentrations, indicating the relatively clean water onsite.

Keywords: Rainwater harvesting, low impact development, stormwater management, urban commercial infill, nutrient management, irrigation, stormwater runoff, nitrogen, phosphorus, total suspended solids, L.I.D., North Carolina

4.2 Introduction

Low Impact Development (LID), a term coined by Prince George's County (1999), describes an approach to stormwater management, focusing on minimizing stormwater impacts onsite and mimicking predevelopment hydrology. Restoring the predevelopment hydrology of a site places a focus on increasing evapotranspiration and infiltration, decreasing runoff volumes and peak flows, and addressing water capture and use onsite (Perrin et al., 2009).

LID emphasizes water use onsite through the use of rainwater harvesting (RWH) systems. The primary objective for these systems is to provide an alternative water source for non-potable needs such as irrigation, toilet flushing, car washing, and laundry. Up to a 70% reduction in the demand for potable water can be expected for an LID incorporating RWH systems versus a conventional development (van Roon, 2007).

Historically implemented in arid or semi-arid regions, RWH has recently surged in popularity in more humid regions, such as the southeastern U.S., due to increased interest in water conservation, severe drought conditions, restrictions on lawn irrigation stemming from drought conditions, and economic incentives (DeBusk et al., 2013; Jones and Hunt, 2010).

If a RWH system is designed and used properly, regulatory credit is given for hydrologic mitigation of runoff volume and peak flow rates by some authorities (e.g. NCDENR, 2009).

Rooftop runoff captured by RWH systems, may contain pollutants such as metals, organics, sediment, and microbiological contaminants (Abbasi and Abbasi, 2011). Though no regulatory credit is given to RWH for nutrient removal, observed reductions in particle-

bound TN (81%) and TP (90%), as well as TSS (97%) loads have been *modeled* due to the sedimentation potential of RWH systems (Khastigir and Jayasuriya, 2010). Rainwater harvesting not only provides for a safe and supplemental water supply, but it also has the potential for mitigation of sediment-bound pollutants.

4.3 Methodology

A RWH component of a LID commercial site in Raleigh, North Carolina, was monitored from January 2012 – December 2012 for runoff quality. Located in the piedmont of North Carolina, Raleigh, the capital city of NC, has a population of approximately 400,000 (U.S. Census Bureau, 2013). Normal average monthly temperatures range from 4.2°C in January – 25.9°C in August (SCO, 2012). Normal precipitation for Raleigh, NC is 1179 mm/year (SCO, 2012).

4.3.1 Site Description

The 2.5-ha commercial LID site, consisting of a mix of businesses and restaurants, an asphalt parking lot, vegetated parking islands, and a preserved natural wooded area drained into a series of aboveground and underground SCMs, including three cisterns (Figure 4.1). The drainage area and land use characteristics of the site are found in Table 4.1. The LID site was intended to (1) match predevelopment peak discharges for the Type II, 2-year and 10-year return period storms (142 mm/hour and 181 mm/hr, respectively; NOAA, 2006), and (2) capture and infiltrate runoff from the 25-mm water quality event.

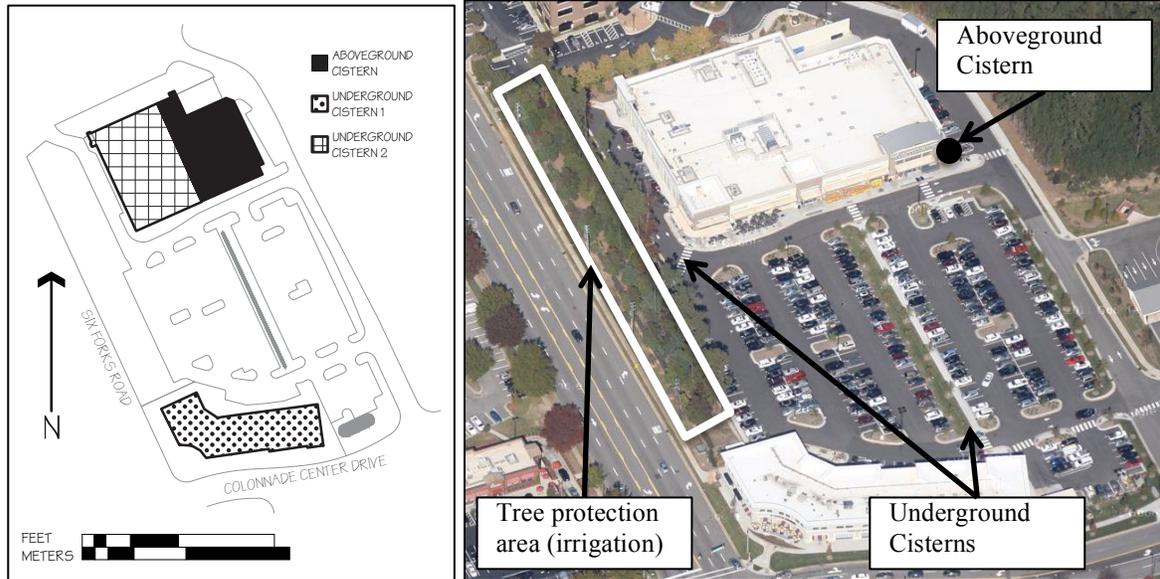


Figure 4.1: From left: (a) drainage area of each cistern, and (b) placement of cisterns onsite

Three cisterns, totaling approximately 162,800 L of storage, captured runoff from the rooftops and utilized the water onsite for indoor and outdoor needs (Figure 4.2). A 44,300-liter aboveground cistern (Figure 4.2a) and a 57,900-liter underground cistern, hereafter referred to as Underground Cistern 1, (Figure 4.2b) each captured runoff from the northern rooftop (3,800 m² surface area). The aboveground cistern (drainage area 1,700m²) was used for indoor toilet flushing, while water in Underground Cistern 1 (drainage area 2,100m²) was used to irrigate the tree protection area through a drip irrigation system scheduled to cycle daily. A third cistern (60,600 liters), Underground Cistern 2, located underground, captured rooftop runoff from the southern building (1,600 m²) for use in the irrigation (drip and spray) of site landscaped areas. Parking lot runoff and rooftop runoff exceeding the capacity of the cisterns drained to an underground detention and infiltration system beneath the parking lot.

Table 4.1: Site characteristics of the LID site studied

Location	Intersection of Six Forks & Strickland Roads, Raleigh, NC (35.900°N, -78.651°W)
Year built	2011
Watershed	Neuse River Basin
Land uses	Commercial businesses, grocery store, and restaurant
Underlying soil	Appling (Hydr. Soil Group B)
Drainage area, ha	0.54
Aboveground Cistern	0.17
Underground Cistern 1	0.21
Underground Cistern 2	0.16
% of catchment as DCIA	100%

Underground Cistern 1, the focus of this study, contained a 10-mm diameter drawdown orifice that emptied the tank within five days after a storm event. In order to irrigate the tree protection area, the water was pumped through drip irrigation lines; the submersible pump was located in Underground Cistern 1.



Figure 4.2: Three cisterns, from left: (a) the aboveground cistern, (b) Underground Cistern 1, and (c) Underground Cistern 2

4.3.2 Monitoring

The inflow and outflow of Underground Cistern 1 were monitored to determine how water quality changed from entry into the cistern to that which exited the cistern when applied onto the tree protection area. Figure 4.3 identifies monitoring locations. Representative inlet samples were collected at one of four contributing downspouts to the cistern. Outflow monitoring was conducted underneath the drip irrigation lines in the tree protection area.

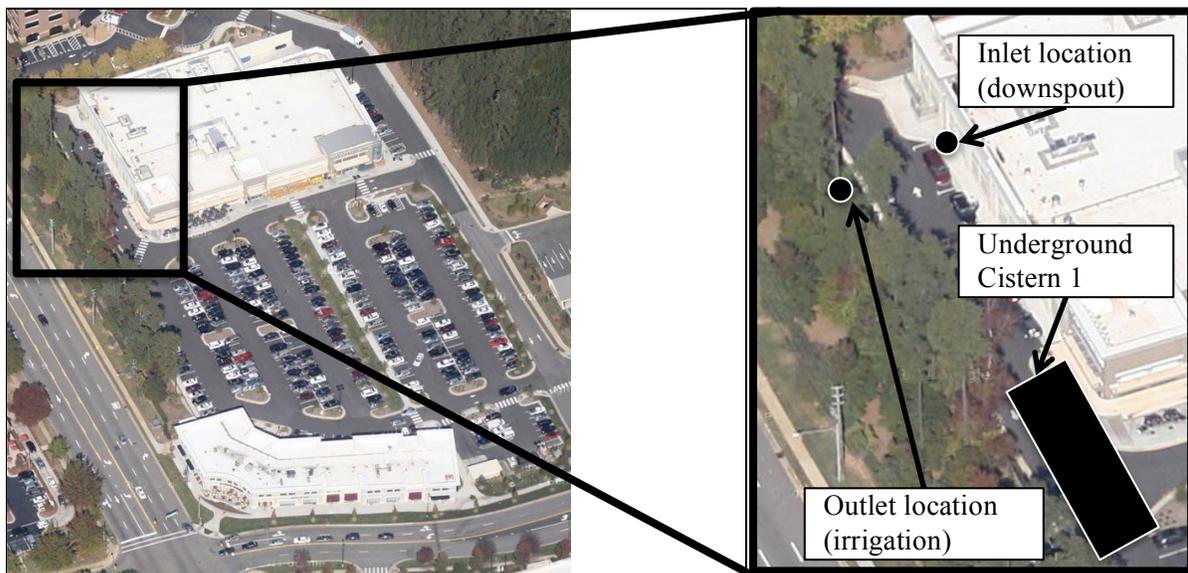


Figure 4.3: Monitoring location of Underground Cistern 1

An ISCO 674 tipping-bucket rain gage located adjacent to the monitoring site recorded precipitation depth and intensity from storm events. A manual rain gage that measured precipitation depth calibrated the automatic rain gage,. On average, a correction coefficient

for adjustment of automatic rain gage data was 1.15, and the adjustment was independent of rainfall intensity.

The inlet monitoring station included a flow diversion device constructed inside the downspout to capture flow-proportional samples, a 20L sampling bottle fitted with an overflow tube, and a box to house the sampling bottle (Figure 4.4). The flow diversion device consisted of bubbler tubing (3.2-mm diameter) supported by an aluminum arm, used to place the bubbler tubing in the interior corner of the downspout, where high flow was observed (Figure 4.4a). This device captured a small portion of each storm event, partially filling the 20L sample bottle with a flow-proportional fraction of the influent stormwater (Figure 4.4b). The system did not overflow during this study.



Figure 4.4: Inlet (downspout) monitoring setup, from left: (a) flow diversion device constructed to capture flow-proportional samples, (b) connection of downspout to sampling bottle, and (c) monitoring box

The outlet monitoring station was located in the tree preservation area underneath a drip irrigation line. Because the drip irrigation system routinely ran for one hour daily, the drip

irrigation system collected a representative sample of water exiting the irrigation system. Outflow sampling occurred within one day of a storm event. Collecting water from one perforation in the drip irrigation line produced a sufficient sampling volume. A 20L sampling bottle fit with a funnel captured water (Figure 4.5).



Figure 4.5: Outlet (irrigation) monitoring setup, from left: (a) sampling bottle with funnel, and (b) monitoring setup with irrigation line

Flow-weighted, composite influent water quality samples were taken throughout each runoff-producing storm event. Effluent samples were collected at the completion of each storm event. Samples were collected within 24 hours of the cessation of rainfall and were chilled to $<4^{\circ}\text{C}$ until analyzed. Composite stormwater samples were split into three bottles: (1) 20 ml was filtered through a 0.45-micron filter into a glass bottle to measure orthophosphate (Ortho-P), (2) a 250-ml plastic bottle pre-acidified with H_2SO_4 for all other nutrient forms, and (3) a non-acidified 1-liter plastic bottle for TSS. Malfunction at the irrigation system

occurred at several points during the monitoring period, including a period of April 23 – June 13. This caused a gap in data collection for the spring season.

4.3.3 Data Analysis

Nitrogen species [TKN, TAN, NO₂+NO₃], phosphorus species [TP, Ortho-P], and TSS were analyzed at the North Carolina State University Center for Applied Aquatic Ecology (CAAE) laboratory, approximately 18 km from site. Samples were analyzed using EPA and standard methods (Eaton et al., 1995). TN was calculated by adding concentrations of TKN to those of NO₂+NO₃. EMCs below the practical quantitation limit (PQL) determined by the CAAE were assigned a value of one-half of the PQL for statistical purposes.

Data analysis was performed with SAS® 9.3 software (SAS Institute, Inc. 2012) to statistically compare pollutant concentration and loading differences between the two watersheds. Differences of paired data sets were first tested for normality with three goodness-of-fit tests: Kolmogorov-Smirnov, Cramer-von Mises, and Anderson-Darling. Resulting normally distributed data were tested for statistical significance using the student's t test; non-normal data were log-transformed and reanalyzed for a normal distribution. Log-transformed data fitting a normal distribution were analyzed using the student's t test; non-normal data were analyzed for statistical significance using nonparametric tests (Wilcoxon signed rank or sign test). Due to the assumption of symmetry in the Wilcoxon signed rank test, outliers of nonparametric data were analyzed for symmetry. If the number of outliers on each tail differentiated by one or less, symmetry was assumed, and the Wilcoxon signed rank

test was performed. However, excessive outliers on only one side suggested asymmetry, and so the less statistically powerful sign test was used to determine runoff differences between the two sites. Analysis of Covariance (ANCOVA) determined the effect of seasonality on results. Data were analyzed for significance at the $\alpha=0.05$ level. SAS® code is included in Appendix D.

4.4 Results

4.4.1 Precipitation Characteristics

Water quality samples were collected from January 11 – February 20, 2012 and July 10 – December 18, 2012. During 2012, annual precipitation of 1285 mm was 9% above the climate normal of 1179 mm. As a result of an occasionally malfunctioning automatic rain gage, annual rainfall was not compiled at the site; however, these annual precipitation values were observed in Raleigh, NC (Station #317079: Raleigh State Univ., 35.794°N, 78.699°W), approximately 15 km from the study site. The 17 storm events sampled for water quality ranged in depth from 4.8 – 39.9 mm, with a mean and median event size of 18.1 and 14.9 mm, respectively. Collected occurred throughout the winter (6 samples), summer (6 samples), and fall (5 samples) seasons, but a malfunctioning irrigation system prevented spring sample collection.

4.4.1 Pollutant Concentrations

Influent and effluent pollutant concentrations were compared for each of 17 monitored storm events (Table 4.2). Concentrations of TKN, TN, Ortho-P, and TP were slightly higher in effluent concentrations than influent concentrations, although these changes were not significant. Significant TAN addition (+67%), NO₂+NO₃ reduction (-17%), and TSS reduction (-53%) were likely caused by settling and nitrogen cycle processes within the tanks.

Table 4.2: Mean observed influent and effluent EMCs, and mean nutrient reductions

	Inflow n = 17	Outflow n = 17	Change ¹	Significance ²	Paired Differences	
					Test Statistic	p
TKN (mg/L)	0.46	0.63	-	N.S.	t = -1.65	0.1178
TAN (mg/L)	0.24	0.32	+67%	*	t = -2.29	0.0362
NO ₂ +NO ₃ (mg/L)	0.37	0.29	-17%	*	t = 2.61	0.0189
TN (mg/L)	0.84	0.92	-	N.S.	t = -0.77	0.4531
Ortho-P (mg/L)	0.007	0.009	-	N.S.	t = 0.31	0.7581
TP (mg/L)	0.02	0.03	-	N.S.	M = -3	0.1796
TSS (mg/L)	5.44	1.81	-53%	***	t = 6.18	<0.0001

¹Positive change = addition, negative change = reduction

²N.S. = Not significant, * $p < 0.05$, ** $p < 0.01$, *** $p < 0.0001$

4.4.2.1 Nitrogen Species

Though TN and TKN concentrations were not found to be statistically different between inflow and outflow, TAN increased 67% ($p=0.0362$), and NO₂+NO₃ decreased 17% ($p=0.0189$) between outflow and inflow (Figure 4.6). The reduction in NO₂+NO₃ EMCs may be a result of several processes occurring, including denitrification of NO₂+NO₃ into

gaseous N_2 and/or dissimilatory nitrate reduction to ammonium (DNRA; Reddy and DeLaune, 2008). If the sole process of NO_2+NO_3 reduction were denitrification, a reduction of TN would have been observed; however, a slight of addition of TN was observed instead. DNRA, the process by which anaerobic bacteria convert NO_2+NO_3 to TAN, may explain both the reduction in NO_2+NO_3 and the addition of TAN. Though particulate nitrogen was not sampled, another possible process to explain TAN addition is conversion of particulate organic nitrogen (ON) to dissolved ON to TAN. DeBusk and Hunt (submitted) observed a similar trend of NO_2+NO_3 reduction and TAN addition in a RWH system in Raleigh, NC, attributing this nitrogen transformation to denitrification, DNRA, or conversion of ON to TAN. Because anaerobic conditions and particulate ON were not tested in this study, it cannot be concluded which combination of nitrogen processes occurred within the RWH system.

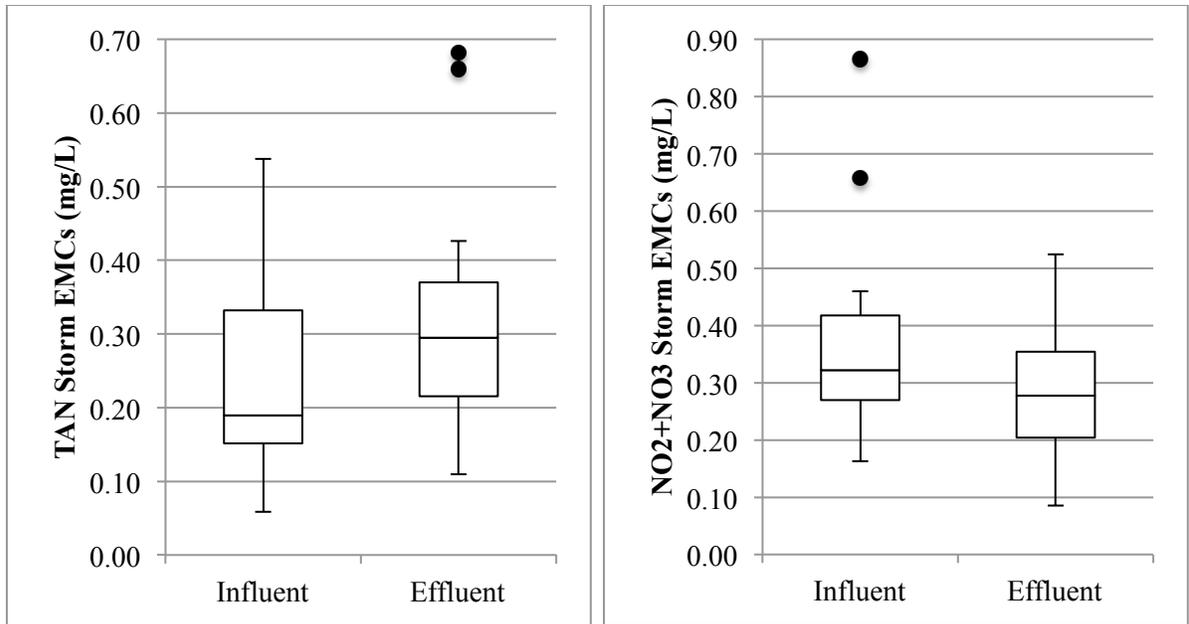


Figure 4.6: Influent and Effluent storm EMCs, from left: (a) TAN, and (b) NO_2+NO_3

Previous RWH and rooftop studies have reported comparable TN influent and effluent EMCs. Hathaway et al. (2008) reported TN EMCs of 0.06 mg/L to 2.41 mg/L for a rooftop in Goldsboro, NC; TN rooftop EMCs of 0.26 mg/L to 20.36 mg/L were reported for four rooftops in Raleigh, NC (DeBusk and Hunt, submitted). DeBusk and Hunt (submitted) reported comparable TN effluent EMCs of 0.24 to 2.39 mg/L in Raleigh, NC. However, Despins et al. (2009) reported mean effluent EMCs ranging from 1.5 ± 0.4 mg/L to 2.0 ± 0.6 mg/L in Ontario, Canada, higher than the observed mean TN EMC herein (0.84 mg/L).

With the exception of NO_2+NO_3 , nutrient reduction was not observed in this RWH system. DeBusk and Hunt (submitted) reported significant reductions of TN, TKN, TAN, and NO_2+NO_3 EMCs; however, the only significant change at this site was an increase in TAN. Reduction in NO_2+NO_3 (17%) was less than observed by DeBusk and Hunt (submitted)

(47.9%). As observed in DeBusk and Hunt (submitted), a majority of nitrogen species reduction occurred over a period of days, due to the biological and chemical nature of nitrogen removal processes. The underperformance of this RWH may be due to a short detention time, as the irrigation system was scheduled to run daily, often allowing water detained only for several hours to exit via the irrigation system relatively untreated.

4.4.2.2 Phosphorus Species

Neither TP nor ortho-P was significantly different between the two monitoring locations (Table 4.2). The lack of phosphorus mitigation was likely due to relatively low rooftop TP and ortho-P EMCs (0.02 mg/L and 0.007 mg/L, respectively). Of the 17 storms sampled, three influent and four effluent TP EMCs were below the PQL of 0.01 mg/L, indicating relatively low concentrations and a lack of further reduction capability. Similarly, ortho-P EMCs were below the PQL of 0.006 mg/L for nine influent and twelve effluent storm events sampled.

In Hathaway et al. (2008), TP EMCs for rainfall were 0.05 mg/L on average, approximately 2 times greater than both influent and effluent EMCs in this study, confirming (1) the low phosphorus concentrations and (2) low potential of further nutrient reduction via the cistern. Influent TP concentrations ranging from 0.005 mg/L to 3.85 mg/L were reported by DeBusk and Hunt (submitted), with a median concentration of 0.03 mg/L, approximately 1.5 times that observed herein. Bannerman et al. (1993) reported an average TP EMC of 0.20 mg/L for

commercial rooftops, approximately 10 times the median EMC observed herein. Due to the low EMCs observed, no reduction of phosphorus species is likely.

4.4.2.3 Total Suspended Solids

Mean influent and effluent TSS concentrations are found in Table 4.2 and illustrated in Figure 4.7. Of the 17 storm events monitored, three influent and 14 effluent EMCs were below reported PQLs, indicating that despite clean influent, effluent TSS concentrations were still reduced (Figure 4.7). A mean TSS reduction of 53% was observed, most likely due to particle settling within the cistern. The median influent TSS EMC at this site (5.44 mg/L) was less than reported by similar sites, indicating the relatively clean influent water.

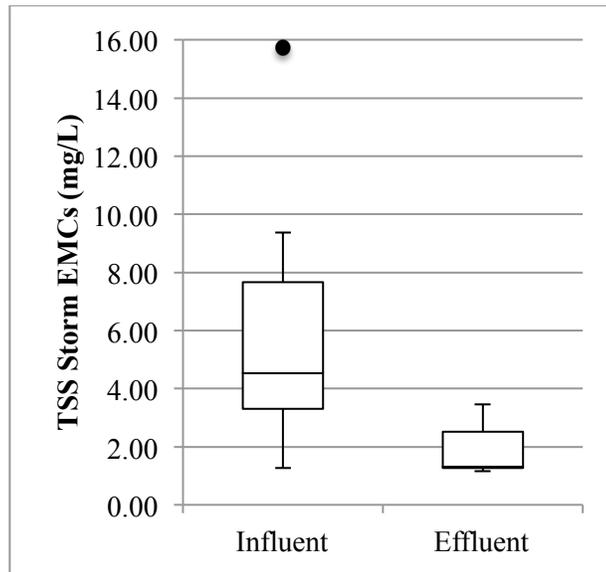


Figure 4.7: Influent and effluent TSS storm EMCs

Unlike biological and chemical nutrient removal processes, physical processes, such as particle settling, occur immediately upon entering the RWH system (DeBusk and Hunt, submitted), allowing for potential TSS mitigation, even in circumstances of short detention time. Rooftop TSS EMCs of 15 mg/L observed at commercial sites by Bannermann et al. (1993) verify that, by comparison, rooftop runoff from this site was clean. As in this study, DeBusk and Hunt (submitted) measured influent median TSS EMCs (3.48 mg/L) at four sites in Raleigh.

4.5 Conclusions

Although RWH systems are typically designed for runoff volume capture and use, and given credit for peak flow attenuation (NCDENR, 2009), some changes in pollutant concentrations were observed in this RWH system. Significant TAN addition (+67%) and NO_2+NO_3 reduction (-17%) was likely due to nitrogen transformation, notably DNRA. Settling of particles from rooftop runoff in the RWH tank yielded a TSS reduction of 53%, although phosphorus, typically bound to sediment, did not significantly change. The effluent stormwater EMCs measured herein were overall less polluted than similar influent data reported by Bannerman et al. (1993), DeBusk and Hunt (submitted) and Hathaway et al. (2008). The cistern was successful at mitigating stormwater runoff to at or below typical rainfall and runoff EMCs, reducing the impact of development on water quality.

Reasoning behind the pollutant performance may be attributed to the “overdesigning” of the RWH system. The catchment area for Underground Cistern 1 was 2100 m², while the

capacity of the cistern was 57.9 m³. This design allowed for an average rainfall of 28 mm to be captured by the cistern, a factor of 1.1x the design storm; three storms (18% of the monitoring period) exceeded 28 mm during the monitoring period. This added storage capacity could allow for sediment settling and storage, as well as a larger volume capture, mitigating a larger storm than designed.

4.6 References

Abbasi, T. and Abbasi, S.A. (2011). "Sources of pollution in rooftop rainwater harvesting systems and their control." *Critical Reviews in Environmental Science and Technology*, 41(23), 2097-2167.

DeBusk, K.M., Hunt, W.F. and Wright, J.D. (2013). "Characterization of rainwater harvesting performance in humid regions of the United States." *J.Am. Water Resour. Assoc.*, (accepted February 2013).

DeBusk, K.M. and Hunt, W.F. (submitted). "Impact of rainwater harvesting systems on nutrient and sediment concentrations in roof runoff." *Water Sci. Technol.*

Despins, C., Farahbakhsh, K. and Leidl, C. (2009). Assessment of rainwater quality from rainwater harvesting systems in Ontario, Canada. *J. Water Supply Res. T.* 58(2): 117-134.

Eaton, A.D., Clesceri, L.S. and Greenberg, A.R. (1995). Standard Methods for the Examination of Water and Wastewater, 19th Edition. American Public Health Association. Washington, D.C.

Hathaway, A.M., Hunt, W.F. and Jennings, G.D. (2008). "A field study of green roof hydrologic and water quality performance." *Transactions of the ASABE*, 51(1), 37-44.

Jones M.P. and Hunt, W.F. (2010). "Performance of rainwater harvesting systems in the Southeastern United States." *Resources Conservation and Recycling*, 54, 623-629.

Khastagir, A., and Jayasuriya, L.N.N. (2010). "Impacts of using rainwater tanks on stormwater harvesting and runoff quality." *Water Sci. Technol.*, 62(2), 324-329.

NCDENR (North Carolina Department of Environmental and Natural Resources). (2009). "Stormwater Best Management Practices Manual."

Perrin, C., Milburn, L., Szpir, L., Hunt, W.F., Bruce, S., McLendon, R., Job, S., Line, D., Lindbo, D., Smutko, S., Fisher, H., Tucker, R., Calabria, J., DeBusk, K., Cone, K.C., Smith-Gordon, M., Spooner, J., Blue, T., Deal, N., Lynn, J., Rashash, D., Rubin, R., Senior, M., White, N., Jones, D. and Eaker, W. (2009). "Low Impact Development: A Guidebook for North Carolina (AG-716)." NC Cooperative Extension Service, NC State University. website: <http://www.ncsu.edu/WECO/LID>

Prince George's County. (1999). "Low-Impact Development design Strategies. An Integrated Design Approach."

Reddy, Ramesh, K. and DeLaune, T.D. (2008). *Biogeochemistry of Wetlands: Science and Applications*. Taylor and Francis Group, LLC. Boca Raton, FL, USA.

State Climate Office of North Carolina (SCO). (2012). State Climate Office of North Carolina. North Carolina State University. website: <http://www.nc-climate.ncsu.edu>

U.S. Census Bureau. (2013). *State & county Quickfacts: Wake County, N.C.* website: <http://quickfacts.census.gov>

van Roon, M. (2007). "Water localisation and reclamation: Steps towards low impact urban design and development." *J. Environ. Manage.*, 83(4), 437-447.

Chapter 5: Summary of Studies, Recommendations and Further Research

5.1 Synthesis of Studies

As concluded in Chapter 2, low impact development (LID) has great potential to not only reduce runoff volumes and peak discharges to that of predevelopment conditions, but can even infiltrate more water than predevelopment for large events (e.g. 10-year, 24-hour). This commercial LID outperformed not only the conventional development, but also similar studies (Bedan and Clausen, 2009; Line et al., 2012; Rushton, 2001). Due to the detention and infiltration SCMs, peak flow mitigation greater than 98% was observed. Runoff coefficients computed at the commercial LID concluded that detention and infiltration are important stormwater management goals for meeting predevelopment conditions.

In terms of pollutant mitigation (Chapter 3), EMCs were generally not significantly different between the conventional development and the LID presumably due to observed event mean concentration (EMCs) at low influent concentrations and similar treatment mechanisms at each site. However, because the LID site reduced a large volume of runoff, the conventional development exported a factor of 23 – 85 times more pollutant loads than the LID. Despite concentration data not being conclusive to which development is better for water quality mitigation, the high runoff volume mitigation enabled the LID site to outperform all current peer-reviewed commercial LID studies in terms of pollutant loadings (Line et al., 2012; Rushton, 2001).

Chapter 4 discussed the performance of a rainwater harvesting (RWH) system and its influent and effluent EMCs. Influent and effluent total nitrogen (TN) and total Kjeldahl nitrogen (TKN) EMCs were not significantly different; however, reduction of nitrate – nitrite (NO_2+NO_3) and addition of total ammoniacal nitrogen (TAN) was observed. Denitrification, dissimilatory nitrate reduction to ammonium (DNRA), and conversion of organic nitrogen (ON) to TAN were possible nitrogen transformation processes; the lack of TN reduction excludes denitrification as the sole NO_2+NO_3 reduction mechanism, however. A similar trend was observed in DeBusk and Hunt (submitted). Since influent and effluent phosphorus species EMCs were below values reported by Hathway et al. (2008), reported practical quantification limits (PQLs), and values reported by DeBusk and Hunt (submitted), it was determined that influent phosphorus EMCs may not have been likely further reducible. As a result, total phosphorus (TP) and orthophosphorus (Ortho-P) concentrations were not significantly different between inflow and outflow. Low influent TSS concentrations were reduced, presumably due to settling.

Three commercial LID studies have thus far focused on (1) incorporating wetlands, bioretention, and permeable pavement (Line et al., 2012), (2) testing the incorporation of swales into various permeable and impermeable surfaces (Rushton, 2001), and (3) this study utilizing underground detention basins, infiltration trenches, and RWH with swales and bioretention. Though all three studies improved water quality and mitigated water quantity, more research is needed incorporating LID SCMs with large-scale detention. This study

proved that incorporating detention and infiltration into site design allowed for predevelopment hydrology, even in urban settings with 84% impervious land cover.

Reasoning behind the high runoff volume reduction at the LID site is likely due to the infiltration capacity of the underlying hydrologic soil group B soil, Appling. Additionally, high peak flow reductions at both the conventional development and the LID are likely a result of the large-scale detention facilities at each site. Because the LID site performed at a much higher peak flow reduction than the conventional development and previous studies (Bedan and Clausen, 2009; Line et al., 2012; Rushton, 2001), a question is raised as to whether or not the system was “overdesigned.” Between the cisterns, detention chamber, and infiltration gallery, approximately 1950 m³ of storage volume was capable of capturing 77 mm of runoff over the entire catchment area, rather than the 25.4 mm design storm as in Bedan and Clausen (2009) and Hood et al. (2007). Because of this extra storage volume, the system may capture and detain large events, such as the three storms more intense than the 10-year storm experienced during the monitoring period (Ch. 3).

5.2 Recommendations for Design and Evaluation

Leaking into the underground system appeared to cause the LID site to outflow more water than expected (though the amount of water was still nearly negligible). As a result of the underground “out of sight, out of mind” mentality, the leak was not easily detectable or maintainable. An issue was also discovered at the drip irrigation system. When a cistern is full, it has the potential to be used for supply, but has no stormwater mitigation potential

(DeBusk et al., 2013). During April – June 2012 when the irrigation system was not functioning properly, the cistern remained full and was not emptied. Frequent maintenance and inspection is necessary to ensure these systems are functioning as intended rather than remaining full for extended periods of time without water release. Had this site not been the focus of a research study, it is likely both issues would have gone unnoticed.

One tenant of LID is public education; however, this is difficult to attain when the majority of the LID was underground and not accessible to the public. The onsite grocery store did take advantage of the LID, posting informational graphics on how the system works, as well as numerical data on water use from the attached aboveground cistern. An idea to “bring the LID aboveground” would be to install a water level probe inside the detention basin and infiltration trenches, and create a live graphic showing the water level beneath the parking lot.

5.3 Further Research

Although monitoring water quality at the outlet of each site was sufficient to compare the performance of each site, monitoring inflow water quality at each site would allow for a more precise assessment. For storms measured at the inflow of both sites toward the end of the monitoring period, the LID and conventional development received differing pollutant concentrations, with the most extreme value for TSS being 4 times higher at the LID than at the conventional development. Because of this difference in inflow concentrations, it would

be better to directly measure inflow and outflow concentrations at each site, calculating actual nutrient reduction capabilities of each site.

Of the influent EMCs sampled, Chapter 3 reported variable performance in pollutant reduction and addition, likely due to the lack of vegetation-based SCMs, such as constructed stormwater wetlands, bioretention, and grassed swales. Because these particular SCMs are given local credit for pollutant removal (NCDENR, 2009), it would be interesting to determine the effect on effluent EMCs if these SCMs were a larger part of the LID design.

To further increase knowledge of the LID and how it functions, inflow and outflow of each SCM onsite would allow for comprehension of where water quality mitigation occurs in this “treatment train.” Because the LID encompasses grassed swales, bioretention, rainwater harvesting, and the infiltration gallery, there is much opportunity to learn about each component individually and how they function as a system, because they all have water quality mitigation potential.

Two reasons the LID performed so well were the underlying high-infiltration capacity soils and the “overdesign” of the system. Because underlying soil type was an integral component in the hydrologic performance of the LID site, an interesting future study could determine the performance of an LID built upon soils with less infiltration capacity, such as soil groups C or D. Similarly, modeling the LID based on field data and decreasing the detention volume of the SCMs would allow for the determination of optimal cost-benefit performance.

5.4 References

- Bedan, E.S. and Clausen, J.C. (2009). "Stormwater Runoff Quality and Quantity From Conventional and Low Impact Development Watersheds." *J.Am. Water Resour. Assoc.*, 45(4), 998-1008.
- DeBusk, K.M., Hunt, W.F. and Wright, J.D. (2013). "Characterization of rainwater harvesting performance in humid regions of the United States." *J.Am. Water Resour. Assoc.*, (accepted February 2013)
- Hathaway, A.M., Hunt, W.F. and Jennings, G.D. (2008). "A field study of green roof hydrologic and water quality performance." *Transactions of the ASABE*, 51(1), 37-44.
- Line, D.E., Brown, R. A., Hunt, W. F. and Lord, W. G. (2012). "Effectiveness of LID for Commercial Development in North Carolina" *J. Environ. Eng.*, 138(6), 680-688.
- Rushton, B.T. (2001). "Low-impact parking lot design reduces runoff and pollutant loads." *J. Water Resour. Plann. Manage.*, 127(3), 172-179.

Appendices

Appendix A: Raw Data

LID and Conventional Development Raw Data: Event Mean Concentrations

Storm Date	Rainfall (mm)	TKN (mg/L)		TAN (mg/L)		NO ₂ +NO ₃ (mg/L)		TN (mg/L)		TP (mg/L)		Ortho-P (mg/L)		TSS (mg/L)	
		Conv.	LID	Conv.	LID	Conv.	LID	Conv.	LID	Conv.	LID	Conv.	LID	Conv.	LID
1/12/12	15.24	0.82	0.67	0.17	0.22	0.32	0.25	1.13	0.91	0.09	0.05	0.01	0.00	15.36	3.56
1/22/12	16.51	0.48	0.00	0.08	0.00	0.17	0.00	0.65	0.00	0.05	0.00	0.01	0.00	8.50	0.01
1/27/12	9.40	1.02	0.56	0.25	0.21	0.35	0.30	1.37	0.87	0.22	0.05	0.01	0.02	62.97	10.87
2/6/12	11.43	0.56	0.85	0.13	0.26	0.34	0.58	0.90	1.43	0.04	0.05	0.01	0.01	3.31	10.47
2/17/12	8.64	0.82	0.69	0.38	0.34	0.23	0.29	1.05	0.98	0.05	0.04	0.02	0.02	2.95	7.62
2/20/12	19.05	0.31	0.48	0.13	0.19	0.11	0.23	0.42	0.71	0.03	0.03	0.01	0.00	8.76	6.18
4/23/12	30.73	0.70	0.51	0.10	0.10	0.13	0.23	0.83	0.74	0.08	0.06	0.00	0.02	33.47	10.45
5/10/12	21.08	0.52	1.12	0.10	0.18	0.12	0.53	0.65	1.65	0.08	0.10	0.02	0.01	13.27	23.00
5/15/12	12.95	0.56	0.59	0.10	0.12	0.17	0.46	0.72	1.05	0.06	0.27	0.01	0.02	14.04	11.69
6/12/12	14.48	0.73	1.08	0.12	0.11	0.20	0.62	0.93	1.70	0.06	0.07	0.01	0.01	3.08	9.18
6/13/12	13.97	0.47	0.49	0.09	0.08	0.12	0.31	0.59	0.80	0.05	0.04	0.01	0.01	11.40	6.83
7/12/12	19.05	0.45	0.74	0.08	0.02	0.35	0.45	0.80	1.19	0.04	0.06	0.00	0.00	8.46	10.93
8/8/12	39.88	0.49	0.58	0.04	0.05	0.21	0.36	0.70	0.94	0.10	0.04	0.04	0.00	7.98	8.38
8/9/12	36.32	0.45	0.46	0.06	0.06	0.15	0.18	0.61	0.65	0.14	0.07	0.04	0.02	35.60	10.03
8/21/12	39.62	0.26	0.44	0.07	0.07	0.13	0.32	0.39	0.76	0.07	0.09	0.04	0.00	9.94	8.52
9/8/12	20.57	0.77	1.33	0.04	0.73	0.20	0.16	0.98	1.49	0.09	0.04	0.00	0.00	21.65	5.63
10/30/12	12.95	0.21	0.21	0.03	0.03	0.09	0.18	0.31	0.39	0.07	0.02	0.05	0.01	1.90	3.29
11/14/12	4.83	0.00	1.08	0.00	0.42	0.00	0.25	0.00	1.33	0.00	0.09	0.00	0.03	0.01	10.22
11/16/12	8.64	0.23	0.41	0.04	0.13	0.11	0.18	0.34	0.58	0.11	0.05	0.08	0.02	4.23	8.97
12/18/12	6.86	2.01	0.52	1.31	0.16	0.60	0.35	2.61	0.87	0.51	0.04	0.45	0.00	6.16	12.56

RWH Raw Data: Event Mean Concentrations

Storm Date	Rainfall (mm)	TKN (mg/L)		TAN (mg/L)		NO ₂ +NO ₃ (mg/L)		TN (mg/L)		TP (mg/L)		Ortho-P (mg/L)		TSS (mg/L)	
		influent	effluent	influent	effluent	influent	effluent	influent	effluent	influent	effluent	influent	effluent	influent	effluent
1/12/12	15.24	0.42	0.47	0.25	0.26	0.28	0.31	0.70	0.78	0.02	0.02	0.0072	0.0030	4.47	1.69
1/22/12	16.51	0.44	0.46	0.33	0.27	0.27	0.35	0.71	0.81	0.01	0.01	0.0030	0.0030	3.30	1.29
1/27/12	9.40	0.62	0.92	0.26	0.34	0.27	0.25	0.89	1.17	0.03	0.04	0.0062	0.0092	9.37	3.45
2/6/12	11.43	0.70	0.61	0.54	0.43	0.66	0.52	1.36	1.14	0.03	0.02	0.0030	0.0030	5.72	1.25
2/17/12	8.64	1.50	0.73	0.43	0.34	0.41	0.36	1.91	1.09	0.05	0.02	0.0357	0.0030	7.70	1.27
2/20/12	19.05	0.38	0.57	0.25	0.29	0.30	0.34	0.67	0.91	0.02	0.02	0.0030	0.0030	3.19	2.54
7/10/12	14.48	0.43	1.23	0.15	0.68	0.42	0.09	0.84	1.32	0.01	0.04	0.0030	0.0063	4.52	2.53
8/9/12	36.32	0.25	0.19	0.11	0.11	0.26	0.16	0.52	0.35	0.02	0.005	0.0062	0.0030	4.62	1.27
8/21/12	39.62	0.59	0.34	0.34	0.21	0.32	0.20	0.91	0.55	0.005	0.005	0.0169	0.0030	1.28	1.28
9/6/12	32.51	0.17	0.26	0.18	0.22	0.18	0.14	0.35	0.39	0.005	0.005	0.0030	0.0030	1.27	1.16
9/8/12	20.57	0.16	0.33	0.15	0.25	0.16	0.17	0.33	0.50	0.005	0.005	0.0068	0.0030	4.08	1.28
9/17/12	12.70	0.63	0.55	0.46	0.41	0.87	0.47	1.49	1.03	0.01	0.02	0.0030	0.0030	1.26	1.31
10/30/12	12.95	0.19	0.28	0.08	0.12	0.46	0.23	0.65	0.51	0.01	0.01	0.0114	0.0030	3.43	1.79
11/14/12	4.83	0.60	0.78	0.19	0.37	0.41	0.30	1.01	1.08	0.04	0.04	0.0073	0.0030	15.71	1.93
11/16/12	8.64	0.17	1.00	0.06	0.15	0.22	0.24	0.39	1.24	0.02	0.13	0.0030	0.0699	7.66	2.55
12/13/12	9.14	0.27	1.19	0.14	0.66	0.42	0.45	0.70	1.64	0.02	0.08	0.0030	0.0196	6.49	1.28
12/18/12	6.86	0.38	0.81	0.18	0.37	0.38	0.28	0.77	1.09	0.02	0.06	0.0030	0.0079	8.33	2.96

Appendix B: Inflow Calculations

Volume Inflow: NRCS Curve Number Method (modified):

Step 1: Determine curve number for each land use and antecedent moisture condition (AMC)

Note: AMC for each storm is determined by the preceding 5-day rainfall (Table B.1)

Step 2: Determine storage (S , inches) for each land use and AMC

$$S = \frac{1000}{CN} - 10$$

Step 3: Determine initial abstraction (I_a , inches) for each land use and AMC

$$I_a = 0.2S$$

Step 4: Determine runoff depth (R , inches) from initial abstraction (I_a , inches) and rainfall depth (P , inches)

$$R = P - I_a$$

Step 5: Determine inflow runoff volume from runoff depth (R , inches) and watershed area (A , acres)

$$V = RA$$

Step 6: Convert volume from acre-inches to m^3 . Values are found on Table B.2.

Computed Volume Inflow Values

Table B.1: AMC Conditions

5-day preceding rainfall (inches)		
AMC	Dormant Season	Growing Season
I	<0.5	< 1.4
II	0.5 - 1.1	1.4 - 2.1
III	> 1.1	> 2.1

Table B.2: Inflow Volume Calculations

Date	AMC	Precipitation Depth (in)	Inflow Volume – Conventional (m ³)	Inflow Volume – LID (m ³)
1/11/12	II	0.60	265	303
1/17/12	II	0.24	84	107
1/20/12	II	0.65	300	331
1/27/12	II	0.37	140	176
2/4/12	II	0.45	174	219
2/16/12	II	0.34	127	160
2/19/12	II	0.75	370	388
3/16/12	I	0.22	39	49
3/18/12	I	0.46	141	178
3/19/12	II	0.44	168	212
3/20/12	III	0.45	251	238
3/20/12	III	0.55	324	297
3/23/12	III	0.77	477	436
3/30/12	I	0.21	34	43
4/4/12	I	0.54	175	221
4/5/12	II	0.24	85	108
4/21/12	I	1.21	469	581
4/25/12	I	0.39	112	141
5/9/12	II	0.83	426	434
5/13/12	I	0.24	48	61
5/14/12	I	0.51	162	205
5/27/12	I	0.18	24	30
5/30/12	I	0.58	190	240
6/1/12	I	1.04	385	486
6/11/12	I	0.57	188	237
6/12/12	I	0.55	179	226
6/21/12	I	0.54	175	221
7/1/12	I	0.22	39	49
7/4/12	I	1.45	636	717

7/10/12	I	0.75	264	334
7/20/12	I	0.40	117	147
7/21/12	I	3.13	1814	1682
7/28/12	I	1.18	451	567
8/6/12	I	1.57	721	786
8/8/12	II	1.43	847	782
8/10/12	III	0.76	469	429
8/11/12	III	0.37	196	193
8/19/12	I	1.56	714	780
9/16/12	I	0.50	158	200
9/29/12	I	0.84	303	383
10/7/12	I	0.38	106	134
10/8/12	I	0.47	143	181
10/28/12	I	0.32	82	103
10/29/12	I	0.52	166	210
11/13/12	I	0.21	35	44
11/15/12	I	0.36	99	125
12/18/12	I	0.27	60	76

Peak Inflow

Rational Method

Step 1: Determine runoff coefficients (C) for each land use

Step 2: Determine peak inflow (Q_p , m^3/s) from runoff coefficient (C), rainfall intensity (i , inches/hour), and watershed area (acres)

$$Q_p = \frac{CiA}{360}$$

Step 3: Convert Q_p from m^3/s to L/s. Values are found on Table B.3.

Computed Peak Inflow Values

Table B.3: Peak Inflow Calculations

Date	Precipitation Depth (in)	Rainfall Intensity (in/hour)	Peak Inflow – Conventional (L/s)	Peak Inflow – LID (L/s)
3/16/12	0.22	0.69	93.16	101.69
3/18/12	0.46	0.69	93.16	101.69
3/19/12	0.44	2.99	403.67	440.65
3/20/12	0.45	1.61	217.36	237.27
3/20/12	0.55	1.15	155.26	169.48
3/23/12	0.77	0.92	124.21	135.59
3/30/12	0.21	0.23	31.05	33.90
4/4/12	0.54	1.61	217.36	237.27
4/5/12	0.24	0.23	31.05	33.90
4/21/12	1.21	1.81	244.28	266.65
4/25/12	0.39	1.61	217.36	237.27
5/9/12	0.83	5.14	694.43	758.05
5/13/12	0.24	0.23	31.05	33.90
5/14/12	0.51	1.43	192.79	210.45
5/27/12	0.18	2.76	372.62	406.76
5/30/12	0.58	0.46	62.10	67.79
6/1/12	1.04	4.83	652.09	711.82
6/11/12	0.57	1.40	188.46	205.72
6/12/12	0.55	2.75	371.27	405.28
6/21/12	0.54	1.84	248.41	271.17
7/1/12	0.22	2.53	341.57	372.86
7/4/12	1.45	8.51	1148.92	1254.16
7/10/12	0.75	2.50	337.52	368.44
7/20/12	0.40	3.68	496.83	542.34
7/21/12	3.13	8.05	1086.81	1186.37
7/28/12	1.18	7.59	1024.71	1118.58
8/6/12	1.57	5.23	706.54	771.26
8/8/12	1.43	6.62	893.51	975.36
8/10/12	0.76	4.19	565.08	616.84
8/11/12	0.37	1.90	256.90	280.43
8/19/12	1.56	5.02	678.24	740.37
9/16/12	0.50	1.56	210.01	229.25
9/29/12	0.84	1.33	179.63	196.08
10/7/12	0.38	2.44	329.32	359.49
10/8/12	0.47	2.88	389.20	424.85
10/28/12	0.32	0.24	32.00	34.93
10/29/12	0.52	0.41	55.06	60.11
11/13/12	0.21	0.44	59.69	65.16
11/15/12	0.36	0.42	57.18	62.42
12/18/12	0.27	0.27	36.45	39.79

Appendix C: Pollutant Loading Calculations

Step 1: For each storm event, calculate the mass of pollutant exported from the EMC (mg/L) and outflow volume (V ; m^3). Convert mass to kg.

$$Mass = EMC * V$$

Step 2: Sum pollutant masses over the monitoring period.

Step 3: Sum rainfall depth monitored over the monitoring period. Determine amount of rainfall monitored of the year.

$$Loading (kg/year) = \Sigma(Mass) * \frac{Rainfall Monitored}{Annual Rainfall}$$

Step 4: Normalize the pollutant loads by watershed area. Loading Data are found in Table C.1 and Table C.2.

$$Loading(kg/ha/year) = \frac{Loading(kg/year)}{Watershed\ area\ (ha)}$$

Table C.1: Conventional Development Pollutant Loadings

Date	Rainfall Depth (in)	TKN (µg/L)	TKN (kg)	TAN (µg/L)	TAN (kg)	NO ₂ +NO ₃ (µg/L)	NO ₂ +NO ₃ (kg)	TP (µg/L)	TP (kg)	Ortho-P (µg/L)	Ortho-P (kg)	TSS (mg/L)	TSS (kg)	Volume (ft ³)
1/12/12	0.60	815.14	0.1039	172.52	0.0220	315.84	0.0402	87.87	0.0112	12.01	0.0015	15.36	1.96	4499
1/22/12	0.65	475.99	0.0659	81.84	0.0113	170.19	0.0236	50.77	0.0070	8.19	0.0011	8.50	1.18	4887
1/27/12	0.37	1018.89	0.0829	253.42	0.0206	350.18	0.0285	224.64	0.0183	12.49	0.0010	62.97	5.12	2872
2/6/12	0.45	561.68	0.0307	128.84	0.0070	338.12	0.0185	42.91	0.0023	7.38	0.0004	3.31	0.18	1930
2/17/12	0.34	824.48	0.0320	377.16	0.0146	229.84	0.0089	53.40	0.0021	17.42	0.0007	2.95	0.11	1369
2/20/12	0.75	313.89	0.0316	125.46	0.0126	108.34	0.0109	32.62	0.0033	9.19	0.0009	8.76	0.88	3558
4/23/12	1.21	703.39	0.1388	103.48	0.0204	129.12	0.0255	78.32	0.0155	3.00	0.0006	33.47	6.60	6967
5/10/12	0.83	524.27	0.1004	98.31	0.0188	122.76	0.0235	78.51	0.0150	19.68	0.0038	13.27	2.54	6765
5/15/12	0.51	558.74	0.0580	96.93	0.0101	165.92	0.0172	62.43	0.0065	12.85	0.0013	14.04	1.46	3668
6/12/12	0.57	732.06	0.0756	115.10	0.0119	195.48	0.0202	61.80	0.0064	10.49	0.0011	3.08	0.32	3645
6/13/12	0.55	465.74	0.0621	86.08	0.0115	123.45	0.0165	45.82	0.0061	7.38	0.0010	11.40	1.52	4711
7/12/12	0.75	448.41	0.0628	84.50	0.0118	348.00	0.0487	36.59	0.0051	3.00	0.0004	8.46	1.18	4945
8/8/12	1.57	492.78	0.2764	41.96	0.0235	208.36	0.1169	99.37	0.0557	44.44	0.0249	7.98	4.48	19809
8/9/12	1.43	454.98	0.1413	60.29	0.0187	151.65	0.0471	143.63	0.0446	43.58	0.0135	35.60	11.05	10965
8/21/12	1.56	259.02	0.1603	65.24	0.0404	130.70	0.0809	66.82	0.0414	35.90	0.0222	9.94	6.15	21855
9/8/12	0.81	772.62	1.4748	38.29	0.0731	202.64	0.3868	90.36	0.1725	3.00	0.0057	21.65	41.33	67410
10/30/12	0.51	210.89	0.0669	26.57	0.0084	94.30	0.0299	68.78	0.0218	52.19	0.0166	1.90	0.60	11202
11/14/12	0.19	0	0.0000	0	0.0000	0	0.0000	0	0.0000	0	0.0000	0	0.00	0
11/16/12	0.34	229.54	0.0521	44.43	0.0101	108.03	0.0245	107.07	0.0243	81.63	0.0185	4.23	0.96	8021
12/18/12	0.27	2013.88	0.1200	1306.45	0.0778	600.06	0.0357	506.50	0.0302	448.24	0.0267	6.16	0.37	2104
14.26	kg		3.14		0.42		1.00		0.49		0.14		88.00	
33.16%	kg/year		9.46		1.28		3.03		1.48		0.43		265.34	
	kg/ha/yr		3.43		0.46		1.10		0.53		0.16		96.14	

Table C.2: LID Pollutant Loadings

Date	Rainfall Depth (in)	TKN (µg/L)	TKN (kg)	TAN (µg/L)	TAN (kg)	NO ₂ +NO ₃ (µg/L)	NO ₂ +NO ₃ (kg)	TP (µg/L)	TP (kg)	Ortho-P (µg/L)	Ortho-P (kg)	TSS (mg/L)	TSS (kg)	Volume (ft ³)
1/12/12	0.60	665.00	0.0033	217.06	0.0011	246.89	0.0012	52.93	0.0003	3.00	0.00002	3.56	0.018	178
1/22/12	0.65	0.01	0.0000	0.01	0.0000	0.01	0.0000	0.01	0.0000	0.01	0.0000	0.01	0.0000	1
1/27/12	0.37	564.73	0.0021	207.92	0.0008	303.88	0.0011	50.98	0.0002	17.09	0.00006	10.87	0.040	130
2/6/12	0.45	852.06	0.0068	260.65	0.0021	582.58	0.0047	53.97	0.0004	14.35	0.00011	10.47	0.084	282
2/17/12	0.34	688.97	0.0021	341.66	0.0010	292.74	0.0009	44.43	0.0001	16.39	0.00005	7.62	0.023	108
2/20/12	0.75	477.73	0.0046	187.31	0.0018	233.94	0.0023	30.43	0.0003	3.00	0.00003	6.18	0.060	340
4/23/12	1.21	511.84	0.0070	97.06	0.0013	225.86	0.0031	62.50	0.0009	17.17	0.00023	10.45	0.142	480
5/10/12	0.83	1124.61	0.0033	176.59	0.0005	528.70	0.0016	104.03	0.0003	10.69	0.00003	23.00	0.068	105
5/15/12	0.51	586.77	0.0006	116.52	0.0001	458.62	0.0005	272.23	0.0003	19.55	0.00002	11.69	0.012	36
6/12/12	0.57	1075.26	0.0038	105.77	0.0004	621.90	0.0022	66.85	0.0002	7.70	0.00003	9.18	0.032	125
6/13/12	0.55	490.71	0.0009	81.90	0.0002	311.64	0.0006	43.11	0.0001	10.55	0.00002	6.83	0.013	67
7/12/12	0.75	737.51	0.0030	22.18	0.0001	451.38	0.0018	57.80	0.0002	3.00	0.00001	10.93	0.044	142
8/8/12	1.57	583.92	0.0063	53.40	0.0006	355.96	0.0038	42.39	0.0005	3.00	0.00003	8.38	0.090	381
8/9/12	1.43	461.90	0.0166	56.08	0.0020	183.48	0.0066	68.01	0.0024	16.42	0.00059	10.03	0.360	1267
8/21/12	1.56	444.95	0.0078	66.36	0.0012	315.74	0.0056	94.89	0.0017	3.00	0.00005	8.52	0.150	621
9/8/12	0.81	1334.25	0.0007	733.34	0.0004	157.68	0.0001	35.69	0.0000	3.00	0.00000	5.63	0.003	19
10/30/12	0.51	213.04	0.0020	29.75	0.0003	181.02	0.0017	19.22	0.0002	6.90	0.00006	3.29	0.030	325
11/14/12	0.19	1080.15	0.0025	422.18	0.0010	251.76	0.0006	93.35	0.0002	33.96	0.00008	10.22	0.024	83
11/16/12	0.34	405.60	0.002	128.19	0.0007	179.23	0.0009	49.99	0.0003	17.43	0.00009	8.97	0.046	183
12/18/12	0.27	524.90	0.002	163.03	0.0006	348.74	0.0013	36.82	0.0001	3.00	0.00001	12.56	0.046	128
14.26	kg		0.08		0.02		0.04		0.01		0.002		1.29	
33.16%	kg/year		0.23		0.05		0.12		0.03		0.005		3.88	
kg/ha/yr			0.09		0.02		0.05		0.01		0.002		1.53	

Appendix D: SAS Code

Runoff Volume

```
proc import out= work.volume
datafile= "\\tsclient\Mac HD\Users\Corinne\Desktop\Thesis\Data\SAS\data.xls"
  DBMS=EXCEL REPLACE;
  SHEET="V";
  RANGE="A1:V48";
  GETNAMES=YES;
run;

data volume;
  set volume;
  logtradin=log(tradin);
  logtradindcia=log(tradindcia);
  logtradout = log(tradout);
  logtradoutdcia=log(tradoutdcia);
  loglidin=log(lidin);
  loglidindcia=log(lidindcia);
  loglidout = log(lidout);
  loglidoutdcia=log(lidoutdcia);
  logtradrcoeff=log(tradrcoeff);
  loglidrcoeff=log(lidrcoeff);
  logindiff = logtradin - loglidin;
  loginciadiff = logtradindcia - loglidindcia;
  logoutdiff = logtradout - loglidout;
  logoutdciadiff = logtradoutdcia - loglidoutdcia;
  logreddiff = logtradred - loglidred;
  logcoeffdiff = logtradrcoeff - loglidrcoeff; run;

proc univariate data=volume normal;
  var indiff inciadiff outdciadiff reddiff coeffdiff
  logindiff loginciadiff logoutdciadiff logreddiff logcoeffdiff;
run;

proc reg data=volume;
  model logtradout = logtradin;
run;

proc glm data=volume;
  class season;
  model logtradout = logtradin|season;
  lsmeans season/pdiff;
run;

proc glm data=volume;
  class season;
  model logtradout = logtradin season;
  lsmeans season/pdiff;
run;

proc reg data=volume;
```

```

        model loglidout = loglidin;                run;

proc glm data=volume;
  class season;
  model loglidout = loglidin|season;
  lsmeans season/pdiff;                          run;

proc glm data=volume;
  class season;
  model loglidout = loglidin season;
  lsmeans season/pdiff;                          run;

proc reg data=volume;
  model logtradoutdcia = logtradindcia;          run;

proc glm data=volume;
  class season;
  model logtradoutdcia = logtradindcia|season;
  lsmeans season/pdiff;                          run;

proc glm data=volume;
  class season;
  model logtradoutdcia = logtradindcia season;
  lsmeans season/pdiff;                          run;

proc reg data=volume;
  model loglidoutdcia = loglidindcia;            run;

proc glm data=volume;
  class season;
  model loglidoutdcia = loglidindcia|season;
  lsmeans season/pdiff;                          run;

proc glm data=volume;
  class season;
  model loglidoutdcia = loglidindcia season;
  lsmeans season/pdiff;                          run;

proc reg data=volume;
  model loglidout = logtradout;                  run;

proc glm data=volume;
  class season;
  model loglidout = logtradout|season;
  lsmeans season/pdiff;                          run;

proc glm data=volume;
  class season;
  model loglidout = logtradout season;
  lsmeans season/pdiff;                          run;

```

```

proc reg data=volume;
    model loglidin = logtradin;          run;

proc glm data=volume;
    class season;
    model loglidin = logtradin|season;
    lsmeans season/pdiff;              run;

proc glm data=volume;
    class season;
    model loglidin = logtradin season;
    lsmeans season/pdiff;              run;

proc reg data=volume;
    model loglidoutdcia = logtradoutdcia;  run;

proc glm data=volume;
    class season;
    model loglidoutdcia = logtradoutdcia|season;
    lsmeans season/pdiff;              run;

proc glm data=volume;
    class season;
    model loglidoutdcia = logtradoutdcia season;
    lsmeans season/pdiff;              run;

proc reg data=volume;
    model loglidindcia = logtradindcia;     run;

proc glm data=volume;
    class season;
    model loglidindcia = logtradindcia|season;
    lsmeans season/pdiff;              run;

proc glm data=volume;
    class season;
    model loglidindcia = logtradindcia season;
    lsmeans season/pdiff;              run;

proc reg data=volume;
    model loglidrcoeff = logtradrcoeff;     run;

proc glm data=volume;
    class season;
    model loglidrcoeff = logtradrcoeff|season;
    lsmeans season/pdiff;              run;

proc glm data=volume;
    class season;
    model loglidrcoeff = logtradrcoeff season;
    lsmeans season/pdiff;              run;

```

```

proc glm data=volume;
  class amc;
  model logtradout = logtradin|amc;
  lsmeans amc/pdiff;
run;

proc glm data=volume;
  class amc;
  model logtradout = logtradin amc;
  lsmeans amc/pdiff;
run;

proc reg data=volume;
  model loglidout = loglidin;
run;

proc glm data=volume;
  class amc;
  model loglidout = loglidin|amc;
  lsmeans amc/pdiff;
run;

proc glm data=volume;
  class amc;
  model loglidout = loglidin amc;
  lsmeans amc/pdiff;
run;

proc reg data=volume;
  model logtradoutdcia = logtradindcia;
run;

proc glm data=volume;
  class amc;
  model logtradoutdcia = logtradindcia|amc;
  lsmeans amc/pdiff;
run;

proc glm data=volume;
  class amc;
  model logtradoutdcia = logtradindcia amc;
  lsmeans amc/pdiff;
run;

proc reg data=volume;
  model loglidoutdcia = loglidindcia;
run;

proc glm data=volume;
  class amc;
  model loglidoutdcia = loglidindcia|amc;
  lsmeans amc/pdiff;
run;

proc glm data=volume;
  class season;
  model loglidoutdcia = loglidindcia amc;
  lsmeans amc/pdiff;
run;

proc reg data=volume;

```

```

        model loglidout = logtradout;          run;

proc glm data=volume;
  class amc;
  model loglidout = logtradout|amc;
  lsmeans amc/pdiff;                        run;

proc glm data=volume;
  class amc;
  model loglidout = logtradout amc;
  lsmeans amc/pdiff;                        run;

proc reg data=volume;
  model loglidin = logtradin;                run;

proc glm data=volume;
  class amc;
  model loglidin = logtradin|amc;
  lsmeans amc/pdiff;                        run;

proc glm data=volume;
  class amc;
  model loglidin = logtradin amc;
  lsmeans amc/pdiff;                        run;

proc reg data=volume;
  model loglidoutdcia = logtradoutdcia;      run;

proc glm data=volume;
  class amc;
  model loglidoutdcia = logtradoutdcia|amc;
  lsmeans amc/pdiff;                        run;

proc glm data=volume;
  class amc;
  model loglidoutdcia = logtradoutdcia amc;
  lsmeans amc/pdiff;                        run;

proc reg data=volume;
  model loglidindcia = logtradinindcia;      run;

proc glm data=volume;
  class amc;
  model loglidindcia = logtradinindcia|amc;
  lsmeans amc/pdiff;                        run;

proc glm data=volume;
  class amc;
  model loglidindcia = logtradinindcia amc;
  lsmeans amc/pdiff;                        run;

```

```
proc reg data=volume;
    model loglidrcoeff = logtradrcoeff;          run;

proc glm data=volume;
    class amc;
    model loglidrcoeff = logtradrcoeff|amc;
    lsmeans amc/pdiff;                          run;

proc glm data=volume;
    class amc;
    model loglidrcoeff = logtradrcoeff amc;
    lsmeans amc/pdiff;                          run;
```

Peak Discharge

```
proc import out= work.peak
datafile= "\\tsclient\Mac HD\Users\Corinne\Desktop\Thesis\Data\SAS\data.xls"
    DBMS=EXCEL REPLACE;
    SHEET="Qp";
    RANGE="A1:Z41";
    GETNAMES=YES;
run;

data peak;
    set peak;
    logtradin=log(tradin);
    logtradout=log(tradout);
    logtradindcia=log(tradindcia);
    logtradoutdcia=log(tradoutdcia);
    loglidin=log(lidin);
    loglidout=log(lidout);
    loglidindcia=log(lidindcia);
    loglidoutdcia=log(lidoutdcia);
    logtradred=log(tradred);
    loglidred=log(lidred);
    logindiff = logtradin - loglidin;
    logoutdiff = logtradout - loglidout;
    logoutdciaiff = logtradoutdcia - loglidoutdcia;
    logreddiff = lodtradred - loglidred;
run;

proc univariate data=peak normal;
    var i indiff outdciaiff reddiff logindiff logoutdiff logreddiff;
run;

proc glm data=peak;
    class season;
    model i = season;
    lsmeans season/pdiff;
run;

proc reg data=peak;
    model logtradout = logtradin;
run;

proc glm data=peak;
    class season;
    model logtradout = logtradin|season;
    lsmeans season/pdiff;
run;

proc glm data=peak;
    class season;
    model logtradout = logtradin season;
    lsmeans season/pdiff;
run;

proc reg data=peak;
    model loglidout = loglidin;
run;
```

```

proc glm data=peak;
  class season;
  model loglidout = loglidin|season;
  lsmeans season/pdiff;
run;

proc glm data=peak;
  class season;
  model loglidout = loglidin season;
  lsmeans season/pdiff;
run;

proc reg data=peak;
  model loglidout = logtradout;
run;

proc glm data=peak;
  class season;
  model loglidout = logtradout|season;
  lsmeans season/pdiff;
run;

proc glm data=peak;
  class season;
  model loglidout = logtradout season;
  lsmeans season/pdiff;
run;

proc reg data=peak;
  model loglidin = logtradin;
run;

proc glm data=peak;
  class season;
  model loglidin = logtradin|season;
  lsmeans season/pdiff;
run;

proc glm data=peak;
  class season;
  model loglidin = logtradin season;
  lsmeans season/pdiff;
run;

proc reg data=peak;
  model loglidoutdcia = logtradoutdcia;
run;

proc glm data=peak;
  class season;
  model loglidoutdcia = logtradoutdcia|season;
  lsmeans season/pdiff;
run;

proc glm data=peak;
  class season;
  model loglidoutdcia = logtradoutdcia season;
  lsmeans season/pdiff;
run;

proc reg data=peak;
  model loglidindcia = logtradindcia;
run;

```

```

proc glm data=peak;
  class season;
  model loglidindcia = logtradindcia|season;
  lsmeans season/pdiff;
run;

proc glm data=peak;
  class season;
  model loglidindcia = logtradindcia season;
  lsmeans season/pdiff;
run;

proc reg data=peak;
  model loglidred = logtradred;
run;

proc glm data=peak;
  class season;
  model loglidred = logtradred|season;
  lsmeans season/pdiff;
run;

proc glm data=peak;
  class season;
  model loglidred = logtradred season;
  lsmeans season/pdiff;
run;

```

Water Quality

```
proc import out= work.wqual
  datafile= "\\tsclient\Mac HD\Users\Corinne\Desktop\Thesis\Data\SAS\data.xls"
  DBMS=EXCEL REPLACE;
  SHEET="WQ";
  RANGE="A1:AE21";
  GETNAMES=YES;
run;

data wqual;
  set wqual;
  tknconcdiff=tradtknconc-lidtknconc;
  nh3nconcdiff=tradnh3nconc-lidnh3nconc;
  no23concdiff=tradno23conc-lidno23conc;
  opconcdiff=tradopconc-lidopconc;
  tssconcdiff=tradtssconc-lidtssconc;
  tpconcdiff=tradtpconc-lidtpconc;
  tnconcdiff=tradtnconc-lidtnconc;
  tknloaddiff=tradtknload-lidtknload;
  nh3nloaddiff=tradnh3nload-lidnh3nload;
  no23loaddiff=tradno23load-lidno23load;
  oploaddiff=tradopload-lidopload;
  tssloaddiff=tradtssload-lidtssload;
  tploaddiff=tradtpload-lidtpload;
  tnloaddiff=tradtnload-lidtnload;
  logtradtknconc=log(tradtknconc);
  logtradtknload=log(tradtknload);
  logtradnh3nconc=log(tradnh3nconc);
  logtradnh3nload=log(tradnh3nload);
  logtradno23conc=log(tradno23conc);
  logtradno23load=log(tradno23load);
  logtradopconc=log(tradopconc);
  logtradopload=log(tradopload);
  logtradtssconc=log(tradtssconc);
  logtradtssload=log(tradtssload);
  logtradtpconc=log(tradtpconc);
  logtradtpload=log(tradtpload);
  logtradtnconc=log(tradtnconc);
  logtradtnload=log(tradtnload);
  loglidtknconc=log(lidtknconc);
  loglidtknload=log(lidtknload);
  loglidnh3nconc=log(lidnh3nconc);
  loglidnh3nload=log(lidnh3nload);
  loglidno23conc=log(lidno23conc);
  loglidno23load=log(lidno23load);
  loglidopconc=log(lidopconc);
  loglidopload=log(lidopload);
  loglidtssconc=log(lidtssconc);
  loglidtssload=log(lidtssload);
  loglidtpconc=log(lidtpconc);
```

```

loglidtpload=log(lidtpload);
loglidtnconc=log(lidtnconc);
loglidtnload=log(lidtnload);
logtknconcdiff=logtradtknconc-loglidtknconc;
lognh3nconcdiff=logtradnh3nconc-loglidnh3nconc;
logno23concdiff=logtradno23conc-loglidno23conc;
logopconcdiff=logtradopconc-loglidopconc;
logtssconcdiff=logtradtsconc-loglidtsconc;
logtpconcdiff=logtradtpconc-loglidtpconc;
logtnconcdiff=logtradtnconc-loglidtnconc;
logtknloaddiff=logtradtknload-loglidtknload;
lognh3nloaddiff=logtradnh3nload-loglidnh3nload;
logno23loaddiff=logtradno23load-loglidno23load;
logoploaddiff=logtradopload-loglidopload;
logtssloaddiff=logtradtsload-loglidtsload;
logtploaddiff=logtradtpload-loglidtpload;
logtnloaddiff=logtradtnload-loglidtnload;    run;

proc univariate data=wqual normal;
    var tknconcdiff nh3nconcdiff no23concdiff opconcdiff tssconcdiff tpconcdiff tnconcdiff
    tknloaddiff nh3nloaddiff no23loaddiff oploaddiff tssloaddiff tploaddiff tnloaddiff
    logtknconcdiff lognh3nconcdiff logopconcdiff logtssconcdiff logtpconcdiff
    logtknloaddiff lognh3nloaddiff logno23loaddiff logoploaddiff
    logtssloaddiff logtploaddiff logtnloaddiff;    run;

proc reg data=wqual;
    model lidtknconc=tradtknconc;    run;

proc glm data=wqual;
    class season;
    model lidtknconc=tradtknconc|season;
    lsmeans season /pdiff;    run;

proc glm data=wqual;
    class season;
    model lidtknconc=tradtknconc season;
    lsmeans season /pdiff;    run;

proc reg data=wqual;
    model loglidnh3nconc=logtradnh3nconc;    run;

proc glm data=wqual;
    class season;
    model loglidnh3nconc=logtradnh3nconc|season;
    lsmeans season /pdiff;    run;

proc glm data=wqual;
    class season;
    model loglidnh3nconc=logtradnh3nconc season;
    lsmeans season /pdiff;    run;

```

```

proc reg data=wqual;
    model lidno23conc=tradno23conc;          run;

proc glm data=wqual;
    class season;
    model lidno23conc=tradno23conc|season;
    lsmeans season /pdiff;                  run;

proc glm data=wqual;
    class season;
    model lidno23conc=tradno23conc season;
    lsmeans season /pdiff;                  run;

proc reg data=wqual;
    model loglidopconc=logtradopconc;       run;

proc glm data=wqual;
    class season;
    model loglidopconc=logtradopconc|season;
    lsmeans season /pdiff;                  run;

proc glm data=wqual;
    class season;
    model loglidopconc=logtradopconc season;
    lsmeans season /pdiff;                  run;

proc reg data=wqual;
    model lidtssconc=tradtssconc;           run;

proc glm data=wqual;
    class season;
    model lidtssconc=tradtssconc|season;
    lsmeans season /pdiff;                  run;

proc glm data=wqual;
    class season;
    model lidtssconc=tradtssconc season;
    lsmeans season /pdiff;                  run;

proc reg data=wqual;
    model loglidtpconc=logtradtpconc;       run;

proc glm data=wqual;
    class season;
    model loglidtpconc=logtradtpconc|season;
    lsmeans season /pdiff;                  run;

proc glm data=wqual;
    class season;
    model loglidtpconc=logtradtpconc season;
    lsmeans season /pdiff;                  run;

```

```

proc reg data=wqual;
    model lidtnconc=tradtnconc;          run;

proc glm data=wqual;
    class season;
    model lidtnconc=tradtnconc|season;
    lsmeans season /pdiff;
run;

proc glm data=wqual;
    class season;
    model lidtnconc=tradtnconc season;
    lsmeans season /pdiff;          run;

proc reg data=wqual;
    model loglidtknload=logtradtknload;  run;

proc glm data=wqual;
    class season;
    model loglidtknload=logtradtknload|season;
    lsmeans season /pdiff;          run;

proc glm data=wqual;
    class season;
    model loglidtknload=logtradtknload season;
    lsmeans season /pdiff;          run;

proc reg data=wqual;
    model loglidnh3nload=logtradnh3nload;  run;

proc glm data=wqual;
    class season;
    model loglidnh3nload=logtradnh3nload|season;
    lsmeans season /pdiff;          run;

proc glm data=wqual;
    class season;
    model loglidnh3nload=logtradnh3nload season;
    lsmeans season /pdiff;          run;

proc reg data=wqual;
    model loglidno23load=logtradno23load;  run;

proc glm data=wqual;
    class season;
    model loglidno23load=logtradno23load|season;
    lsmeans season /pdiff;          run;

proc glm data=wqual;
    class season;

```

```

        model loglidno23load=logtradno23load season;
        lsmeans season /pdiff;          run;

proc reg data=wqual;
    model loglidopload=logtradopload;    run;

proc glm data=wqual;
    class season;
    model loglidopload=logtradopload|season;
    lsmeans season /pdiff;          run;

proc glm data=wqual;
    class season;
    model loglidopload=logtradopload season;
    lsmeans season /pdiff;          run;

proc reg data=wqual;
    model loglidtssload=logtradtssload;    run;

proc glm data=wqual;
    class season;
    model loglidtssload=logtradtssload|season;
    lsmeans season /pdiff;          run;

proc glm data=wqual;
    class season;
    model loglidtssload=logtradtssload season;
    lsmeans season /pdiff;          run;

proc reg data=wqual;
    model loglidtpload=logtradtpload;    run;

proc glm data=wqual;
    class season;
    model loglidtpload=logtradtpload|season;
    lsmeans season /pdiff;          run;

proc glm data=wqual;
    class season;
    model loglidtpload=logtradtpload season;
    lsmeans season /pdiff;          run;

proc reg data=wqual;
    model loglidtnload=logtradtnload;    run;

proc glm data=wqual;
    class season;
    model loglidtnload=logtradtnload|season;
    lsmeans season /pdiff;          run;

proc glm data=wqual;

```

```
class season;  
model loglidtnload=logtradtnload season;  
lsmeans season /pdiff;          run;
```

RWH Water Quality

```
proc import out= work.volume
datafile= "\\tsclient\Mac HD\Users\Corinne\Desktop\Thesis\Data\SAS\data.xls"
  DBMS=EXCEL REPLACE;
  SHEET="V";
  RANGE="A1:V48";
  GETNAMES=YES;
run;

data volume;
  set volume;
  logtradin=log(tradin);
  logtradindcia=log(tradindcia);
  logtradout = log(tradout);
  logtradoutdcia=log(tradoutdcia);
  loglidin=log(lidin);
  loglidindcia=log(lidindcia);
  loglidout = log(lidout);
  loglidoutdcia=log(lidoutdcia);
  logtradrcoeff=log(tradrcoeff);
  loglidrcoeff=log(lidrcoeff);
  logindiff = logtradin - loglidin;
  logindciadiff = logtradindcia - loglidindcia;
  logoutdiff = logtradout - loglidout;
  logoutdciadiff = logtradoutdcia - loglidoutdcia;
  logreddiff = logtradred - loglidred;
  logcoeffdiff = logtradrcoeff - loglidrcoeff;
run;

proc univariate data=volume normal;
  var indiff indciadiff outdciadiff reddiff coeffdiff
  logindiff logindciadiff logoutdciadiff logreddiff logcoeffdiff;
run;

proc reg data=volume;
  model logtradout = logtradin;
run;

proc glm data=volume;
  class season;
  model logtradout = logtradin|season;
  lsmeans season/pdiff;
run;

proc glm data=volume;
  class season;
  model logtradout = logtradin season;
  lsmeans season/pdiff;
run;

proc reg data=volume;
  model loglidout = loglidin;
run;

proc glm data=volume;
  class season;
```

```

        model loglidout = loglidin|season;
        lsmeans season/pdiff;                                run;

proc glm data=volume;
    class season;
    model loglidout = loglidin season;
    lsmeans season/pdiff;                                run;

proc reg data=volume;
    model logtradoutdcia = logtradindcia;                run;

proc glm data=volume;
    class season;
    model logtradoutdcia = logtradindcia|season;
    lsmeans season/pdiff;                                run;

proc glm data=volume;
    class season;
    model logtradoutdcia = logtradindcia season;
    lsmeans season/pdiff;                                run;

proc reg data=volume;
    model loglidoutdcia = loglidindcia;                run;

proc glm data=volume;
    class season;
    model loglidoutdcia = loglidindcia|season;
    lsmeans season/pdiff;                                run;

proc glm data=volume;
    class season;
    model loglidoutdcia = loglidindcia season;
    lsmeans season/pdiff;                                run;

proc reg data=volume;
    model loglidout = logtradout;                        run;

proc glm data=volume;
    class season;
    model loglidout = logtradout|season;
    lsmeans season/pdiff;                                run;

proc glm data=volume;
    class season;
    model loglidout = logtradout season;
    lsmeans season/pdiff;                                run;

proc reg data=volume;
    model loglidin = logtradin;                          run;

proc glm data=volume;

```

```

class season;
model loglidin = logtradin|season;
lsmeans season/pdiff;          run;

proc glm data=volume;
class season;
model loglidin = logtradin season;
lsmeans season/pdiff;          run;

proc reg data=volume;
model loglidoutdcia = logtradoutdcia;    run;

proc glm data=volume;
class season;
model loglidoutdcia = logtradoutdcia|season;
lsmeans season/pdiff;          run;

proc glm data=volume;
class season;
model loglidoutdcia = logtradoutdcia season;
lsmeans season/pdiff;          run;

proc reg data=volume;
model loglidindcia = logtradindcia;    run;

proc glm data=volume;
class season;
model loglidindcia = logtradindcia|season;
lsmeans season/pdiff;          run;

proc glm data=volume;
class season;
model loglidindcia = logtradindcia season;
lsmeans season/pdiff;          run;

proc reg data=volume;
model loglidrcoeff = logtradrcoeff;    run;

proc glm data=volume;
class season;
model loglidrcoeff = logtradrcoeff|season;
lsmeans season/pdiff;          run;

proc glm data=volume;
class season;
model loglidrcoeff = logtradrcoeff season;
lsmeans season/pdiff;          run;

proc glm data=volume;
class amc;
model logtradout = logtradin|amc;

```

```

lsmeans amc/pdiff; run;

proc glm data=volume;
class amc;
model logtradout = logtradin amc;
lsmeans amc/pdiff; run;

proc reg data=volume;
model loglidout = loglidin; run;

proc glm data=volume;
class amc;
model loglidout = loglidin|amc;
lsmeans amc/pdiff; run;

proc glm data=volume;
class amc;
model loglidout = loglidin amc;
lsmeans amc/pdiff; run;

proc reg data=volume;
model logtradoutdcia = logtradindcia; run;

proc glm data=volume;
class amc;
model logtradoutdcia = logtradindcia|amc;
lsmeans amc/pdiff; run;

proc glm data=volume;
class amc;
model logtradoutdcia = logtradindcia amc;
lsmeans amc/pdiff; run;

proc reg data=volume;
model loglidoutdcia = loglidindcia; run;

proc glm data=volume;
class amc;
model loglidoutdcia = loglidindcia|amc;
lsmeans amc/pdiff; run;

proc glm data=volume;
class season;
model loglidoutdcia = loglidindcia amc;
lsmeans amc/pdiff; run;

proc reg data=volume;
model loglidout = logtradout; run;

proc glm data=volume;
class amc;

```

```

        model loglidout = logtradout|amc;
        lsmeans amc/pdiff;                                run;

proc glm data=volume;
    class amc;
    model loglidout = logtradout amc;
    lsmeans amc/pdiff;                                run;

proc reg data=volume;
    model loglidin = logtradin;                        run;

proc glm data=volume;
    class amc;
    model loglidin = logtradin|amc;
    lsmeans amc/pdiff;                                run;

proc glm data=volume;
    class amc;
    model loglidin = logtradin amc;
    lsmeans amc/pdiff;                                run;

proc reg data=volume;
    model loglidoutdcia = logtradoutdcia;              run;

proc glm data=volume;
    class amc;
    model loglidoutdcia = logtradoutdcia|amc;
    lsmeans amc/pdiff;                                run;

proc glm data=volume;
    class amc;
    model loglidoutdcia = logtradoutdcia amc;
    lsmeans amc/pdiff;                                run;

proc reg data=volume;
    model loglidindcia = logtradindcia;                 run;

proc glm data=volume;
    class amc;
    model loglidindcia = logtradindcia|amc;
    lsmeans amc/pdiff;                                run;

proc glm data=volume;
    class amc;
    model loglidindcia = logtradindcia amc;
    lsmeans amc/pdiff;                                run;

proc reg data=volume;
    model loglidrcoeff = logtradrcoeff;                 run;

proc glm data=volume;

```

```
class amc;  
model loglidrcoeff = logtradcoeff|amc;  
lsmeans amc/pdiff; run;  
  
proc glm data=volume;  
class amc;  
model loglidrcoeff = logtradcoeff amc;  
lsmeans amc/pdiff; run;
```