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A Randomization Process for Modeling Constructed Wetlands with an Optimization Example

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A Randomization Process for Modeling Constructed Wetlands with an Application for Optimization.

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Dedication

To my family, especially my wife, Rachel. You have given me the greatest gifts on earth.

To all the stormwater wetland designers…may this document serve you well.

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Abstract

The goals of this research were to 1) develop a methodology for creating randomly generated wetland designs, 2) use these designs to develop a set of equations predicting peak flow reduction, and 3) redesign Villanova University's constructed stormwater wetland. Using various software packages, a five tiered methodology was developed for creating randomly generated wetland designs. Using this approach, 2,000 wetland designs were generated. Previous literature examining the relationship between performance and wetland design suffer from two major flaws: 1) small sample size and 2) non-random samples. This methodology was developed to overcome these flaws.

Channel length, wetland area to drainage area ratio, and Manning's n roughness were statistically significant predictors of peak flow reduction. Channel length was the best predictor accounting for greater than 70% of the total variability within the dataset. The area and roughness variables accounted for 29 and 16% of the variation within the dataset respectively. Using this information, as well as practical design principles, a redesign example is included of Villanova University's constructed stormwater wetland.

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1.0 Introduction

Constructed wetlands are becoming increasingly popular for their use in attenuating peak flows and floods as well as treating a variety of wastewaters (e.g. aquaculture, domestic, and industrial; Ogawa and Male 1986; Demissie and Khan 1993; Guardo et al. 1995; Koob et al. 1999, Shepherd et al. 2001; Lin et al. 2002; Boutilier et al. 2009). Especially in light of increasingly stringent stormwater management requirements, many works have focused on better understanding how constructed wetlands attenuate flows and treat contaminated water.

General guidelines for designing constructed stormwater wetlands can be found in many state stormwater management manuals. For example, New Jersey recommends a minimum drainage area of 10-25 acres and a minimum length to width ratio (L:W) of 1:1 (NJDEP 2004), while Pennsylvania recommends a minimum drainage area of 10 acres, a wetland area of 3-5% of the drainage area, and a minimum length to width ratio of 2:1 (PADEP 2007). These manuals include other recommendations including sizing of sediment forebays, pool volumes, vegetation, and others (NJDEP 2004 and PADEP 2007). While the sources of these recommendations were not explicitly cited, most agree with recommendations agree those from the literature as discussed in Ch. 2 Literature Review.

Despite the growing body of knowledge regarding constructed wetlands, few studies exist that evaluate the mathematical relationship between design variables and performance. Furthermore, current methodologies used to optimize constructed wetland performance are insufficient for generating general predictive equations based on design parameters. Most studies examining this relationship suffer from at least one of two major flaws: small sample size and non-random samples. Larger sample sizes lead to increased precision in estimates of various properties of the population. Obtaining a large sample size of *in situ* wetlands can be exceedingly difficult, especially considering instrumentation and the logistics of data collection. As a result, many studies examine a small number (n) of wetland systems (e.g. Thackston et al. 1987, n=12; Kadlec 1990, n=1; Walker 1998, n=5; Persson et al. 1999, n=13; Somes et al. 1999, n=1; Persson 2000, n=13; Jenkins and Greenway 2005, n=10; Conn and Fiedler 2006, n=12). Although *in situ* studies can be invaluable, making deductions or applying equations based on a small number of samples may be inappropriate and lead to false conclusions about the performance of a designed wetland system.

More important than small sample sizes, previous studies have not taken random samples of *in situ* wetlands or modeled non-randomly generated designs (e.g. Ogawa and Male 1986; Thackston et al. 1987; Kedlec 1990; Demissie and Kahn 1993; Walker 1998; Persson et al. 1999; Carleton et al. 2000; Persson 2000; Tsihrintzis and Madiedo 2000; Economopoulou and Tsihrintziz 2004; Jenkins and Greenway 2005; Conn and Fiedler 2006). Random samples are necessary when estimating the true mean of a population. In order to generalize the performance of all wetlands, obtaining random samples prevents bias toward one design in particular. This is particularly evident in many studies where the L:W is used to characterize wetlands. For example, Kadlec (1990) developed the following equation:

$$Q=aWybS \qquad (1)$$

where Q is the flow rate (m3/d), a is a roughness factor (a=107/d/m or 5x107/d/m for dense and sparse vegetation, respectively), W is the wetland width (m), y is the depth of flow (m), b is a constant (3.0), and S is the surface water slope. The surface water slope can be written as the change in the water surface elevation divided by the length that change occurred (Δws/L), and the equation can be rewritten as follows:

$$Q=a(L:W)^{-1}yb\Delta ws \qquad (2)$$

where L:W is the length to width ratio.

This equation is biased toward designs with constant L:W. For square shaped wetlands, this methodology may be appropriate, but for most wetland systems, the L:W is constantly changing throughout the system. In order to create generalized predictive equations that are not biased to particular shapes, wetland designs should be randomly generated. Although existing modeling methodologies can remain the same, methodologies for randomizing designs have not been developed within the literature.

Despite the successes that have been documented in treating contaminated waters using constructed wetlands, designing optimum constructed wetlands is critical for meeting stormwater regulations and current methodologies are severely limited in their applicability, therefore, the goals of this research were to 1) develop a methodology for creating randomly generated wetland designs for use in computer simulations and 2) use the methodology to develop a set of equations to predict constructed wetland performance from the randomly generated wetland designs.

While several metrics (e.g. nutrient removal, hydraulic retention time, etc.) could be used to evaluate performance of constructed wetlands, peak flow reduction was used in this study; however, any metric could be coupled within the methodology generated within. Minimizing peak flows is important for reducing the erosive power of fast moving water. Nutrient removal is ultimately governed by the system's hydrodynamics. Reducing peak flows result in decreased water velocities and thereby promotes the natural processes that degrade and remove contaminants from the water column. In this context, peak flow performance can be viewed as a surrogate measure of pollutant removal performance.

Ultimately, mathematical representations of constructed wetlands should be based on actual systems; however, constructing a large number of randomly generated wetland designs is highly impractical. Therefore, a modeling approach was used to obtain a high degree of control over wetland parameters (e.g. roughness) as well as high precision in output data (e.g. flow). For this study, wetland performance was evaluated using the hydrodynamic module of MIKE 11(DHI, v. 2009, Hørsholm, Denmark). The hydrodynamic module of MIKE 11 is a fully dynamic one-dimensional model that solves the complete nonlinear St. Venant equations for open-channel flow between all grid points at specified time intervals for given boundary conditions (DHI 2008). The solutions to these equations are based on an implicit finite difference scheme developed by Abbott and Ionescu (1967). The hydrodynamic module of MIKE 11 is widely used and has been applied to wetland systems (see Duvail and Hamerlynck 2003; Thompson 2004; Thompson et al. 2004, Hammersmark et al. 2005).

2.0 Literature Review

2.1 Regulatory History of Stormwater Management

Since the late 19th century, many environmental laws have been enacted within the United States to control sources of pollution to better manage the nation's water resources. The earliest of these, the Rivers and Harbors Act of 1899, prohibited the discharge of refuse matter into the navigable waterways. Other laws, including the Oil Pollution Act (US Congress 1924) and the Water Pollution Control Act (US Congress 1948), attempted to control point sources of pollution by authorizing various agencies to eliminate or reduce pollution discharged into the nation's waterways.

Despite these and other measures, the water quality within the United States continued to decline. Urban sprawl has been an increasing trend throughout the nation's history. By 1950, urban and suburban areas with a population greater than one million encompassed approximately 7,000 square miles of land and would grow by 2000 square miles per year (Nechyba and Walsh 2000). This sprawl has had a cascading effect on the environment. In addition to increased impervious cover, sprawl has resulted in increased dependence on motorized vehicles, which in turn has lead to an increased deposition of nutrients, metals, and polycyclic aromatic hydrocarbons within the water column (Van Metre et al. 2000; Burian et al. 2001; Bergbäck et al. 2001). Environmental catastrophes, such as the Cuyahoga River in Cleveland in 1969, were highly publicized. The river was described by Time

Magazine as a river that “oozes rather than flows,” (1969). Popular books, such as *Silent Spring* (Carson 1962), brought the plight of the environment into popular culture and helped spawn environmental stewardship among a small part of society (Adler et al. 1993).

In response to the increasing awareness of the declining environmental health and because of the poor state of the nation’s water quality, Congress passed the Clean Water Act (originally called the Federal Water Pollution Control Act Amendments; US Congress 1972). Currently, the Clean Water Act is the primary federal law in the United States governing water pollution. The primary objective of the Clean Water Act is to restore and maintain the biological, chemical, and physical integrity of waters within the United States (USEPA 2009). In order to achieve this, the Clean Water Act established a variety of programs including the National Pollutant Discharge Elimination System (NPDES, US Congress 1972). Under the 1972 guidelines, owners of publicly owned treatment works, dischargers of industrial wastewater, and other point source dischargers were required to obtain discharge permits regulating the quality of the water being discharged (USEPA 2009). Water quality within the United States improved as a result of the Clean Water Act and the NPDES program for managing point sources of pollution (USEPA 2009).

Although stormwater runoff was a known contributor to water quality impairment (Weibel et al. 1964; Evans et al. 1968; Geldreich et al. 1968), the foundations of the Clean Water Act were initially aimed at controlling point source pollutants from industrial and municipal wastewaters. Furthermore, early stormwater managers were concerned with the effects of increased flows and flooding associated with stormwater instead of pollution control. As a result, many state governments initiated regulations to manage peak rate reduction. For example, the Pennsylvania legislature passed the Stormwater Management Act (Act 167), which required counties to prepare and adopt watershed-based stormwater management plans (CPA 1978). These plans include standards for managing the quantity of stormwater runoff given the characteristics of the watershed including current and future development plans.

Shortly after the Clean Water Act was passed, the Nationwide Urban Runoff Program (NURP) examined stormwater constituents between 1979 and 1983 (USEPA 1983). A major component of the NURP project was an analysis of water samples collected during 2,300 storms in 28 metropolitan areas. The studies quickly recognized that stormwater could be a major contributor to water quality impairment. Results from this study indicated that total suspended solids discharging from municipal separate storm sewer systems (MS4s) draining areas were ten times greater than the annual loadings of municipal sewage treatment plants (USEPA 1983). Other contaminants, including copper, lead, zinc, and fecal coliform bacteria, were prevalent in stormwater discharged from MS4s (USEPA 1983).

As a result of the NURP Program, the Water Quality Act of 1987, Section 402(p), mandated the EPA to control the quality of stormwater discharges in two phases under the existing NPDES guidelines. Phase I guidelines (published in 1990) regulated storm water discharges from medium and large MS4s, construction activities ≥5 acres, and industrial activities (NRC 2008; USEPA 2000). The Stormwater Phase I rule required all operators of regulated industrial facilities to obtain an NPDES permit, and develop a stormwater management program designed to prevent pollutants from directly entering receiving water bodies or MS4s. The stormwater management program included measures to identify major outfalls and pollutant loadings, detect and eliminate non-stormwater discharges to the system, reduce pollutants in runoff from industrial, commercial, and residential areas, and reduce pollutants from construction sites within their jurisdiction (NRC 2008; USEPA 2000).

Phase II (published in 1999) regulations built upon the existing Phase I program by requiring small MS4s and construction activities that disturb ≥1 but <5 acres of land to obtain a permit (NRC 2008; USEPA 2000). The Phase II regulations emphasize the presumptive approach, where it is assumed that each municipality has a general urban runoff problem and that this problem can be addressed through the implementation of six minimum control programs (NRC 2008; USEPA 2000). These measures are expected to reduce pollution discharged into receiving bodies of water and include public education and outreach, public participation and involvement, illicit discharge detection and elimination, construction site runoff control, post construction runoff, and pollution prevention and good housekeeping (NRC 2008; USEPA 2000).

Under the Phase I and II guidelines, stormwater management goals included the control of post-development stormwater runoff rate, replicating post-development volume and quality to pre-development conditions to prevent additional downstream flooding and to protect water resources and their uses. The Clean Water Act appeared to be a comprehensive pollution prevention piece of legislation; however, unlike the results of the point source control where water quality improvements were vast, the outcomes of the non-point source control are less apparent. Based on the National Water Quality Inventory Report to Congress, which is a biennial report written by the EPA outlining water quality of the nation, the water quality trends within the nation appear to be slowly deteriorating (Figure 1). Although the explanation of this data may not reflect the effectiveness of the federal NPDES program and its' regulations, it is an indication that more work needs to be done to improve the nation's water quality.

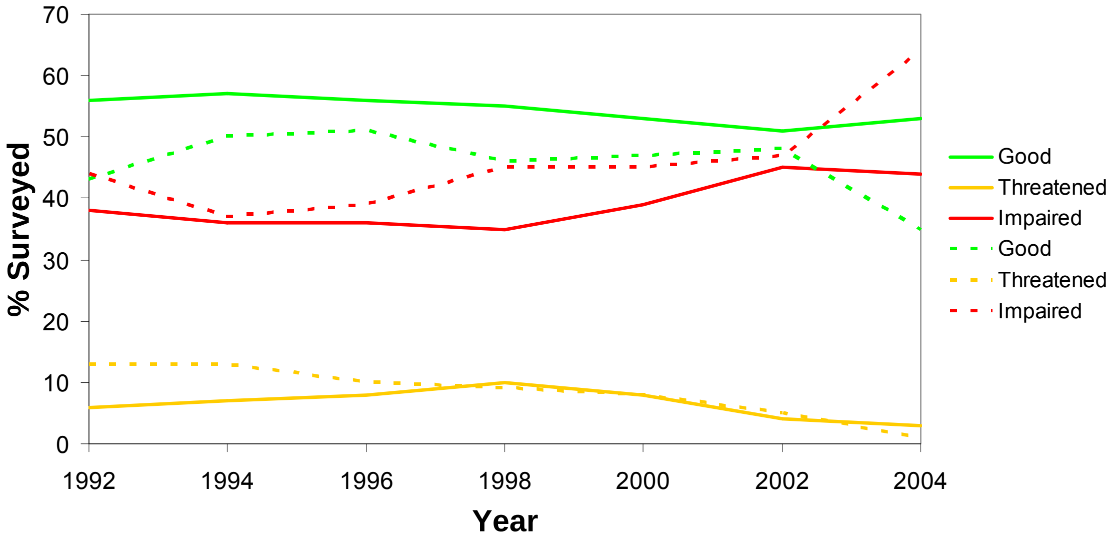

Figure 1. Data from the National Water Quality Inventory Report to Congress illustrating water quality from lakes (dotted lines) and rivers (solid lines) within the United States.

Many states have elected to adopt more stringent stormwater guidelines to meet their own stormwater management goals. In these cases, the NPDES permitting authority is operated on a state level. For example, Wisconsin requires peak flow runoff rates from post-construction conditions to match pre-development rates for the 2-year, 24-hour storm event (WDNR 2009). Similarly, Washington requires stormwater discharges to match developed discharge durations to pre-developed durations for the range of pre-developed discharge rates from 50% of the 2-year peak flow up to the full 50-year peak flow for counties west of the Cascade Mountains (WDE 2005). Regarding water quality, Pennsylvania recommends an 85% reduction in total suspended solids, total phosphorus, and a 50% reduction in nitrate (PADEP 2007).

2.2 Stormwater Control Measures

In order to meet the various water quality and quantity NPDES regulations, permittees employ various stormwater control measures (SCMs), which range from structural measures to non-structural measures. Structural SCMs include engineered or constructed facilities (e.g. stormwater wetlands, infiltration basins, etc.) that reduce pollutant loading and modify volumes and flowrates. Non-structural SCMs are preventative measures that include activities such as education and better site design that limit the generation of stormwater runoff pollutants (NRC 2008). Although a variety of SCMs exist, the remainder of this analysis will focus on constructed wetlands.

2.2.1 Constructed Wetlands

Although several definitions exist for wetlands, both natural and constructed wetlands can be defined as ecotones between terrestrial and aquatic ecosystems and are characterized by the presence of water in addition to emergent vegetation (Scholz and Lee 2005). Although the importance of natural wetlands has been overlooked historically, the destruction of wetland ecosystems has had considerable environmental ramifications, such as species diversity reduction and water reductions in water clarity. Losses in wetlands have resulted in a disproportionate fraction of threatened and endangered species within the United States as compared to other ecosystems (Boylan and MacLean 1997). Reductions in water clarity in many oligotrophic lakes, such as Lake Tahoe, have been attributed with losses of natural wetlands which remove sediments and nutrients before they reach primary water bodies (Goldman 1988).

2.2.2 Designing Wetlands for Better Performance

Currently, scientists and engineers recognize the importance of natural wetlands and their impact on water quality. Many of the same processes that occur in natural wetlands occur in constructed wetlands. By manipulating physical features within constructed wetlands, engineers can understand how performance is affected by design variables. This process can ultimately be used to optimize the processes within constructed wetlands to meet stormwater management goals.

As treatment reactors, both constructed and natural wetlands facilitate the biological, chemical, and physical processes that degrade, utilize, or remove contaminants by from the water column (Greenway 2004); such processes include microbial uptake, photodegradation, and sedimentation (Wood 1995; Kadlec and Knight 1996; Greenway 2004). Carleton et al. (2000) summarized data from 49 wetlands and concluded that long-term treatment of common stormwater constituents can be predicted based on the wetland area to drainage area ratio. Several authors have indicated that nutrient removal performance is governed by hydraulic efficiency (how well the water distributes throughout the system) with optimum efficiency described as a uniform velocity profile (e.g. plug flow; Wong and Somes 1995; Walker 1998; Persson et al. 1999; Persson 2000). Walker (1998) studied wetland basins with various length to width ratios and concluded that basins with low length to width rations (0.5:1) resulted in poor residence time and basins with length to width ratio greater than 4:1 resulted in performance similar to plug flow. Similarly, Persson et al. (1999) concluded that large length to width ratios, wide uniform depths, and wide wetlands with transverse baffles were the key characteristics for efficient wetlands. Length to width ratios of 4:1 or less resulted in poor performance. These suggestions are corroborated by others (Thacketson et al. 1987; Economopoulou and Tsihrintzis 2000; Persson 2000; Conn and Fiedler 2006).

Within wetland systems, peak flow reduction can be achieved through various mechanisms. Many wetland systems are retrofitted detention basins. Traditionally, these basins have control structures which are designed to reduce the post-development peak flow rates for a design storm; however, detention basins and their control structures are typically impractical for use in stormwater management (Emerson et al 2005). Although structures could be designed for stormwater management, other methods, such as increasing the vegetative contact have additional benefits including nutrient removal (see Ch. *2.2.2.*). Peak flow attenuation also is achieved through frictional drag caused by vegetation and other material within the flow path, which cause localized ponding within the system. This localized storage is affected by several wetland factors. Kadlec (1990) proposed that the flow rate was governed by the system roughness, the length to width ratio, the water depth, as well as the system slope (see equation 1). Additionally, in order to retain a desired flow on a watershed scale (>1000 ha), Tilley and Brown (1998) calculated the wetland area (WA) as

$$WA=Qtd^{-1} \tag{3}$$

where Q is the flow rate, t is the residence time, and d is the depth of water within the system. For t=3 days, and d=0.5m, Tilley and Brown determined that the total wetland area should encompass up to 2.5% of the drainage area. Similarly, by examining how stream flow was affected for each percent change in wetland area, Demissie and Khan (1993) found that the volume of water conveyed downstream during peak flow and flood flow decreased 3.7 and 1.4%, respectively, for every percent increase in wetland area. Ogawa and Male (1983, 1986) concluded that in addition to area, flood attenuation was affected by external variables, such as wetland location within the watershed and watershed hydrology.

Despite the growing body of knowledge regarding constructed wetlands, studies examining mathematical relationships between design variables and performance suffer from either small sample size or non-random samples or both. In order to overcome these limitations, a methodology was developed within to randomly generate a large number of wetland designs. This is the first attempt at randomizing wetland designs.

3.0 Methods

In order to develop a methodology for creating randomly generated wetlands and a set of predictive equations describing peak flow reduction through wetlands, a five tiered approach was utilized (Figure 2). First, a hydrologic model of a generic watershed (square with impervious and pervious cover) was created to generate an inflow hydrograph for each wetland design. Although multiple hydrographs could be used to assess performance under a variety of conditions, a single hydrograph was used for simplicity in illustrating the details of the methodology. Second, wetland designs were created from randomly generated minimum convex polygons. This process was repeated 2000 times to generate a large sample size. Third, a hydraulic model was created from each design. The hydrograph generated from the hydrologic model served as the inflow hydrograph for each hydraulic model. Fourth, geometric parameters were extracted from each design. Fifth, the statistical relationship was evaluated between the hydraulic performance and geometric design variables. Each process and the software used within each tier is described below.

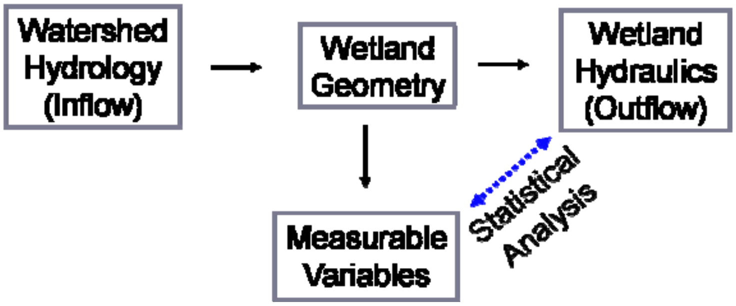

Figure 2. Flow chart illustrating the five steps used in generating wetland designs and evaluating their performance.

3.1 Hydrologic Model

In order to create constructed wetland models that fit within the context of stormwater management, the area of each design had to be scaled appropriately. In such cases where *a priori* values were needed (e.g. drainage area and wetland area) values from Villanova University's constructed stormwater wetland and its' drainage area were used.

Villanova's constructed stormwater wetland is located on Villanova University's campus in Villanova, Pennsylvania. The wetland drains 19.02 hectares of Villanova University's campus (Figure 3). The drainage area was determined by examining elevation contours, roads, stormwater drains, and stormwater pipes on or below Villanova University's campus. The average baseflow discharging from this watershed is 2.83 L/s.

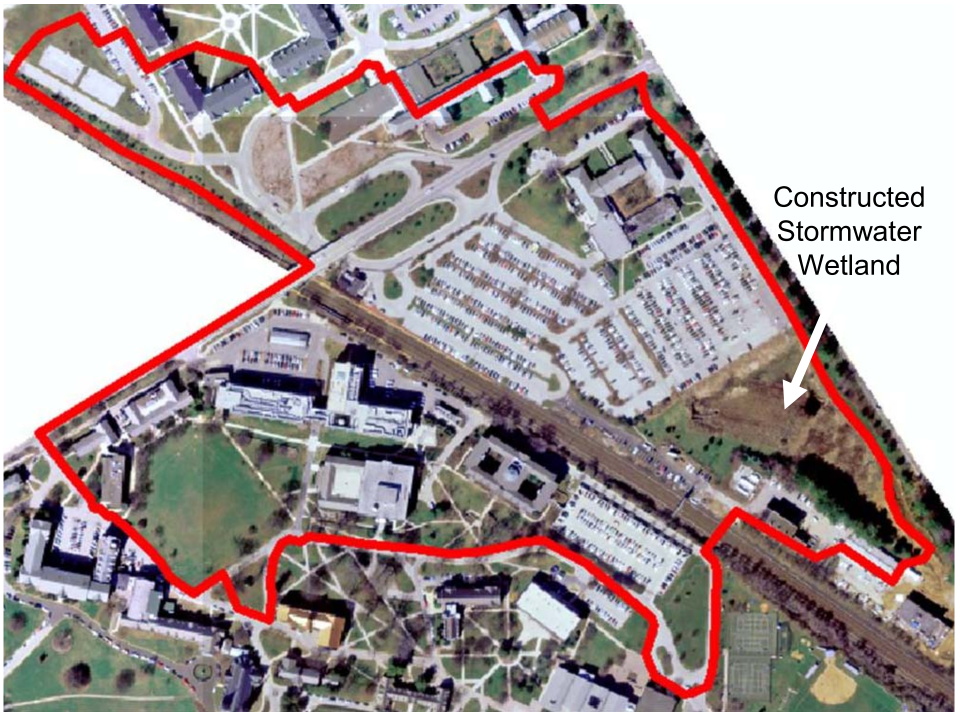

Figure 3. The watershed (red line) of Villanova University's constructed stormwater wetland encompasses 19.02 hectares of the university's campus.

The Army Corps of Engineers' Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS 3.2, US ACE, Davis, CA) was used to create a generic watershed model similar in terms of area and cover type to that of the constructed wetland drainage area on Villanova University's campus. The modeled drainage area was 19.02 hectares. In order to mimic the Villanova University's drainage area, a cover classification map was created by analyzing a remote sensed infrared image within ArcGIS 9.2 (ESRI, Redlands, CA). The goal of this analysis was to obtain three cover types (impervious, semipervious, and pervious cover) which would be used to help describe the watershed hydrology.

The National Agriculture Imagery Program (NAIP) acquires aerial imagery during the agricultural growing seasons in the continental United States and is administered by the US Department of Agriculture. The NAIP images have a ground resolution of 1 m^2. The image containing Villanova University's campus was an infrared image taken during the summer of 2004 and was made available through the Pennsylvania Spatial Data Access (PASDA) website (PSU 2008). The image was imported into ArcGIS 9.2 and was trimmed based on the extent of the Villanova University constructed stormwater wetland drainage area (Figure 4).

Figure 4. Infrared image of the Villanova University constructed stormwater wetland drainage area (solid line).

A two-step process was used to classify the wavelengths within the infrared spectrum. First, using the Spatial Analsyst extension of ArcGIS, “IsoCluster” was used to create a set of definitions that divided the continuous spectrum into twenty ordinal categories. IsoCluster uses a clustering algorithm to define the characteristics of similar raster cells in multidimensional space and stores the definitions in an output ASCII signature file. Although three categories (e.g. impervious, semipervious, pervious) were ultimately desired, twenty categories were defined in order to obtain a finer resolution of the infrared spectrum. This was done to improve classificability of the various 1m^2 cells and to identify “cover” types that did not fall into one of the desired categories. For example, since all shadows reflected similar wavelengths of light, this classification procedure grouped shadows together into the same category. The shadows were caused by above ground features (e.g. buildings and trees) and were not representative of the type of ground cover the shadow was cast upon.

Second, using Spatial Analsyst within ArcGIS and the IsoCluster definitions, “MLClassify” was to used classify the various 1 m^2 cells of the infrared image into one of the twenty groups based a cells average infrared wavelength. MLClassify performs a maximum likelihood classification on a raster image and generates a new raster. Each of the twenty categories was manually identified as impervious, semipervious, pervious, and unclassifiable cover which encompassed 57%, 31%, 12%, and <1% of the total drainage area respectively (Figure 5). The majority of unclassifiable raster cells were shadows.

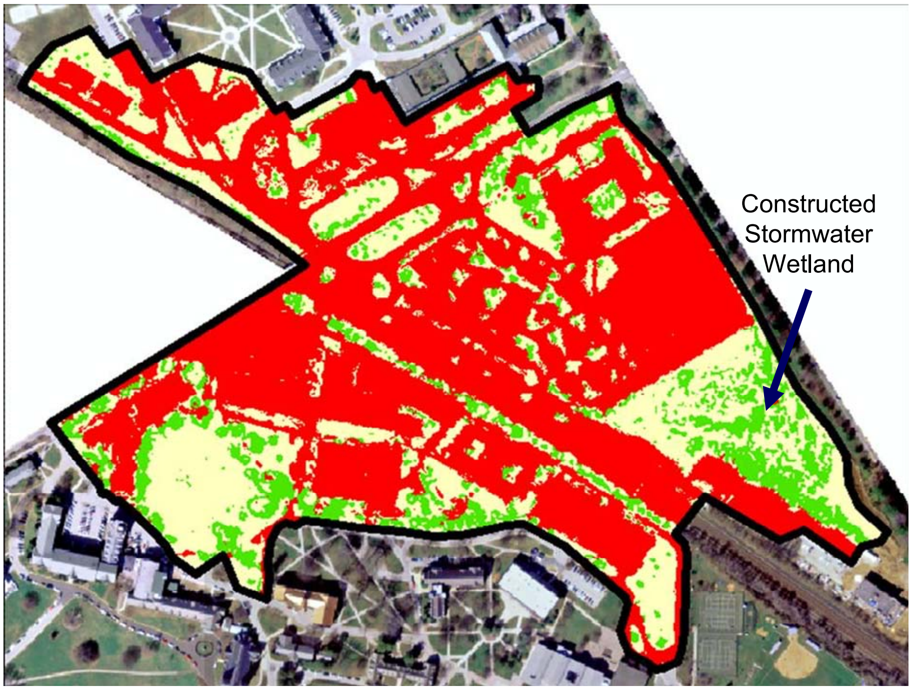

Figure 5. Impervious (red), semipervious (yellow) and pervious (green) cover of Villanova University's constructed stormwater wetland watershed.

The Soil Conservation Service Curve Number method was used to calculate the surface water runoff and losses within the HEC-HMS model (USDA-SCS 1972). Since the SCS curve number method relies on land uses, semipervious and pervious cover readily fell into the category of "lawns, open spaces, parks, cemeteries, etc." as described by the USDA-SCS (1986). Within HEC-HMS, two sub-basins were created: one for the impervious cover and one for the combined land use. The curve numbers used for impervious and the combined land use cover were 98 and 74, respectively. Runoff was generated for an SCS type 2, 1.3 cm rainfall event (Figure 6). In order to maintain generality within the HEC-HMS model, the only *a priori* values used from the Villanova University constructed stormwater wetland watershed were cover and area. No other variables were used.

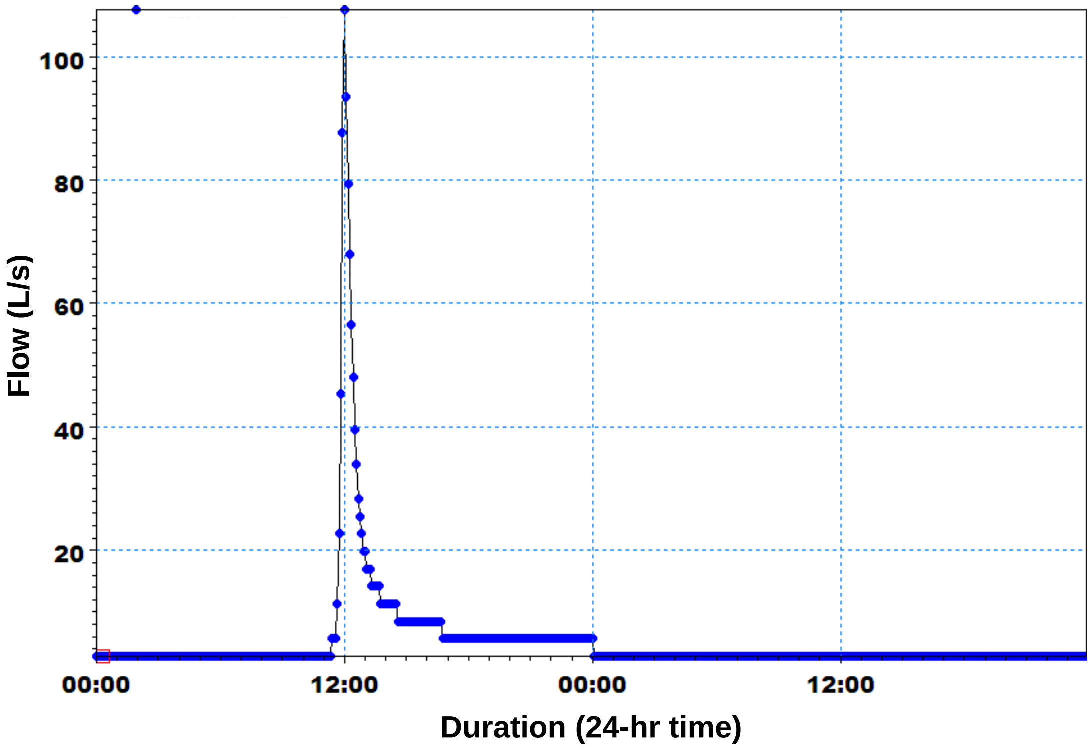

Figure 6. Runoff hydrograph generated from HEC-HMS. The maximum flow was 107.6 L/s while baseflow was 2.83 L/s.

3.2 Random Generation of Wetland Designs

A novel approach was developed to create randomly generated constructed wetland designs. Eight clusters of three to ten points were randomly generated within an arbitrary boundary using Hawth's Analysis Tools, which is an extension of ArcGIS 9.2 (Figure 7a; Beyer 2004). Hawth's Tools performs several spatial analyses that are not conveniently performed within ArcGIS. In addition to generating randomly distributed points, Hawth's

Tools was used to draw a minimum convex polygon (MCP) around each of the eight clusters of points (Figure 7b).

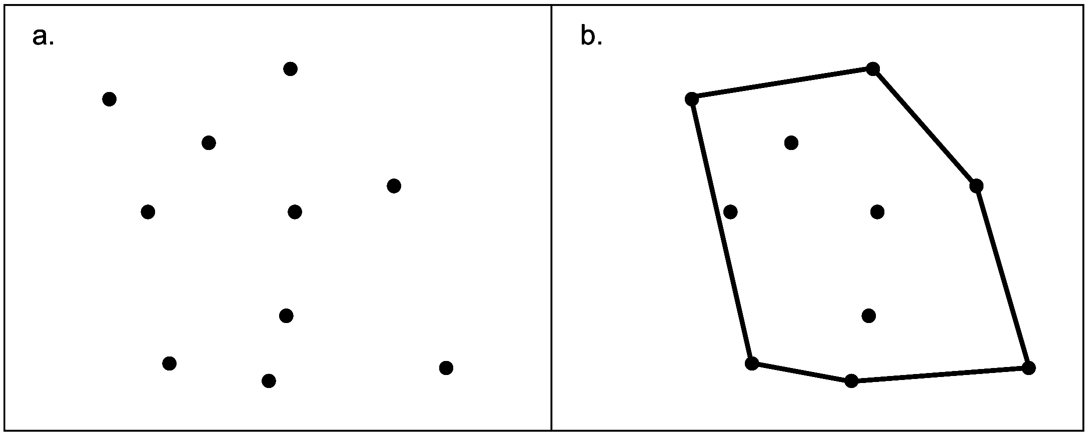

Figure 7. Each wetland design was based on a suite of randomly generated points within space. These points were surrounded by a minimum convex polygon that would serve as the perimeter of each design.

A minimum of three points are needed to create a polygon, and in general, as the number of points increased, the MCP became more globose. Conversely, and as the number of points decreased, the MCP generated had a greater tendency to be longer and narrower (Figure 8). More points were not used because as the number of points increases (ultimately approaching infinity), the MCP resembles the shape of the arbitrary boundary. Each MCP served as the perimeter of each wetland design. Points not used to delineate the design perimeter (i.e. points within the MCP) were ignored. This process was repeated ten times, generating eighty random MCPs (ten for each fixed number of points, Figure 8).

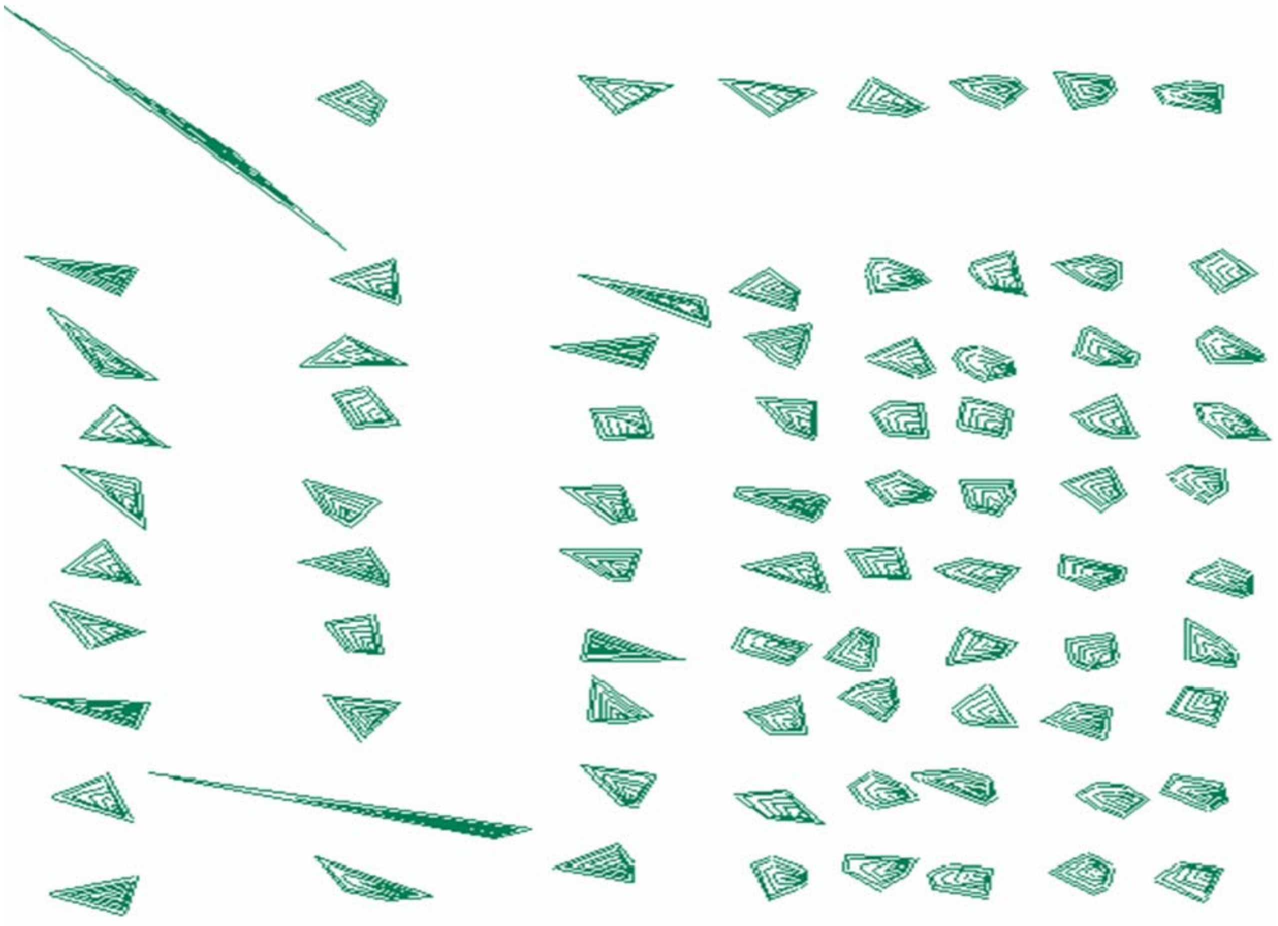

Figure 8. Eighty randomly generated wetlands were created by drawing minimum convex polygons around randomly generated points (3-10, columns) and repeating the process ten times (rows). Wetland designs shown include topography lines.

Each MCP was exported from ArcGIS into AutoCAD 8.0 (Autodesk, San Rafael, CA) to develop each wetland design. Each MCP was assigned an elevation of 2.5 m, which served as the inlet elevation for each design (Figure 9). Five topography lines were evenly distributed across the length of the MCP and were each assigned an elevation 0.5 m less than the previous line (Figure 9). A single topography line was offset five meters from the original MCP and was assigned an elevation of 4 m. This served as the over bank section for each wetland design.

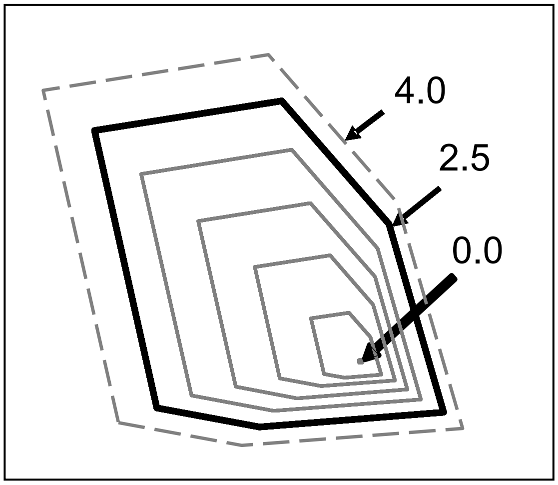

Figure 9. Topography lines were created around the original minimum convex polygons (heavy black line) within AutoCAD. These lines include an overflow area (2.5-4 m, dotted grey line) and form the internal portions of the wetland (0-2.5 m, solid grey lines).

Each wetland design was imported into ArcGIS 9.2 and converted into a shapefile using the “CAD to Feature Class Tool”. Each shapefile was converted into a digital elevation model (DEM) using the “Topo to Raster Tool” (Figure 10). A DEM is an interpolated representation of the topography lines. Interpolation is necessary for creating a continuous elevation surface instead of a discrete elevation surface using topography lines. Each DEM was scaled to 1% of the total drainage area of Villanova University's constructed stormwater wetland, so each design was 1.902 hectares.

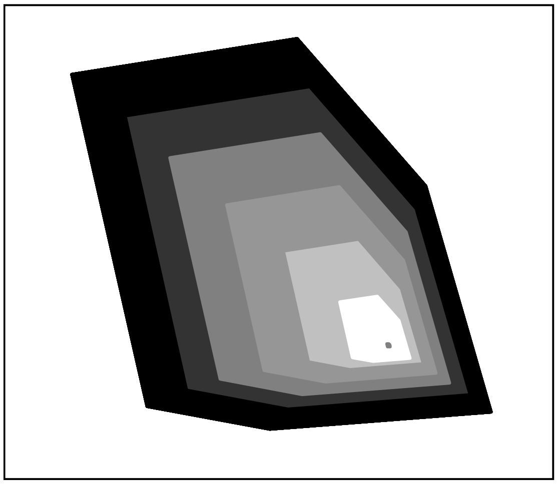

Figure 10. A digital elevation model representing a continuous elevation surface was created for each wetland design.

In order to assess how area affected performance, each wetland design was scaled to 1, 3, 5, 7, and 9% of the total drainage area of Villanova's constructed stormwater wetland, generating a total of 400 wetland designs. The process should be repeated until an adequate sample size is achieved.

For a given area, each set of 80 wetlands was assigned five different roughness values (Manning's n_m; See section "3.3 Hydraulic Model" for roughness designation) for a total of 400 designs per designated area. Wetlands with the same geometry and area, but different Manning's n_m

values were considered different designs. For each area and Manning's n_m, a total of 2000 wetland designs were included within the analysis (5x5x80=2000).

3.3 Hydraulic Model

The hydrodynamic module of MIKE 11 was used to simulate flows through each wetland design. The hydrodynamic module of MIKE 11 requires four general file types: a network file, cross section file, boundary condition file, and hydrodynamic parameters file. Each file type is described individually below.

The network file contains the upstream and downstream boundary locations as well as the channel reach for each design and was generated using MIKE 11 GIS which is an extension of ArcGIS (DHI, v. 2009, Hørsholm, Denmark). Each wetland DEM (see section "3.2 Randomizing Wetland Designs") was imported into MIKE 11 GIS and was used to create a stream channel. The inlet and outlet of the stream were the upstream and downstream boundary locations. The downstream boundary location was the lowest elevation (0 m) of each design, while the upstream boundary location was the point farthest from the outlet at an elevation of 2.5 m (Figure 11). The channel reaches were drawn along the steepest slope between the inlet and outlet of each wetland design (Figure 11).

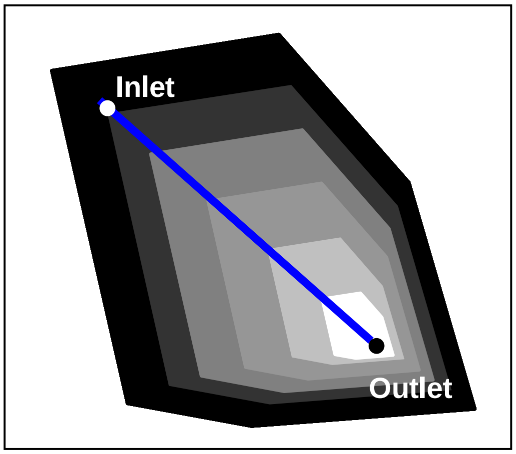

Figure 11. Inlet, outlet, and stream channels were defined using DEMs for each wetland design

Cross sections also were created for each design using MIKE 11 GIS. Using the existing DEM, boundaries and channel reaches, eleven cross sections were generated along but perpendicular to each reach (Figure 12). Each cross section was evenly spaced along the length of the reach with a cross section at the beginning and end of the reach. The resulting spacing of each cross section was 10% of the total reach length.

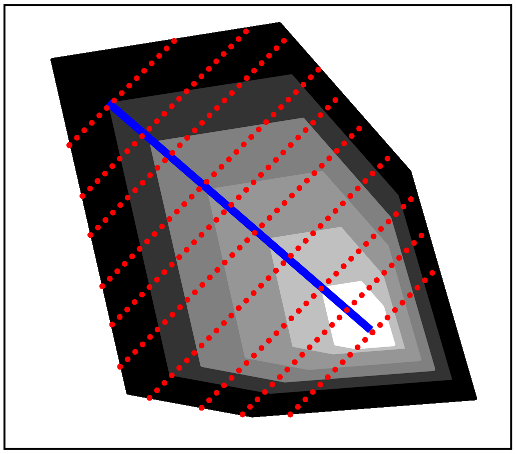

Figure 12. Eleven equally spaced cross sections were generated for each wetland design. Cross sections were based on the DEM for each design.

Within the cross section file, stream roughness coefficients were identified. For natural streams with weedy reaches with heavy underbrush, Manning's n_m values range from 0.075-0.15 (Mays 2005). For each wetland design, the Manning's n was held constant within each cross section and among cross sections. Manning's n_m values did not vary with depth; however, five simulations were run for each design using five different roughness values (0.075, 0.094, 0.113, 0.131, or 0.150). This was done to examine how variations in roughness affected wetland flow.

Within the boundary condition file, upstream and downstream boundary conditions were set for each design. The upstream boundary condition was the HEC-HMS watershed hydrograph previously described (Figure 6), while the downstream boundary condition was a flow-depth (Q-h) rating curve developed specifically for each design. Based on the cross sectional area and hydraulic radius of the last cross section of each design as well as the roughness and slope of the entire reach, MIKE 11 auto-calculated the Q-h curve using Manning's equation.

The hydrodynamic parameters file contains several parameters including wind conditions, eddy viscosity, wave radiation, etc. Default values for all hydrodynamic parameters were used in all simulations, which are sufficient for obtaining satisfactory simulation results in most cases (DHI 2008).

In addition to the hydrodynamic module, the “unsteady” simulation mode was used to calculate the surface profile for each design since the upstream boundary condition was a time dependent hydrograph. The each simulation was run using a 3 second time step with a 5 minute output storing interval. Furthermore, each design was assumed to be at steady state conditions initially. The total duration of the simulation was 48 hours, which included the 24-hour precipitation event followed by 24 hours of baseflow (2.83 L/s). A visual inspection of each outflow hydrograph revealed that a 24-hour period proceeding precipitation was sufficient for each design to return to baseflow conditions. The peak flow from each design was obtained from each outflow hydrograph.

3.4 Obtaining Meaningful Variables

In order to predict performance, selecting and obtaining meaningful variables from each design is critical. Depending on the optimization procedure, these variables may vary. In terms of peak flow reduction, four variables were selected from each wetland design. These included two length variables (channel length [m] and perimeter [m]) and two area variables (wetland area [m^2] and wetland area to drainage area ratio or the percent drainage area [m^2/m^2]). These variables were selected because each was unambiguous and easy to identify, unlike L:W. Furthermore, these variables have been used to describe hydrodynamic performance (see Ch. 2 Literature Review). Slope was not included because the elevation change within each model was held constant. Therefore, any relationship between flow and slope would be a result of length (see “3.5 Statistical Analysis” for further explanation on spurious relationships). In addition to length and area variables, each Manning’s n_m value was included in the statistical analysis.

3.5 Statistical Analysis

The predictor variable for each analysis was percent of peak inflow (%PI) at the outlet and was calculated as follows:

$$\%\text{Peak Inflow} = \frac{\text{Peak Outflow}}{\text{Peak Inflow}} 100\% \qquad (4)$$

This variable is similar to peak flow reduction, which was calculated as 1-%PI. The %PI was examined for outliers by creating scatter plots with predictor variables. Outliers were considered observations that were numerically distant from the rest of the data on the flow axis and were removed from the analysis. Extreme channel length values were not removed to better understand how these values affected peak flow within each wetland design.

Spearman correlations were used to compare the linear association between the predictor variables and %PI. In order to prevent colinearity and over parameterization within the regression models, only one length and area variable was used to generate performance predicting models since length variables are inherently correlated with each other as well as area variables. The length and area variables with the greatest correlation coefficient (R) with %PI were retained for further analyses.

Because the length and area variables are inherently correlated (e.g. increasing channel length decreases slope), predictor variables were scanned for spurious correlations. Spurious correlations occur when a correlation between a predictor (e.g. slope) and response variable (e.g. %PI) exists that does not result from any direct relation between them, but from their relation to another predictor variable (e.g. length). If the R value between the length and area variables was greater than 0.7, partial correlations were used to scan for any spurious correlations between predictor variables and %PI (McCune and Grace 2002). If a spurious correlation was detected, the irrelevant variable was removed from the analysis.

Linear and curvilinear regression analyses were used to determine which predictor variable(s) had the greatest affect on %PI. Curvilinear regression included inverse, power, exponential, and logarithmic models. The variable with the greatest R^2 was considered the best performing model since it accounted for the most variation within the dataset. Area and Manning's n_m are ordinal variables, so box and whisker plots were used instead of scatter plots to better summarize the data. Each box represents the interquartile range which contains 50% of all values. The whiskers are lines that extend from the box to the highest and lowest values, excluding outliers. The line across the box indicates the median. All statistical analyses were conducted using SPSS 10.0 (SPSS, Chicago, IL).

4.0 Results

Because channel length and percent drainage area had the greatest linear association with %PI, perimeter and wetland area were removed from further analyses (Table 1). Removing one length and area variable was appropriate since and both length variables and both area variables were highly correlated (Table 1). Although channel length and percent drainage area were highly correlated, spurious relationships were not detected using partial correlations (Table 2). Therefore, both variables were retained in the regression analyses.

Table 1. Spearman correlation coefficients (R) between predictor variables and %PI. All predictor variables were statistically correlated with %PI (P<0.001). Area and length variables were highly correlated (P<0.001). Flow results for different Manning's n_m values are indicated in parentheses.

	Channel Length	Perimeter	Wetland Area	% Area
%PI(n_m=0.075)	-0.75	-0.68	-0.50	-0.50
%PI (n_m=0.094)	-0.79	-0.74	-0.58	-0.59
%PI (n_m=0.113)	-0.82	-0.78	-0.65	-0.65
%PI (n_m=0.131)	-0.84	-0.80	-0.67	-0.68
%PI (n_m=0.150)	-0.86	-0.81	-0.69	-0.70
Channel Length	—	0.95	0.83	0.84
Perimeter	—	—	0.90	0.91
Wetland Area	—	—	—	0.98

Table 2. Partial correlation coefficients (R) between predictor variables and %PI. Flow results for different Manning's n_m values are indicated in parentheses.

Controlling for:	% Area	Channel Length
Predictor Variable:	Channel Length	% Area
%PI (n_m=0.075)	**-0.87	**-0.37
%PI (n_m=0.094)	**-0.84	*-0.11
%PI (n_m=0.113)	**-0.80	*-0.12
%PI (n_m=0.131)	**-0.77	**-0.27
%PI (n_m=0.150)	**-0.77	**-0.33

*0.01<P<0.05 ; **P<0.001

Channel length, % drainage area, and Manning's n_m were all statistically significant predictors of %PI (Table 3). Channel length was the best predictor variable and accounted for the 71% of the variability within the dataset (Table 3, Figure 13). The relationship between channel length and %PI was best explained by an exponential relationship (Table 3).

Table 3. Best performing predictive regression equations predicting %PI.

	Model	R^2	d.f.	F	P
Channel Length	%PI=107.81*e-.0019*CL	0.71	1921	4610.8	<0.001
% Drainage Area	%PI=-9.43*LN(%)+92.7228	0.29	1921	482.1	<0.001
Manning's n_m	%PI=-21.44*LN(n)+32.1151	0.16	1921	354.7	<0.001

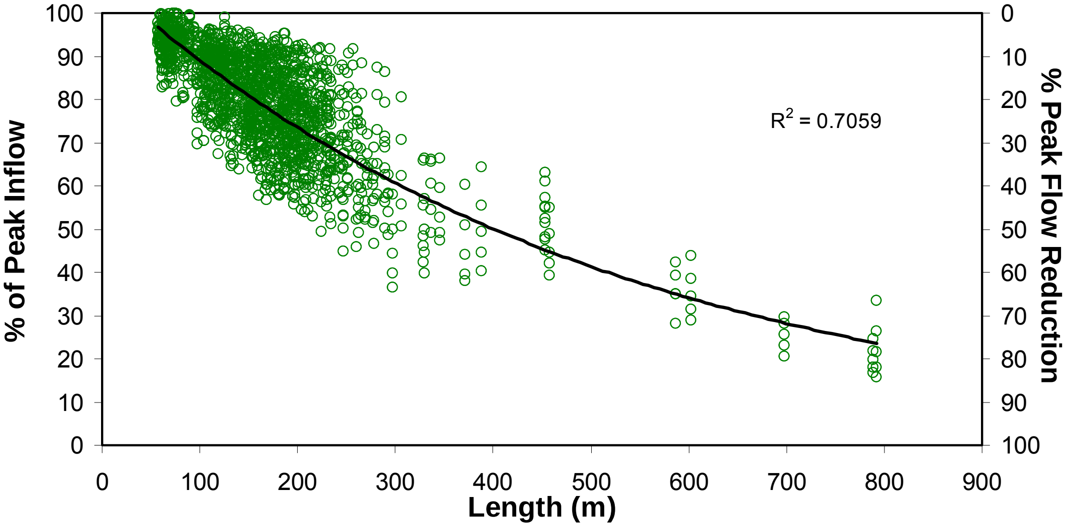

Figure 13. Scatter plot showing the relationship between channel length and %PI.

Percent drainage area and Manning's n_m accounted for 29% and 16% of the total variability within the dataset (Table 3, Figures 14 and 15). Unlike channel length, the relationship between these variables and %PI were logarithmic (Table 3).

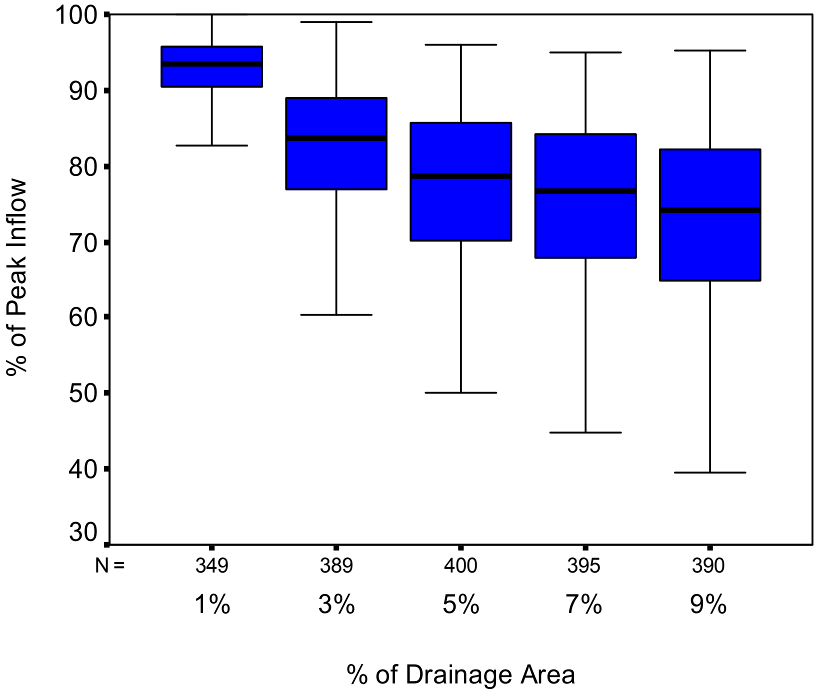

Figure 14. Box and whisker plots displaying the relationship between % drainage area and %PI.

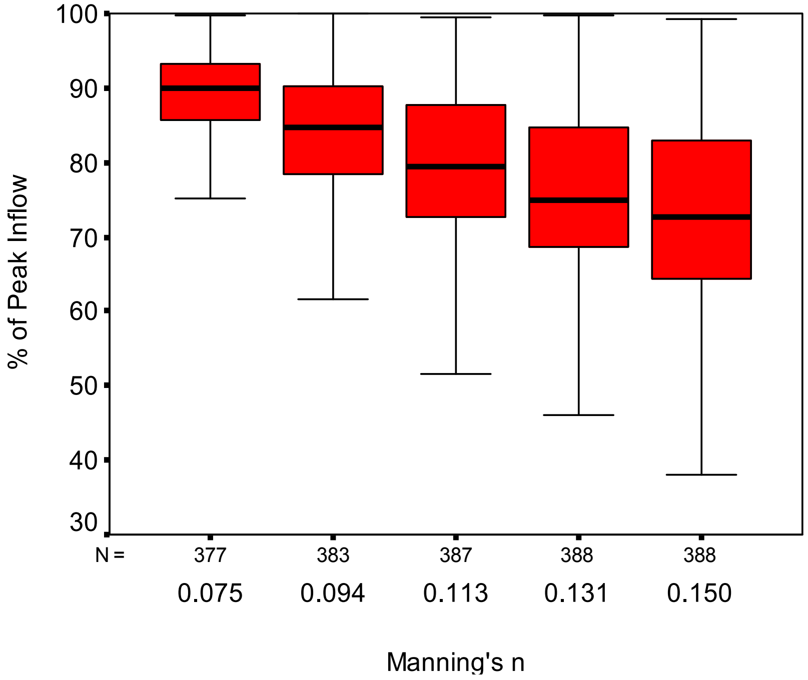

Figure 15. Box and whisker plots displaying the relationship between Manning's n_m and %PI.

Separating the dataset by Manning's n_m, channel length and % drainage area were statistically significant predictors of %PI and accounted for 79-84% and 15-46% of the total variability respectively (Table 4, Figures 16 and 17). The relationship between length and %PI while holding Manning's n_m constant was exponential, and the relationship between area and %PI while holding Manning's n_m constant was logarithmic. Manning's n_m also was a statistically significant predictor of %PI and accounted for 13-28% of the total variability (Table 4, Figure 16). The relationship between Manning's n_m and %PI while holding area constant was linear for 1% and 5% drainage area and logarithmic for 3%, 7%, and 9% drainage area.

Table 4. Predictive regression equations for various Manning's n_m and % drainage area values.

		Equation	R^2	d.f.	F	P
Channel Length	%PI(n_m=0.075)	111.007*e-0.0015*CL	0.793	377	1430.9	<0.001
	%PI (n_m=0.094)	109.476*e-0.0017*CL	0.827	383	1787.1	<0.001
	%PI (n_m=0.113)	107.285*e-0.0019*CL	0.829	387	1815.2	<0.001
	%PI (n_m=0.131)	105.893*e-0.0021*CL	0.828	388	1800.7	<0.001
	%PI (n_m=0.150)	105.280*e-0.0024*CL	0.843	388	2008.5	<0.001
% Drainage Area	%PI (n_m=0.075)	-5.2442*LN(%)+94.8749	0.156	377	69.1	<0.001
	%PI (n_m=0.094)	-7.4601*LN(%)+93.6304	0.257	383	129.4	<0.001
	%PI (n_m=0.113)	-9.6491*LN(%)+92.1934	0.352	387	202.9	<0.001
	%PI (n_m=0.131)	-11.501*LN(%)+91.5026	0.429	388	280.5	<0.001
	%PI (n_m=0.150)	-12.600*LN(%)+90.0567	0.457	388	314.9	<0.001
Manning's n	%PI (1% Area)	-56.330*n_m+ 99.405	0.131	348	52.2	<0.001
	%PI (3% Area)	-18.052*LN(n_m)+42.096	0.227	388	113.7	<0.001
	%PI (5% Area)	-240.978*n_m+104.187	0.29	399	162.5	<0.001
	%PI (7% Area)	-26.842*LN(n_m)+15.537	0.273	394	147.9	<0.001
	%PI (9% Area)	-30.970*LN(n_m)+3.497	0.276	389	147.6	<0.001

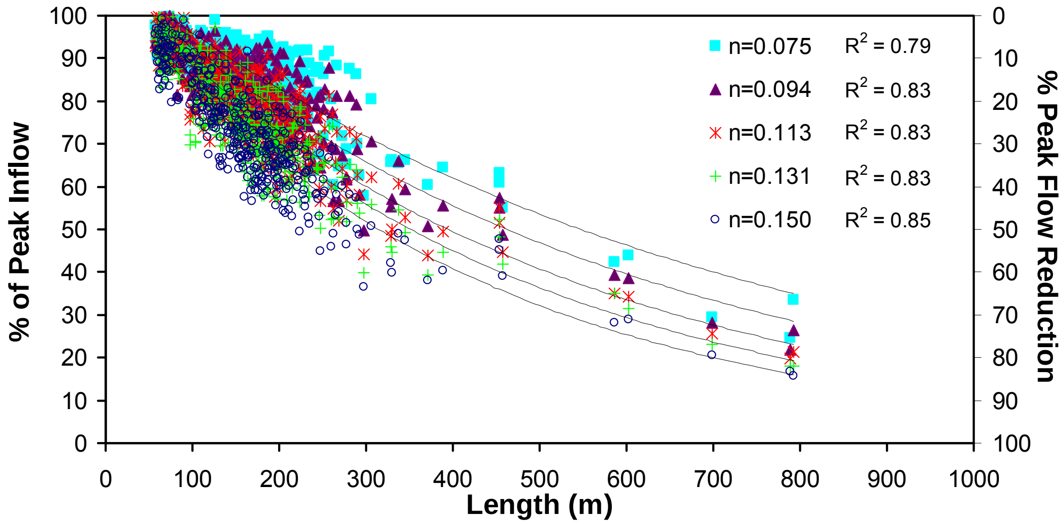

Figure 16. Scatter plot showing the relationship between length and %PI while holding Manning's n constant.

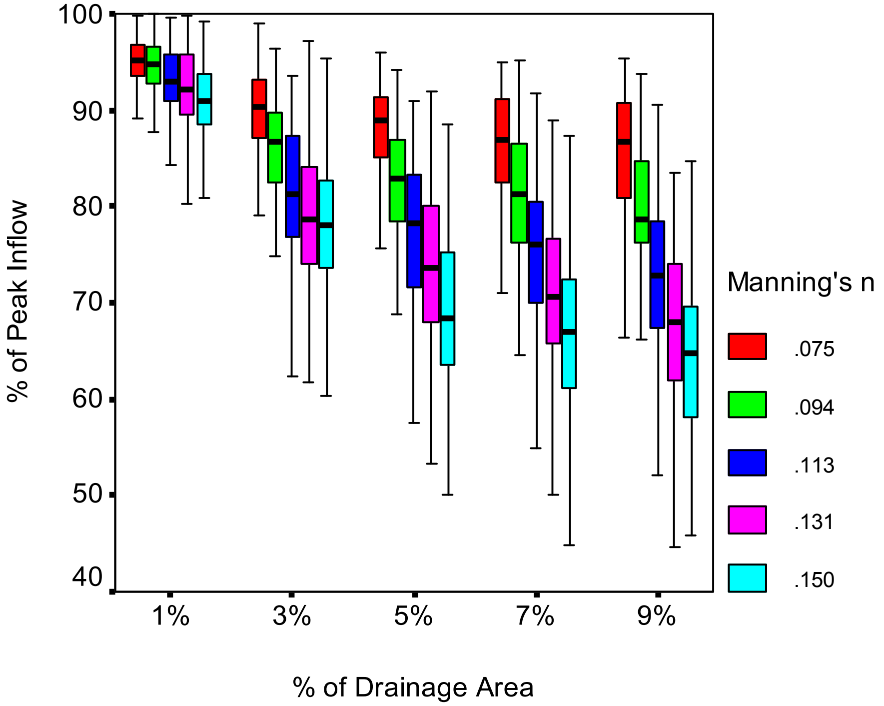

Figure 17. Box and whisker plots displaying the relationship between % drainage area and %PI and between Manning's number and %PI.

5.0 Discussion

The purpose of the methodology developed within was to 1) create a large number of randomly generated constructed wetland designs for computer simulation and 2) create a statistical model predicting the hydraulic performance of a design based on its geometric properties. Although there is still a great need to develop better methodologies to study and generalize *in situ* wetlands, models are critical for finding and developing better techniques for managing stormwater runoff. Unlike highly engineered treatment reactors, a high degree of uncertainty is inherent in studying *in situ* constructed wetlands, and obtaining large sample sizes is difficult. In such cases, using a modeling approach is advantageous for overcoming these obstacles. A high degree of control in values, such as Manning's n_m, was possible using MIKE 11.

Current methods for developing predictive equations suffer from lack of large sample sizes, as well as lack of random samples. Using the methodology developed within, these limitations were eliminated, and the equations developed within for peak flow reduction are the most general to date. Furthermore, this is the largest sample size reported in the literature used to model constructed wetland designs. Because the equations were developed from randomly generated shapes, the equations are applicable for designing constructed wetlands with a variety of shapes. Previous equations, such as the one developed by Kadlec (1990, equation 1), were based on designs with a constant length to width ratio. Although the length to width ratio can be generalized and approximated for an entire system, this equation is limited in its broad applicability.

Each regression equation based on channel length was an exponential relationship (Tables 3 and 4), which seems appropriate since a 100% reduction in peak flow is not possible (i.e. maintaining baseflow during and after a storm event). Instead, peak flow is expected to continually decrease with diminishing returns as channel length increases. Conversely, the regression equations for percent drainage area and Manning's n_m were logarithmic or linear (Table 3 and 4). Although it is expected that increasing both variables will result in lower peak flows, the logarithmic and linear nature of these equations approaches and reaches 100% reduction, which is not possible. This is a statistical artifact of the variability within the dataset. Regardless of the R^2 value, the statistical relationships do not translate to an actual relationship between predictor variables and %PI. Furthermore, because the performance model was statistical in nature, results that are extrapolated beyond the domain of the dataset should be interpreted with caution. Although a majority of the variation was explained by the various predictor variables, the relationship between each variable and performance may not be the same outside the domain of the dataset.

Reductions in peak flow are most easily explained by frictional forces. In fluid dynamics, the no-slip condition for a viscous fluid states that at a solid boundary, the fluid will have zero velocity relative to the boundary. Therefore, as the distance between a water particle and a surface (e.g. vegetation surface of channel wall) decreases, the velocity of the particle decreases and is zero at the surface. In this manner, the friction differentially slows the velocity of all water particles, thereby reducing the overall flow exiting the system.

Independent of friction, reductions in peak flow due to wetland area and channel length are explained by gravity. As in soil mechanics, gravity pulls water particles downward. Where soils can resist some shear stress due to its elasticity and can therefore be piled, water is permanently deformed as shear stress is applied. As a result, water will disperse as it encounters a surface. When water is poured onto a horizontal surface, it disperses radially from the epicenter of contact between the surface and the water. Neglecting viscous forces, water on an infinitely horizontal surface would uniformly spread across the entire surface with and infinitesimal depth. Similarly, as a flood wave enters a wetland system, gravity causes the water to disperse throughout the system. As a result of this dispersal, the depth of water decreases as the surface area increases thereby reducing the flow at the outlet of the system. Increasing channel length is the one dimensional case of the above described phenomenon.

5.1 Limitations

One risk of generating a more general statistical model that is applicable in a wide variety of scenarios is that the predictive equations may become too general to accurately predict peak flow reduction in a specific scenario. The regression equation summarizes the entire dataset into a single mathematical relationship and averages “out” the variability within the dataset. Although the regression equations based on length accounted for more than 70% of the total variation, a higher degree of scatter was observed below a length 300 meters than above this length (Figures 13 and 16)

The randomly generated designs were the centerpiece to each equation. Although each design was a randomly generated MCP, not all constructed wetlands are convex polygons. Theoretically, equations within Tables 3 and 4 should only be applied in situations where constructed wetlands can be approximated by MCPs. However, the fact that MIKE 11 is a 1-D model makes these equations more robust since the model only recognizes channel morphology and not wetland shape. Because the perimeter of each design was randomly generated, the width of each cross section is random (although the depth was not randomly chosen). The 1-D nature of the model is beneficial because the cross sections of a concave shape and a similar but convex shape would be very similar (Figure 18). In this way, a meandering channel can be modeled in a straight-line configuration. A 2-D or 3-D model may be more appropriate for modeling meanders and dendritic patterns, but generating these shapes randomly would be far more difficult especially when considering the necessity of having an inlet with a continuously downhill path to the outlet of each design.

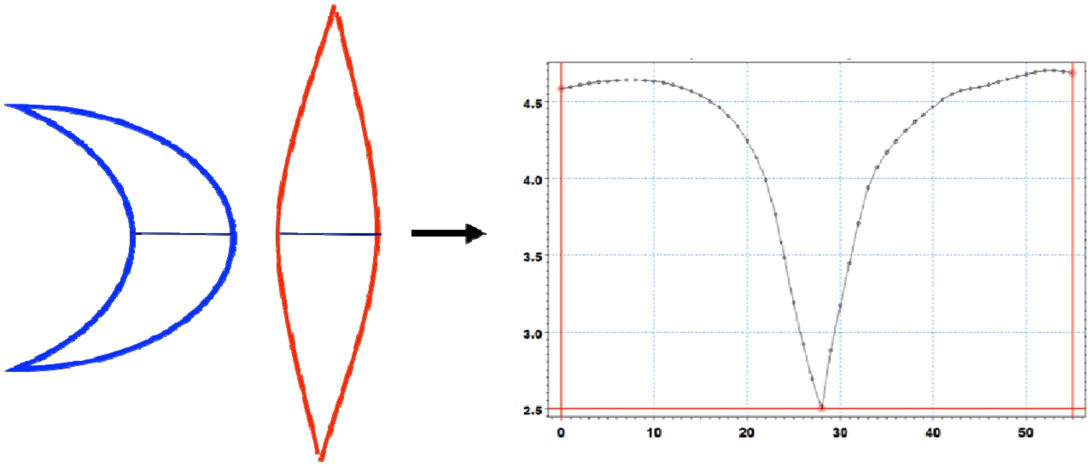

Figure 18. Cross sections between concave and convex wetlands with similar dimensions are likely to have a high degree of similarity.

Because the hydraulic behavior of constructed wetlands is intimately linked to the hydrology of the watershed, watershed characteristics cannot be ignored. The statistical models within were developed from a 1.3 cm rainfall event over a drainage area of 19.02 hectares with an impervious to semipervious and pervious cover ratio of 57:43. Therefore, these equations may apply to systems where considerable deviations exist within a watershed.

5.2 Constructed Wetland Design Considerations

Despite the limitations described above differences, the most valuable aspect of these equations is not necessarily their ability to predict peak flow reduction for all constructed wetlands designs, but to illustrate how changes in design parameters influence peak flow reduction. The reduction trends resulting from increasing channel length, percent drainage area, and Manning's n_m will be similar. Furthermore, in terms of stormwater management, constructed wetlands should be part of the solution and operate in conjunction with other stormwater control measures. In this way, the errors generated using this methodology will be absorbed within the design of the overall stormwater control system.

Increasing the channel length was the best method for reducing %PI. In order to maximize peak flow reduction, constructed wetlands should be designed with the greatest channel length practical. A typical way to increase the channel length is to create a meandering path by increasing the number of berms within the system. This practice, however, utilizes space, which could be used for treatment purposes, whether in terms of water quality or flow control. Furthermore, within a constant area, an increase in channel length must concurrently result in a decrease in channel width, which results in decreased storage volume at a particular cross section. In such instances, stormwater from large storm events can overtop the berms thereby short circuiting the system. Creative and sometimes artistic solutions can be useful for overcoming these constraints. In cases where a meandering path is desired, but earthen berms are not practicable, Jersey Barriers are potential solutions for minimizing the space necessary by creating vertical partitions between meandering channels.

Many stormwater manuals (e.g. NJDEP 2004, PADEP 2007) recommend that the wetland area should be between 3% and 5% of the drainage area. The results from this study support this recommendation. Based on the data within, below 3%, peak flow reduction occurred rapidly with increases in area, and above 5%, the rate in peak flow reduction dropped considerably (Figure 14). Therefore, in terms of peak flow reduction, constructed wetlands should be sized at least 3% of the total drainage area, but increasing the area beyond 5% results in diminishing returns. A similar break was observed in the rate of peak flow reduction by changing the Manning's n_m; however, because it is easier to increase roughness than increase wetland area, constructed wetlands should be designed to maximize roughness. This can be done by planting stronger herbaceous or inundation tolerant woody vegetation.

6.0 Constructed Wetland Design Example

In 1999, Villanova University converted an existing detention basin into a constructed stormwater wetland on its' campus. As previously described, the drainage area of the original detention basin and now the constructed wetland is approximately 19.02 hectares. The drainage sits at the headwaters of the Mill Creek watershed, which discharges directly into the Schuylkill River. Prior to 1999, the detention basin was periodically mowed as part of the regular maintenance of the university and remained dry during non-storm periods because of the presence of an underdrain, which continuously discharged water. During storm events, water entered the detention basin from two pipes that drain campus. The detention basin did not improve stormwater quality discharging from the watershed or manage small storm events. As a result, the basin was converted to a stormwater wetland in 1999.

The original design of the constructed wetland is included in Figure 19. The stormwater wetland was divided into an upper and lower section. As part of the University's long-term development plan, the area of the upper wetland was slated for potential development and subsequently the sediment forebay was placed at the top of the lower portion of the wetland. All water from the upper portion of the wetland was forced through the sediment forebay by a single berm that bisected the upper and lower portions of the wetland. Upstream of the sediment forebay no attempt was made to create a meandering path in the wetland, but three earthen berms were constructed within the lower portion of the wetland. Overall, the total length of the original design was 133 meters. The area of the usable space within the wetland is approximately 3.2% of the total drainage area.

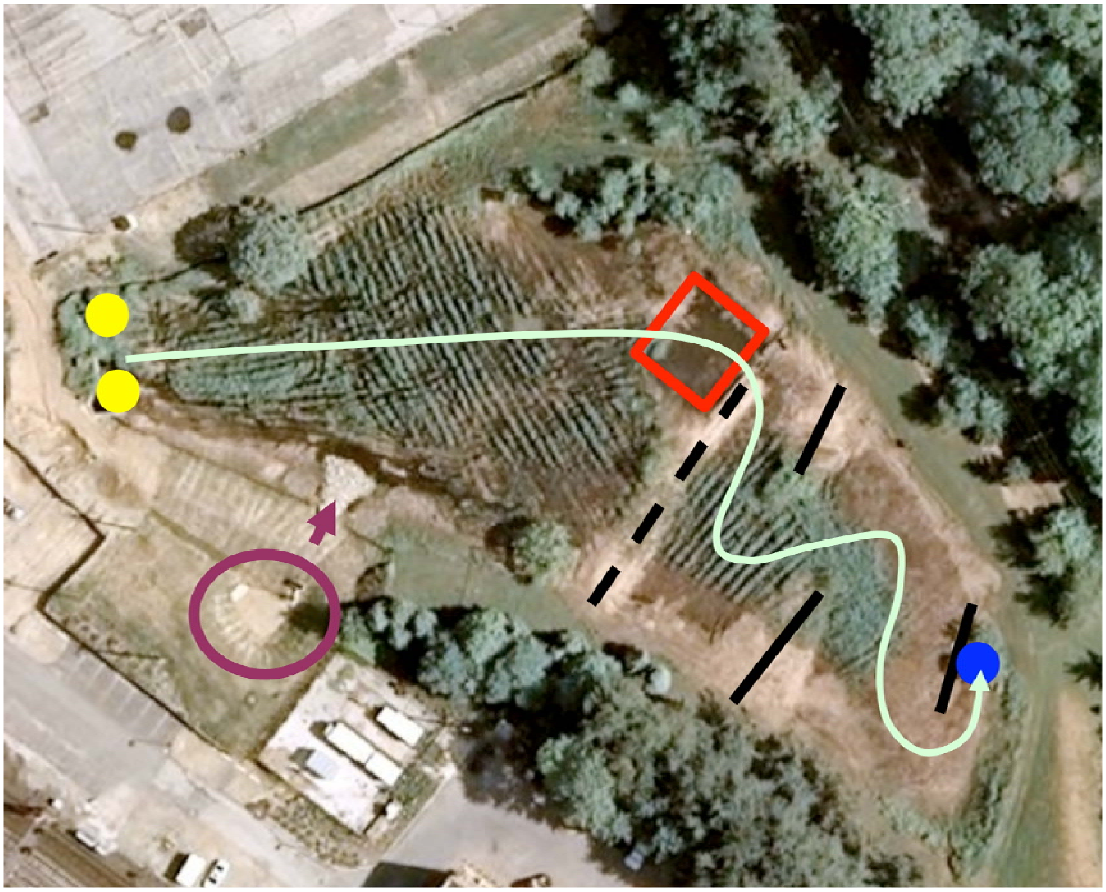

Figure 19. Aerial photograph of Villanova University's constructed stormwater wetland from 2007. Water enters the wetland from two inlet pipes (yellow), flowed through the upper portion of the wetland into the sediment forebay (red), around three earthen berms (solid lines) within the lower wetland, and into the outlet (blue). The wetland is bisected by a rock gabion and earth berm (dotted line). A small detention basin (purple) discharges water into the wetland on the south side (this was added in 2007).

Stormwater wetlands are typically designed to mimic nature. This design approach has limited value in terms of manipulatory capabilities. In order to better understand and study various processes, such as nutrient removal, carbon sequestration, and hydraulic performance, Villanova University's constructed wetland needed to be redesigned.

In the summer of 2009, Villanova University's constructed stormwater wetland was redesigned with construction slated for the spring of 2010. The primary goal of the redesign was to engineer the system to facilitate highly controlled scientific investigations while still maintaining the capacity for nutrient removal and peak flow reduction. The secondary goal of the redesign was to allow for varying levels of inundation by controlling the water level within the wetland. Unlike the original design, the upper portion of the wetland is no longer slated for development; therefore, the total area of the new system will remain at 3.2% of the total drainage area. The final redesign of Villanova University's constructed stormwater wetland is presented in Figure 20.

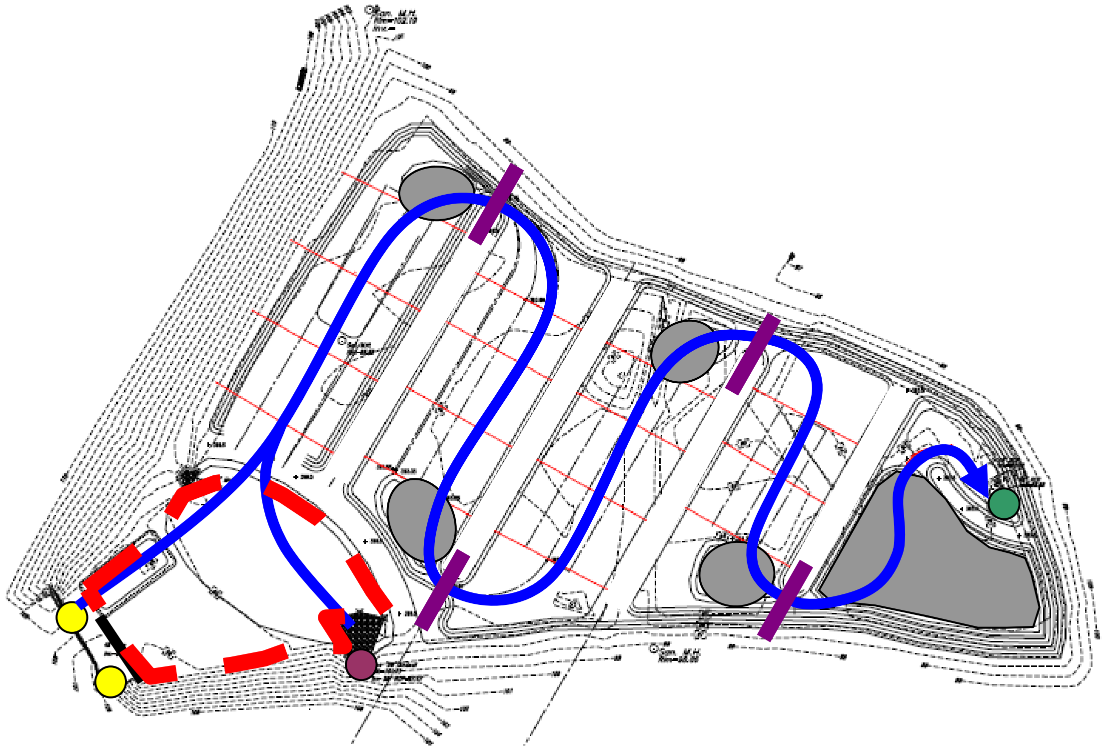

Figure 20. Illustration of the new constructed wetland. Stormwater enters the system through the existing inlet (yellow circle) as well as through the outlet of a small detention basin (purple circle). Water passes through a sediment forebay (red dotted line) before entering a series meanders. The flow passes through plunge pools (solid gray) at the end of each meander before passing under a sluice gate (purple line). A large plunge pool was placed before the outlet (green circle). Locations of typical cross sections are shown (red lines).

During the summer of 2007, construction began on Villanova University's new law school. As part of the construction process, a small detention basin was installed immediately south of the existing wetland to receive flows discharging from the new construction site (Figures 19 and 20). Flows exiting the detention basin were piped into the wetland approximately 38 meters downstream of the wetland inlet. In order to prevent short circuiting, flows exiting the detention basin will be funneled into the new sediment forebay. The new sediment forebay has a surface area of approximately 800 m^2 and is approximately 1.2 m deep for a total volume of 960 m^3, which is nearly 3.5 times greater than the recommended volume (275 m^3, Schueler and Clayton 2009). Flows leaving the sediment forebay enter a series of four meanders. Each meander is approximately twenty-five meters wide and is separated by earthen berms. The stream channel within the berms has a stair-stepped configuration (Figure 21). Each step of the channel and the top of each berm is 5 meters wide. Collectively, the horizontally sloping portions of the channel occur over a 5 meter interval. The bottom of the channel is submerged during baseflow conditions. The first step is approximately 0.15 meters above the channel bottom and is unsubmerged during baseflow conditions. This was done to maintain different plant communities (e.g. herbaceous plants and woody shrubs) that could tolerate different levels of inundation. Additionally, the maximum height of the earthen berms on either side of the channel is approximately 0.91 meters above the bottom of the channel. At this level, the top of each berm experience infrequent inundation periods and could support plant communities that were intolerant to submergence.

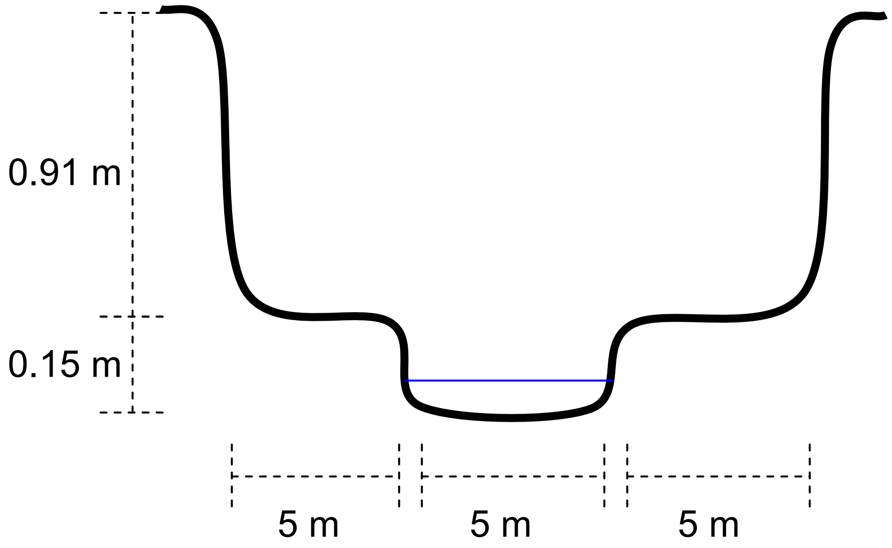

Figure 21. Typical cross section of the design of Villanova University's new constructed stormwater wetland. Line across bottom trough represents baseflow level.

In order to maximize the channel length, the stream channel made four 180-degree turns within the existing wetland (Figure 20). At each of these turns, the stream channel narrowed to 1 m in width. At this narrowest point, a sluice gate was added. Each sluice gate served 2 purposes: 1) to maintain the water level at a specific elevation during baseflow periods and 2) to further dampen peak flow through by increasing the storage (ponding) upstream of the gate during storm events.

Five plunge pools were added to the design to increase sedimentation. A small plunge pool was placed before each sluice gate. After the last meander, a large plunge pool was added to settle organic material suspended within the water column from the wetland before they are discharged to Mill Creek. The total plunge pool surface area of approximately 550 m^2. Water leaving will enter the existing outlet structure, which was part of the original detention basin, at the eastern end of the wetland. The water leaving the wetland will pass under County Line road to the northeast and serves as the headwaters to Mill Creek.

7.0 Conclusion

This represents the first attempt at randomizing wetland designs to generate predictive equations. These equations are the most general equations usable for constructed wetland design but are themselves limited in their generality. Furthermore, the methodology used within can be used to generate wetland designs to examine other aspects of wetland hydrodynamics for 1-D and 2-D models. Previous attempts at describing flow hydrodynamics were limited in sample size of either *in situ* or generated datasets. Using this approach, sample sizes can be greatly increased without the need for exploratory analyses, which ultimately bias results. Future research should examine the effects of increased rainfall as well as varying drainage characteristics, thereby generating a more general set of equations applicable in a wider variety of scenarios.

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