OPTIMIZING GREEN INFRASTRUCTURE PRACTICES FOR THE POND RUN WATERSHED IN HAMILTON, NEW JERSEY

by

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ABSTRACT OF THE THESIS

OPTIMIZING GREEN INFRASTRUCTURE PRACTICES

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Flooding appears to be a disruptive disaster to human health and property during the progress of urbanization. Due to climate change and urbanization, Green Infrastructure (GI) or Low Impact Development (LID) has become a solution for the increasing volume of rainfall by disconnecting the runoff from the sewer system, extending the water retention time, and reducing the impervious surfaces that contribute to the river. This study is seeking a better understanding of the effectiveness and economic feasibility of green infrastructure. The hypothesis is that sufficient green infrastructure practices will reduce flooding in aspects of water elevation and floodplain width.

Based on previous work, five sub-basins within Pond Run Watershed in Hamilton, New Jersey were identified as the priority areas for stormwater runoff mitigation. This study further analyzed the largest sub-basin of those five in aspects of hydrologic and hydraulic characteristics. A total of 45 points along the North Branch of Pond Run are identified as points of interest, where the geometric data of the channel cross-sections are surveyed. The peak discharges at the survey points are modeled with HydroCAD based on land use data from geographic information system (GIS) and field verifications with 1, 2, 5, 10 and 100 year type-III 24-hour-storms. The hydraulic study is modeled with Hydrologic Engineering Centers River Analysis System (HEC-RAS) to predict scenarios under various runoff situations assuming 10%, 20%, 30%, 40% and 50% of imperviousness through the whole sub-basin is disconnected by green infrastructure practices.

By comparing the water surface elevation and floodway width under different assumptions, the effectiveness of GI is analyzed. The water elevation and floodway width of the North Branch of Pond Run are reduced by applying green infrastructure under various amounts of storms. However the effectiveness of GI decreases as the amount of water precipitation increases. In addition, the peak runoff volume reduction is correlated with economic feasibility. A few design plans within the subject watershed are given as examples, which will contribute to a better understanding of both the efficiency and the cost-effectiveness of GI and will help the public to make appropriate decisions.

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Introduction/ Literature Review

Over the years urbanization has increased the conversion of pervious surfaces (forests, meadows, and other natural areas) into impervious surfaces such as roadways, parking lots, rooftops, and sidewalks. The transformation of natural lands into an urban ecology of concrete and asphalt resulted in much more runoff and less infiltration and evaporation (Arnold & Gibbons, 1996). The increase in runoff volume can cause flooding and property destruction. Additionally, urban stormwater runoff washes pollutants from urban areas into local waterbodies. When pollutant loads in the stormwater runoff entering the waterways exceed the ability of these waterbodies to assimilate them, the waterway becomes impaired. The increase in stormwater runoff and the associated pollutant loads to the waterways can result in degradation of habitat and aquatic life as well as negative impacts on water quality. Furthermore, the decrease of infiltration reduces the recharge of groundwater aquifers that provide the base flow to local streams, leaving them to go dry in the hot summer months.

Stormwater runoff has always been a severe problem for cities. From the search engine Web of Science, 14,200 papers on stormwater have been published since 1864. Among them, 60.4% are about stormwater management and design, 28.6% focus on the impacts of stormwater in terms of chemical pollutants, 2% are about the microbiological impacts associated with stormwater, and the remaining 9% are other studies on policy and human culture related to stormwater.

It turns out that a large amount of published research has focused on managing stormwater. A variety of terms are used to discuss stormwater management measures. Stormwater best management practices (BMPs) refer to structural or non-structural management measures designed to control stormwater runoff. Stormwater BMPs are also known as stormwater control measures (SCMs) by many professionals conducting research on stormwater related issues (Fletcher *et al*., 2014). Low impact development (LID) is the application of stormwater BMPs within a site development project to reduce the impact of stormwater runoff when developing land (Dietz, 2007). Green infrastructure (GI) practices are stormwater BMPs that are used to retrofit existing development with stormwater control measures and are commonly applied in urban environments. Green infrastructure practices focus on the restoration of the infiltration and evapotranspiration components of the hydrologic cycle while reducing the runoff component. Most often the location of a practice within the landscape will determine the terminology used to describe the practice.

In the published research on stormwater management and design, only 10% of the studies mentioned GI or LID, which suggest that the idea of GI or LID is relatively new. Figure 1 shows an increasing trend with the number of GI and LID studies that have been published, which suggests a bright future for GI and LID (analysis is based on the result by December, 2014. There are still documents being published).

GI is often associated with retrofitting urban areas in an effort to reduce combined sewer overflows (CSOs). In older communities, stormwater runoff and wastewater are collected in a combined sewer system (CSS). The CSS carries the slurry of wastewater and stormwater to a wastewater treatment plant where the pollutants are removed, and the cleaned effluent is discharged to a local waterway. During small rainfall events, the CSS is able to transport all the wastewater and stormwater to the treatment plant. But during heavier storm events, the CSS cannot convey all the flow to the wastewater treatment plant causing a CSO of untreated wastewater and stormwater to local waterways. This threatens both natural surface water quality and public health. CSO is identified as a link between industrial chemical waste and contaminated surface water sediments (Iannuzzi *et al.*, 1997). It contains a high concentration of contaminants, including nutrients, toxic chemicals, and heavy metals such as copper and lead (Gaffield *et al.*, 2003), which will influence the water quality within the stream. The overflow may constantly erode the river bank, which will lead to riverine flooding (Paul & Meyer, 2001). Furthermore, the CSOs carry pollutants and pathogens from the CSS into the river, altering the ecosystem of the surface water, which will also endanger public health as well. Since separating the stormwater pipe and sanitary sewer pipe is very expensive, cities with CSSs have begun implementing GI to help prevent stormwater runoff from entering the CSSs and causing CSOs.

Using GI to control stormwater runoff where stormwater management previously did not exist has been very successful in cities such as Philadelphia and Seattle (Wise, 2008). Due to this success, GI is being used to retrofit development in communities that have separate sewers to reduce localized flooding and improve water quality. The goal for stormwater management is to install GI that can capture, treat, and infiltrate stormwater runoff at key locations in the watershed to reduce overall flooding while promoting groundwater recharge. Optimizing the location of GI in a watershed is important to provide a cost-effective means of better managing water resources without creating additional problems upstream or downstream in the watershed. GI is often much less expensive than traditional stormwater infrastructure or “grey” infrastructure (Montalto *et al.*, 2007). Grey infrastructure consists of concrete pipes, tunnels, storage tanks, and detention basins to capture stormwater runoff and slowly release it back into the sewer system. The grey infrastructure systems are not designed to reduce stormwater runoff volumes or restore the infiltration or evapotranspiration components of the hydrologic cycle.

Traditional stormwater management has several limitations. Detention basins can attenuate peak flow rates but do not reduce runoff volumes. During a storm event, a detention basin can detain the runoff and release it to the watercourse or drainage system at a desired rate. The amount of stormwater runoff stays the same and can contribute to stream bank erosion (McCuen & Moglen, 1988). The capacity of grey infrastructure is designed for larger storm events (e.g. the 100 year design storm), so that the runoff can be treated appropriately. However, for small storm events such as the one-year design storm (2.8 inches over 24 hours), the performance of detention basins has shown nearly no attenuation with the largest peak flow reduction of 4% (3 ft^3/s) (Emerson *et al.*, 2005). The effectiveness of detention basins on improving water quality is not clear, and they may have little or even negative removal efficiency on dissolved pollutants if designed inappropriately (Pettersson, 1998; Bartone & Uchrin, 1999). Due to global climate change, larger rainfall events with shorter intervals of occurrence will challenge the reliability and resilience of old-fashioned stormwater management (Willems *et al*., 2012).

Given this situation, GI is designed to disconnect runoff from flowing directly into the sewer system or the local waterway during smaller storms. There are many different GI practices including green roofs, rain gardens, pervious pavements, and tree filter systems. Each of them can be an extraordinary solution to treat stormwater runoff. The main idea is to create an additional area where stormwater can be detained, which allows water to have a greater chance to infiltrate rather than being directly discharged into the drainage system. It is obvious that the perspective on stormwater and flooding has shifted. Urban stormwater runoff is regarded as a water resource, rather than waste that has to be collected and drained to disposal sites. Stormwater management is focusing on volume based hydrology rather than simply attenuating peak runoff flow (Reese, 2009). The concept is to mimic the pre-develop hydrology of the land in order to keep the impacts of human development as low as possible. When traditional stormwater management is compared to GI practices, GI is more cost-effective, reduces energy use and flood damage, and provides water quality improvements. It can bring considerable benefits to both society and the natural environment.

Different types of GI practices are described below.

<u>Trees:</u>

Increasing the number of trees on the existing landscape is the simplest green infrastructure practice. Trees can intercept rainfall, and their roots can improve soil filtering function at the same time. Shade trees can improve air quality, and through their special cooling characteristics, can also reduce urban energy consumption for cooling from a macro scale.

Infiltration systems:

Bioretention systems, or rain gardens, are landscape depressions that use herbs or woody plants to treat stormwater runoff. Bioretention systems contain a number of different plant functional areas including infiltration, filtration and sorption. The desired position for construction varies from domestic houses to middle islands in parking lots. They can be built even on mixed gravel and compost soil. Bioretention systems are usually designed to completely drain the runoff of a storm event with a length of 24 hours and no visible stranding rain should be in the system. The structure cannot be built where the groundwater table elevation is too close to the ground surface or areas with a steep slope.

Similar to rain garden, tree box filters achieve filtration and infiltration by trees, soil and gravel. It can be applied along sidewalks, roads and in parking lots. Usually trees are planted in a concrete rectangular box with an inlet and an outlet. Tree box filters are smaller than rain gardens.

Grassed swales are shallow conveyance systems that convey, detain and filter the stormwater runoff. They also can remove water pollutants such as particulate matter and heavy metals (Willis *et al.*, 2013). They are suitable for small storm events in residential, commercial and industrial areas and are usually associated with other GI practices. Big runoff flow can cause surface erosion of the planted area. Grass swales can replace curb and ditches along the road.

Permeable pavement is a rainstorm runoff discharge system that allows the precipitation to penetrate pavement into an artificial aquifer and eventually seep into the soil and the groundwater aquifer. Different types of material include pervious concrete, porous asphalt, grass pavers, and permeable pavers. The system is suitable for many kinds of roads and parking lots. The maintenance of some permeable pavement requires vacuum suction devices to clean the surface and prevent clogging (Balades *et al*., 1995).

A frequently asked question is whether infiltration systems are still functional during the winter months. When stormwater runoff enters a bioretention area, the soil is found to thaw quickly. According to related studies both bioretention areas and permeable pavement are still able to perform in the winter months with proper plowing and salting (Dietz, 2007; Davis *et al.*, 2009).

<u>Green roofs</u>:

A green roof is a plant covered roof that can handle heavy rains. It contains an isolation layer, a waterproof membrane layer, a growing medium layer and a vegetation layer. Green roofs can reduce the negative effect of buildings on surrounding natural conditions and save energy (Fang, 2008). Green roof plants can intercept and offset solar radiation, thereby reducing the energy consumption of buildings used in cooling. A green roof is heavier than a normal roof because of its complex multi-layer structure and additional water storage device. The construction of a green roof requires a comprehensive investigation before the construction of the building and an even more careful review when retrofitting existing buildings.

Rainwater Harvesting Systems:

Rainwater harvesting systems collect rainwater from impervious surfaces and store it in containers for later use. Storage container size ranges from small rain barrels with a capacity of 55 to 90 gallons to tanks with storage of thousands of gallons. Collected rainwater can be used to reduce the consumption of drinking water for non-potable uses, including irrigation, toilet flushing, car washing and fire water. Rain barrels will collect rainwater through pipes during the storm; the portion of water that exceed the volume will form overflow from the upper edge of the barrel, which will produce stormwater runoff.

Downspout disconnection:

A downspout is commonly used in buildings in North America to convey rooftop runoff to pervious areas. Disconnecting the downspout by cutting it off or re-orienting it allows the stormwater collected from the rooftop to be discharged onto a permeable surface such as grass, gardens or permeable pavement. Downspouts can also guide the rainwater to a harvesting system. Disconnection is highly recommended because of low cost and considerable feasibility.

Compared to traditional management GI practices have many advantages. Green infrastructure practices act effectively in treating stormwater runoff. The design objective of GI is to capture a certain amount of stormwater through various methods, thereby reducing the volume of stormwater runoff that flow into the drainage system or directly into the local waterway. This will reduce the frequency of CSOs occurring, thus reducing the amount of sewage discharged into natural waterbodies. Peak flow is the maximum rainstorm runoff flow rate; it is depended major on surface vegetation conditions around the valley. In aa area with more impermeable surfaces, the collected or captured rainwater is detained and slowly seeps into the ground, which will not only reduce stormwater runoff volume but also delay and reduce the peak flow rate and reduce the burden of the sewage treatment facilities. Filtration GI practices (bioretention systems, pervious pavement and grassed swales) are able to filter the stormwater pollutants, such as heavy metals, nitrogen, phosphorus, sediments and pathogens (Dietz, 2007; Kwiatkowski *et al.*, 2007; Davis *et al.*, 2009; Willis *et al.*, 2013). The portion of water that infiltrates is purified due to the effect of filtering from the soil layer and will replenish groundwater and speed up groundwater circulation. Groundwater also supplies base flow in return. The whole process is similar to the pre-development hydrology, which suggests a smaller impact to the downstream watercourse.

Green infrastructure can decrease the energy demand. For instance, green roofs that cover the building prevent heat loss in the colder months and cool down surfaces of the building by evaporation in warmer months (Gaffin *et al.*, 2010), which reduces the energy consumption of heating and cooling. Using harvested rainwater can more or less reduce the demand of water from the drinking water system, thus indirectly lowering the energy used for pumping and cleaning the water (Anand & Apul, 2011). Besides, alleviating the burden of sewage treatment plants also reduces energy consumption.

Green infrastructure enhances individual and community health. By preventing CSOs from occurring, GI lowers the concentration of bacteria and pollutants in a water body, which also lowers the chance of the public being exposed to pathogens during water-related recreational activities. Tree filter boxes can weaken the wind speed to prevent dusty weather, removing pollutants from the air, mitigating the urban heat island effect (Solecki et al., 2005). Practices such as green roofs and rain gardens can further beautify the community, promote the value of land, and create more habitats and green space. All these features make GI more functional than traditional stormwater management.

Green infrastructure practices also face financial, administrative and technical limitations. Although GI has a profound influence on sustainable development and environmental protection, those benefits would emerge gradually over a relatively long term. In addition, the investment includes educating the public on maintenance and operation of GI, since organic materials are vulnerable and sensitive to the changing of seasons and climate. As the survey (Montalto et al., 2007) shows, nearly 80% of property owners are willing to install porous asphalt or green roofs on their property if only the cost is no more than the ordinary cost. Studies suggested that the public rarely understands the benefits of GI, and the related organizations find it is hard to raise funding to implement the projects (Keeley et al., 2013). There will be a long period before the public starts to change their mind and accept GI as routine stormwater management.

Study Background

As the literature search has shown, studies on GI have increased over the years and it is important to test the actual impact from GI. Previous work (RCE Water Resources Program, 2014) delineated the Pond Run Watershed in Hamilton, New Jersey and divided it into several sub-basins. Using the data calculated by the U.S. Army Corps of Engineers (USACOE) HEC-HMS model, five sub-watershed within Pond Run Watershed with the highest runoff volume were identified as the priority areas for stormwater runoff mitigation. With funding from the Township of Hamilton, Mercer County, New Jersey, this study develops knowledge further by analyzing the largest sub-watershed of those five in hydrologic and hydraulic aspects since it is a typical type of sub-watershed in New Jersey with a large portion of residential area (62%) and commercial area (26%) (Table 1). The hypothesis is that sufficient green infrastructure practices will reduce flooding in terms of water elevation and floodplain width. By analyzing the modeling results, the effectiveness of GI is evaluated. In addition, the runoff volume reduction is correlated to cost effectiveness, which will contribute to a better understanding of the efficiency of GI and will help the public to make appropriate decisions.

The subject area, as shown in Figure 2, is located along North Branch of Pond Run from the point where it intersects with Estates Boulevard to the riverhead approximately 300 feet upstream of Paxson Avenue. The watershed has a total area of 436 acres with 85% of the land being urbanized. Detailed land use information of the urban is are given in Table

1. The floodplains along North Branch Pond Run are occupied mostly by residences and businesses. The climate of the area is temperate with an average annual rainfall of 44 inches. Temperatures range from 84 degrees Fahrenheit (°F) to 66°F during summer months and 40°F to 26°F in winter months. The watershed became developed over the past few years, resulting in 38.9% impervious coverage (169.8 acre).

Methods

To understand the relationship between GI and water surface elevation and floodway width, hydrological and hydraulic models of the watershed were constructed. Rainfall runoff from the surface into the river was regarded as a hydrological process, and analyzing the water flow through the open channel was a hydraulic model.

Hydrological analysis was conducted with HydroCAD to estimate the runoff characteristics within the watershed. This software is an adaptation of the soil conservation service's Technical Release 55- Urban Hydrology for Small Watershed (TR55). Rainfall was converted to runoff by using the runoff curve number (CN). CN is an index ranging from 0 to 100 that is dependent on soil types, cover conditions and impervious areas. A smaller CN means the area is more permeable, and usually CN=98 means the area is impervious. Runoff was then converted into a hydrograph that contains runoff volume and peak flow values by applying unit hydrograph theory and considering the longest runoff travel time through the watershed (time of concentration, Tc).

Watershed boundaries were delineated based on topographic data contained in a digital elevation model (DEM). Since the watershed is highly urbanized with a complex storm sewer system, field investigations were conducted along the edge of the watershed boundaries that were delineated with the DEM. The field investigation identified portions of the developed watershed that have storm sewer systems carrying stormwater to other watersheds. While the DEM delineated area was 530 acres with an impervious coverage percentage of 42.7%, the storm sewer piping system diverts 94 acres of drainage to outside the study area. The final delineated watershed was 436 acres with an impervious coverage percentage of 38.9%. Based on the assumption that all impervious surfaces (i.e., rooftops, playground and parking lots) are connected together by pipe system. As shown in Figure 2, this watershed was divided into 20 subareas, within which all the stormwater runoff from each subarea is completely collected and discharged into the river at the same point along with North Branch of Pond Run. Within these 20 subareas, runoff from subarea 19 is collected and drains to subarea 3 by pipe system, so further study will model subarea 19 and subarea 3 together as one single subarea.

The hydrological analysis was conducted within each of these 20 sub-watersheds. Impervious surface area and curve number values were extract a GIS (Table 2). Runoff flow path from the most hydraulic distance to the discharge point was measured from a base map and a topographic map to determine Tc values. Rainfall data that were used to calculated runoff were 1-year, 2-year, 5-year, 10-year and 100-year design storms, since this study focuses on modeling the effects of GI practices that are sized based on the design storms. The design storms are 24 hour rainfall unit hydrographs with type III distribution (Soil Conservation Service, 1986) calculated by the New Jersey Department of Agriculture based on statistical analysis of historical data (New Jersey Natural Resource Conservation Service, 2011). The unit hydrographs of water precipitation were integrated into the hydrograph library of HydroCAD, with type III distribution and duration of 24 hours. According to the New Jersey erosion control, groundwater recharge and runoff quantity standards (N.J.A.C. 7:8-5.4), the hydrological model estimated 2, 10 and 100 year storms. The 1 and 5 year design storms were also calculated to give a more detailed pattern of runoff through different type of storms. Runoff at each point was calculated with storms that re-occur with different intervals including the 1-year, 2-year, 5-year, 10-year and 100-year storms, which means the chances of each storm type occurrence within one year is 100%, 50%, 20% and 1% correspondingly. The amount of each design storm can be found from Table 11. The runoff hydrograph of each point of interest was added to the hydrograph of the point upstream and adjusted with a lag time, which is the travel time from the previous discharge point to the next point, calculated with Manning's equation:

$T_t=\frac{L}{v}$ where v= $\frac{1.486r^{2/3}s^{1/2}}{n}$ and r=$\frac{a}{P_w}$

T_t= travel time (hour)

L= flow length (ft)

v= average velocity (ft/s)

n= manning's coefficient (assuming .055 from the channel)

s= channel slope (ft/ft)

r= hydraulic radius (ft)

a= channel cross-sectional flow area (ft^2; from the survey for HEC-RAS model)

P_w=wetted perimeter (ft)

Figure 4 shows the existing hydrological condition of the channel at each discharge point during a 1-year design storm. It shows the result of the comprehensive hydrograph by adding a hydrograph to the downstream point with a lag time.

Since a stream gage is not located in the study area, the peak flow values from the hydrologic model were compared to the FEMA results (Flood Insurance Study 34021CV003A). FEMA used a downstream gage to calibrate their model (USGS Gage No. 01464000). The difference between FEMA data and HydroCAD results are shown in Table 3. The HydroCAD results are higher than the FEMA values. The FEMA results were several years old and may not accurately represent the newer development in the watershed. Secondly, HydroCAD was used to model the impervious surfaces separately from the pervious surfaces for each of the subwatersheds instead of using a weighted curve number methodology (i.e., modeling pervious and impervious surfaces together). Modeling impervious areas and pervious areas separately would yield larger peak flows. Additionally, time of concentration calculations in HydroCAD considered the movement of water from the surface to the storm sewer system, which could have yielded a quicker time of concentration than the FEMA methodology for determining peak runoff values. More information is needed on the FEMA calculations to make a better comparison of the FEMA modeling efforts and the HydroCAD modeling efforts.

The times when the peak flow occur are around 12.5 hours as shown in Figure 4. Since the HEC-RAS model does not include the change of time as a factor, while choosing the peak value for HEC-RAS will represent the hydrological situation at the peak hour at each point. Even if these peak values do not happen at the same moment, the model would show the worst case at the peak of the flood at each point. The peak values at each point are given in Table 4.

To determine the influence of the GI, a certain percentage of the imperviousness is reduced in the model of each subarea. The reduction of impervious surfaces assumes that GI can be implemented to disconnect impervious surfaces from draining directly into the storm sewer system or directly to the stream. Assuming the GI could intercept stormwater runoff from the 1, 2, 5, 10 or100-year storms, the peak flow value of each discharge point was calculated under these conditions. Five different scenarios were modeled: 10, 20, 30, 40 and 50% reduction in impervious coverage in each subarea. The peak discharges at each point are given in Tables 5-9. Looking at the flow data, impervious area can be regarded as an indicator of urbanization impact on stream (Arnold & Gibbons, 1996), which also gives suggestions for focusing on several subareas that have the largest area of impervious surface percentages. Six subareas that have the highest peak flow rates during 1 to 100 year design storms are identified as priority areas. Assuming 50% of the impervious surfaces are disconnected by GI within these areas, while the other subareas stay the same. This plan is called modified 30% off since the total area of disconnection is equal to 30% of the total impervious area of the watershed. The peak flow values are given in Table 10.

Hydraulic process, an open channel flow backwater analysis, is conducted with HEC-RAS steady flow analysis. Steady flow analysis can calculate the water surface of each cross-section combining Bernoulli's equation, the continuity equation and Manning's equation:

$$z_1+d_1+h_1=z_2+d_2+h_2+\Delta$$

$$Q= VA$$

where d= $\frac{P}{\rho g}$ h= $\frac{V^2}{2g}$

z= elevation of the bottom of cross-section (ft)

d= pressure head (ft)

h= velocity head (ft)

Δ= energy losses between cross-sections, including form loss and friction loss

Q= total flow rate (ft^3/s)

The program is able to calculate the water surface that balances the conservation of energy and mass between cross-sections, with the assumptions that the Manning's coefficient of the channel is 0.055; the Manning's coefficient of over bank area is 0.15; and 1:1 contraction and expansion ratio for bridge section.

Since topographic maps cannot provide detailed data in the channel, the geometric data underneath the water surface were manually surveyed. The survey was conducted with a total station, and the collected topographic information was used to determine the slope of the channel, delineate the elevation at cross-sections that are perpendicular to the stream. Survey points include the discharge points and locations downstream and upstream from where the channel intersects with roads so that the effect of bridges are modeled. A few more sites downstream from the research boundary were surveyed to minimize the influence of boundary conditions on the estimation of the cross-section downstream. The geometric data of the floodplain area was determined using a 2-foot topographic map. The distance of each survey point was determined from a base map. The relative elevation between each cross-section was adjusted by running a test flow. The relative elevation of each cross-section was corrected till the water surface matches the point at the edge of the water at each cross-section. Flow data comes from the hydrological estimation by HydroCAD, assuming the energy grade is generally parallel to the ground surface. Subcritical flow is considered, which predicts the highest water surface elevation with the conservation of energy and mass. The downstream boundary condition was set to normal depth with the slope between last two cross-sections. The water depth and floodway width of each point under various storms, with and without GI, were calculated within the HEC-RAS program.

Results

The water depth and floodway width differences between various GI plans and the existing condition is calculated following the equation below to make the data comparable to each other:

Z= $X - Y$

where Z= reduction (ft)

X= water depth or floodway width of existing conditions (ft)

Y= water depth or floodway width of GI plans (ft)

According to a two-way analysis of variance (ANOVA test), the calculated datasets of differences were statistically significantly different from each other ($p < 0.0001$), which suggests that there is a logical relationship between GI practices and hydraulic alteration. As shown in Figures 5 and 6, each dot represents a water surface elevation or floodway width difference of a cross-section under the conditions with or without GI. The boxplots are based on those data points. At the peak flow moment, the majority of the water surface elevation reduction is increasing as the disconnected area increases. The results support the hypothesis that sufficient GI practices would reduce water surface elevation and floodway width across the 1 year through 100 year design storms.

Comparing the GI performance under various storm events, the result can be found in Figures 7 and 8. The reduction rate goes down with the increase in the amount of rainfall, which turns out to be a margin effect in dealing with different types of storms. This is due to the fact that the capability of pervious surface to detain and infiltrate runoff is not as efficient as the scale of rainfall goes up. During larger storm events such as the 10 year and 100 year design storms, the hydrology of pervious surfaces becomes saturated and is similar to impervious surface. This makes the reduction in flow from the GI practices less important to the overall peak flow of the watershed.

The floodway width reduction of the modified solution for Pond Run, as shown in Figure 7 and Figure 8, is similar to the 30% off plan, while the range of water surface elevation reduction is less concentrated than comprehensively disconnecting 30% of impervious surfaces through the whole watershed. This suggests that making an all-over disconnection arrangement will result in slightly more improvements than just focusing on priority areas that are determined based on peak flow rates.

Discussions

The limitations of this study suggest further work, including: improving the accuracy of measuring geometric data by collecting more detailed surveying data; surveying more cross-sections to describe areas where the river bends to refine model performance; calibrating the model with storm events to improve the accuracy of the model that can be used for accurate prediction. The result from the model estimations does not show its accuracy, it only show differences. In general, it is felt that increasing GI treatment capability will alleviate riverine flooding. Further research, with measured flow data, will be needed to make any conclusions about the accuracy of the model method.

This study focuses on combining economic feasibility with reduction rate of infiltration GI practices. Infiltration GI, such as pervious pavement and bioretention/rain gardens, are widely applied compared to other GI technologies. These types of GI can be simplified as a box model (Figure 9). Assuming pervious pavement is a box with a height of 1.5 feet and a void of 40% filling with filtration media and a rain garden is a box with a height of 1 foot and a void of 100%, water is infiltrated through the bottom of both systems at a velocity of 1 inch/ hour. For each storm event, assume GI is designed to capture all the rainfall from the disconnected area. The GI management capability is calculated with HydroCAD. In Table 11, the capability is presented as acres of area that one acre of GI can disconnect.

Average unit costs of rain gardens and pervious pavement are given based on projects that have been conducted by recent projects in Camden, New Jersey (Obropta, 2015): rain garden- $10/ft^2; pervious pavement- $15/ft^2. For instance, the construction cost for a 1 year design storm with 10% disconnection, achieved all by building rain gardens throughout the watershed, can be calculated by $\frac{Design\ object}{GI\ capability}$×unit cost = $806,217. The estimation of construction costs of rain gardens and pervious pavement are calculated following the same method and can be found in Table 12. The costs from Table 12 are based on the assumption that the improvement is achieved by building all rain gardens or by applying all pervious pavement. Therefore the cost of the whole project ranges between the two values from Table 12 if rain gardens and pervious pavement are combined. Although rain gardens and pervious pavement are more widely designed than other types of GI, considering other GI applications, the real cost will be a wider range than considering just infiltration GI practices. Considering the relatively smaller improvement of GI in dealing with large storm events and the construction cost, the cost of infiltration GI practices designed for large storms is not proportional with the performance of such designs. For instance, to treat 10% imperviousness, the GI designed for the 10 year design storm costs nearly twice that of the GI designed for the 1 year design storm, with a relative lower improvement to the existing condition compared to the GI designed for the smaller storm event such as the 1, 2 and 5 year design storms. This leads to an inference that infiltration GI practices are more economically feasible when dealing with moderate and small storm events, and they perform relatively less reliably when treating large storm events. Besides, frequent flooding along the North Branch of Pond Run is not caused by storms that have a small chance of happening each year, such as the 10 year design storm (chance of 10%), the 100 year design storm (chance of 1%) and events with even lower chance. To optimize the stormwater management within the Pond Run Watershed, GI designed for storms below 4.2 inches /24 hours (i.e., less than the 5 year design storm) is suitable when considering both economic and practical feasibility.

A criticism of green infrastructure planning and modeling is related to the lack of practicality in the recommendations. For example, how feasible is a recommendation to disconnect 50% of the impervious surfaces in a watershed with green infrastructure? To address this issue, 4 design examples with disconnection plans from 10% to 50% are given to show the practical aspect of green infrastructures:

First Design Feasibility Example

The first site is a commercial development (Big Lots, 630 New Jersey 33 Hamilton, NJ). From Figure 10, it can be determined that most of the area is an impervious surface. Total impervious area is 320,000 ft^2, 77% of which is parking space. Since the parking lot is in good shape, and it is highly unlikely that the property owners will be interested in retrofitting the lot with bioretention systems or pervious pavements. The parking space angle is 90 degrees which requires a 24 foot car way between parking rows. If the parking spaces were converted to 30 degrees, the car way width could be reduced to 12 feet. This will allow for bioretention parking lot islands to be installed. Approximately 23,000 square feet of bioretention would need to be installed with a depth of 1.25 feet to capture, treat, and infiltrate stormwater runoff from 50% of the impervious surface (160,000 ft^2). Since pavement must be removed to install the bioretention systems, the cost would be higher than $\$10/ft^2$ to account for pavement removal and restriping the parking lot. At $\$12/ft^2$, the cost of bioretention would be $276,000 to capture, treat and infiltrate the 5 year design storm (4.2 inches of rain over 24 hours).

For this site, porous pavement would be a better option after the existing parking lot deteriorates. Approximately 31,313 square feet of porous asphalt would need to be installed with a depth of 1.50 feet of stone reservoir to capture, treat, and infiltrate stormwater runoff from approximately 50% of the impervious surface (159,996 ft^2). At $\$15/ft^2$, the cost of bioretention would be $469,695 to capture, treat and infiltrate the 5 year design storm (4.2 inches of rain over 24 hours). Table 13 provides a breakdown of costs for each section of parking lot that needs to be replaced.

Second Design Feasibility Example

This site is a public school (Langtree Elementary School, 2080 Whatley Raod, Hamilton, NJ). Pavement and rooftop (impervious surface) on this site is 84,100 ft^2 (Figure 11). An examination of the site indicates that the asphalt playground is already disconnected and flows onto turf grass areas. Assuming that the turf grass can absorb approximately one inch of runoff from the asphalt playground, a bioretention system/rain garden 2,550 ft^2 in size with a depth of one foot would be able to capture, treat, and infiltrate the remaining runoff from the 5-year design storm (4.2 inches of rain over 24-hours). A bioretention system also could be constructed to capture a portion of the roof runoff. A system that is 4,350 ft^2 in size with a depth of one foot would manage 26,230 ft^2 of rooftop. The cost of the bioretention systems would be \$69,000 at \$10/ft^2. Table 14 provides a breakdown of costs for each of section of parking lot that needs to be replaced.

Third Design Feasibility Example

This site is a church (Graceway Bible Church, 1934 Klockner Road, Hamilton, NJ). Most of the rainfall from this site is gathered and drained by the ditches along the western side of the church building (Figure 12). To disconnect the area from the drainage system, potential improvements would include: pervious pavement in area A, B and D to capture water from part of the parking lot and a rain garden in area C to capture water from the rooftop of the western building. Rooftop E is disconnected with the downspouts directed towards the grass. Since the grass will absorb the first inch of rooftop runoff, a rain garden would be built to capture the remainder of the runoff from this impervious surface. This rain garden would be 570 ft^2 in size with a depth of one foot. The total cost of this project would be \$76,620. Detailed design information can be found in Table 15.

Fourth Design Feasibility Example

This site is a high density residential development (Residential area, 1800 Klockner Road, Hamilton, NJ). This site is a residential area with a total impervious surface of 63,700 ft^2 (Figure 13). The design concept for this site is to disconnect the parking lot from two stormwater catchments and disconnect the rooftop from draining into the river directly. The disconnected downspouts would have to be diverted to rain gardens since the turf area cannot capture the entire 5-year design storm. The total cost of this project would be $65,965. The detailed practices can be found in Table 16.

For the four site examples, 50% of the impervious surface can be disconnected by installing GI practices such as bioretention systems/rain gardens or porous asphalt. For all four sites, rooftop runoff would need to be diverted to the GI systems. This may require a substantial effort in redirecting internal downspouts from the buildings. Three of the four sites are not publicly owned properties with the school being the exception. Permission would be needed from private property owners to install GI practices. Overall for these four sites, 6.24 acres of impervious surfaces are being captured, treated, and infiltrated with GI practices at a cost of $681,280, which is equivalent to $109,145 per acre of impervious surface managed. Total area of the sub-watershed of this study is 169.8 acres. Disconnecting 50% (84.9 acres) of the impervious surfaces within this sub-watershed will cost 9.2 million according to the analysis above.

Conclusions

Green infrastructure practices are able to improve hydraulic conditions during 1, 2, 5, 10, 100-year watershed wide storm event. The distribution of each plan with respect to different storm events is similar. The majority of floodway width and water surface elevation reduction is increasing as the percentage of disconnected impervious surfaces increases. A comprehensive disconnection arrangement is better than focusing on priority areas in this subwatershed. Infiltration based GI practices are more feasible for dealing with moderate and small storms rather than large storm events when considering both improvement effect and economic feasibility. Based on theoretical results and real design projects, a total construction cost is calculated for future reference.

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Appendix: Tables and Figures

Table 1. Detailed land use information of study sub-watershed

Land use type	Area (acres)	Percentage
Recreational Land	10.2	2.9
Commercial	95.7	25.8
Industrial	0.0	0.0
Mixed Urban	11.8	3.2
Rural Residential	1.2	0.3
Low Density Residential	3.2	0.9
Medium Density Residential	210.8	56.9
High Density Residential	16.8	4.5
Transportation/Infrastructure	20.5	5.5
Total Urban Area	370.4	100

Table 2. Land use information of each subarea

Subarea No.	Impervious surface area (acres)	Pervious surface area (acres)	Pervious surface CN	Imperviousness %	Total area (acres)
1	13.2	26.3	38.9	33.5	39.5
2	6.6	26.7	70.9	19.9	33.3
3	3.5	12.5	55.7	22.1	16.0
4	16.8	65.5	55.8	20.4	82.3
5	8.4	15.5	67.4	35.0	23.9
6	12.6	3.0	64.8	80.7	15.5
7	9.2	7.6	71.5	54.7	16.8
8	7.0	9.6	77.5	42.2	16.6
9	5.6	4.4	72.6	55.9	10.0
10	39.1	24.5	67.8	61.5	63.6
11	2.1	4.1	72.1	34.0	6.3
12	0.9	7.2	70.4	10.6	8.0
13	9.8	13.5	72.1	41.9	23.3
14	1.1	2.0	71.8	34.9	3.1
15	14.5	38.5	74.6	27.3	53.0
16	1.1	2.4	71.9	31.5	3.5
17	1.3	3.5	58.1	27.8	4.8
18	3.1	17.6	74.3	14.8	20.7
19	13.7	3.8	75.7	78.3	17.5
20	0.3	1.8	77.4	13.5	2.1

Table 3. Comparison between FEMA report and HydroCAD results

Subarea No.	10-year storm peak value (cfs)		100-year storm peak value (cfs)	
	FEMA	HydroCAD	FEMA	HydroCAD
12	150	540	360	1,098
20	540	604	880	1,212

Table 4. Runoff peak value at each discharge point with existing condition

Subarea No.	1-year (cfs)	2-year (cfs)	5-year (cfs)	10-year (cfs)	100-year (cfs)
8	20.09	24.69	33.27	41.1	74.38
2	38.31	49.85	72.34	93.88	189.98
1	63.41	79.51	110.24	138.75	275.81
3+19	89.7	111.35	153.12	192.63	379.98
18	100.8	126.88	177.41	225.25	449.55
17	102.28	128.97	180.82	229.97	460.17
4	129.62	162.89	232.02	301.23	639.25
15	152.47	192.52	275.03	356.85	749.89
14	154.11	194.7	278.02	360.64	757.09
5	164.33	208.57	298.91	388.69	816.88
16	166.11	210.97	302.44	393.29	826.11
10	212.76	267.33	377.21	484.66	992.11
13	232.56	292.31	411.78	528.49	1,074.53
12	236.33	297.77	420.52	540.24	1,098.71
6	248.39	311.98	438.54	561.68	1,133.74
9	248.39	318.69	447.42	572.58	1,153.3
11	251.49	322.71	453.17	579.87	1,167.14
7	260.86	334.67	469.57	600.41	1,205.37
20	262.25	336.56	472.37	604.07	1,212.48

Table 5. Runoff peak value at each discharge point with 10% imperviousness disconnection

Subarea No.	1-year (cfs)	2-year (cfs)	5-year (cfs)	10-year (cfs)	100-year (cfs)
8	18.35	22.65	30.67	38	69.2
2	35.52	46.61	68.49	89.52	183.03
1	58.1	73.3	102.3	129.46	261.06
3+19	81.94	102.25	141.65	179	358.26
18	92.65	117.39	165.42	210.94	426.81
17	93.99	119.3	168.6	215.45	436.92
4	118.62	150.07	215.79	282.31	608.5
15	138.68	176.28	254.61	332.76	710.5
14	140.24	178.29	257.48	336.35	717.51
5	149.69	191.35	277.34	363.05	775.19
16	151.4	193.66	280.71	367.54	784.14
10	193.38	244.6	348.4	450.95	937.85
13	211.63	267.66	380.73	491.56	1,014.97
12	215.4	273.12	389.34	503.24	1,039.15
6	226.19	285.87	405.51	522.58	1,070.62
9	231.18	292.01	413.8	532.82	1,088.58
11	234.14	295.86	419.22	539.71	1,101.56
7	242.91	306.91	434.56	559	1,137.97
20	244.3	308.8	437.36	562.64	1,145.08

Table 6. Runoff peak value at each discharge point with 20% imperviousness disconnection

Subarea No.	1-year (cfs)	2-year (cfs)	5-year (cfs)	10-year (cfs)	100-year (cfs)
8	16.63	20.61	28.06	34.89	64.1
2	32.78	43.45	64.89	85.34	176.17
1	52.81	67.09	94.52	120.33	246.8
3+19	74.19	93.22	130.25	165.4	336.52
18	84.59	107.93	153.46	196.82	404.06
17	85.78	109.67	156.47	201	413.84
4	107.62	137.28	199.87	263.61	578.39
15	126.14	161.74	236.41	311.3	675.51
14	127.63	163.61	239.07	314.71	682.14
5	136.34	175.6	257.84	340.02	737.97
16	138	177.88	261.19	344.4	746.75
10	175.3	223.39	321.84	419.98	888.52
13	191.93	244.61	351.66	457.7	960.77
12	195.66	250	360.27	469.05	983.98
6	205.19	261.26	374.52	486.09	1,012.3
9	209.77	266.88	382.17	495.56	1,028.88
11	212.51	270.47	387.26	502.04	1,040.97
7	220.57	280.69	401.54	519.99	1,074.69
20	221.96	282.57	404.33	523.6	1,081.72

Table 7. Runoff peak value at each discharge point with 30% imperviousness disconnection

Subarea No.	1-year (cfs)	2-year (cfs)	5-year (cfs)	10-year (cfs)	100-year (cfs)
8	14.91	18.57	25.46	31.78	59.04
2	30.07	40.58	61.43	81.35	169.75
1	47.57	60.93	86.91	111.33	232.77
3+19	64.4	81.69	115.57	148.14	308.52
18	74.06	95.55	137.84	178.45	374.19
17	75.12	97.15	140.56	182.29	383.33
4	94.25	121.58	180.32	240.45	541.17
15	111.16	144.34	214.63	285.55	633.62
14	112.51	146.1	217.11	288.8	639.9
5	120.5	157.21	234.74	312.84	693.44
16	122.04	159.25	237.84	316.88	701.74
10	154.73	199.28	291.84	384.5	831.58
13	169.74	218.6	319.14	419.53	899.75
12	173.46	223.94	327.49	430.69	922.5
6	181.78	233.69	339.93	445.19	946.49
9	185.88	238.85	346.89	453.77	961.81
11	188.48	242.2	351.72	460.04	973.52
7	195.76	251.55	364.84	476.72	1,004.87
20	197.15	253.42	367.61	480.3	1,011.75

Table 8. Runoff peak value at each discharge point with 40% imperviousness disconnection

Subarea No.	1-year (cfs)	2-year (cfs)	5-year (cfs)	10-year (cfs)	100-year (cfs)
8	13.19	16.54	22.87	28.75	54.08
2	27.66	37.94	58.21	77.51	163.35
1	42.36	54.96	79.39	102.5	218.85
3+19	58.98	75.34	107.79	139.07	294.34
18	68.69	89.29	130.18	169.38	359.8
17	69.62	90.71	132.68	173.02	368.52
4	86.01	112.02	168.6	226.86	519.18
15	101.46	133.09	200.65	269.2	606.94
14	102.7	134.7	203.02	272.18	612.88
5	109.84	144.98	219.64	295.02	664.4
16	111.26	146.92	222.55	298.93	672.4
10	139.29	181.39	269.93	359.28	791.26
13	152.73	198.75	294.94	391.26	854.55
12	156.42	204.06	303.15	402	876.96
6	163.42	212.33	313.42	414.47	898.05
9	167.08	216.95	319.59	422.1	911.74
11	169.46	220.07	324.13	427.76	922.41
7	175.98	228.54	336.17	442.96	951.58
20	177.36	230.4	338.91	446.5	958.45

Table 9. Runoff peak value at each discharge point with 50% imperviousness disconnection

Subarea No.	1-year (cfs)	2-year (cfs)	5-year (cfs)	10-year (cfs)	100-year (cfs)
8	11.47	14.51	20.33	25.75	49.87
2	25.52	35.48	55.09	73.81	157.14
1	37.32	49.09	72.18	94.07	205.57
3+19	51.43	66.62	96.88	126.19	273.68
18	60.88	80.3	118.88	156.08	338.21
17	61.67	81.56	121.14	159.39	346.43
4	75.29	99.76	153.45	209.07	490.19
15	89.36	119.09	183.34	248.74	573.07
14	90.51	120.59	185.47	251.54	578.85
5	96.96	130.02	200.99	273.1	628.37
16	98.28	131.85	203.76	276.76	636.15
10	121.54	161.01	245.05	330.08	744.43
13	133.41	176.6	267.7	359.38	803.33
12	137.06	181.7	275.49	369.93	824.94
6	142.79	188.33	283.89	379.92	842.59
9	146.02	192.47	289.42	386.63	855.44
11	148.19	195.37	293.49	391.91	865.61
7	154	202.91	304.22	405.81	892.48
20	155.37	204.76	306.91	409.28	899.04

Table 10. Runoff peak value at each discharge point with modified 30% disconnection

Subarea No.	1-year (cfs)	2-year (cfs)	5-year (cfs)	10-year (cfs)	100-year (cfs)
8	20.09	24.69	33.27	41.1	74.38
2	32.19	42.53	62.97	82.86	171.91
1	57.26	72.24	100.86	127.56	257.25
3+19	70.91	89.08	124.43	158.34	323.45
18	82	104.54	148.7	190.96	393.02
17	83.48	106.67	152.13	195.68	403.45
4	97.24	124.51	182.96	243.14	543.96
15	109.78	141.87	210.43	280.15	623.57
14	111.46	144.05	213.35	283.84	630.56
5	121.86	157.93	234.14	311.53	689.68
16	123.7	160.37	237.66	315	698.66
10	149.16	192.39	268.19	371.48	810.56
13	168.82	217.15	302.16	414.2	890.49
12	172.58	222.55	310.76	425.57	913.59
6	178.8	230.05	320.43	437.25	933.59
9	184.01	236.44	328.93	447.78	951.7
11	187.03	240.35	334.39	454.69	964.33
7	196.35	252	350.31	474.68	1,000.99
20	197.74	253.88	353.1	478.29	1,008.02

Table 11. Capability index of the selected GI practices

Design storm	Rainfall (inch over 24 hours)	rain garden	pervious pavement
1-year	2.8	9.2	7.5
2-year	3.3	7.8	6.5
5-year	4.2	6.1	5.3
10-year	5.8	5.1	4.6
100-year	8.3	3.0	3.2

Table 12. Total construction cost of GI practices

Storm frequency (year)	Disconnected area (%)	rain garden cost ($)	pervious pavement cost ($)
1	10	806,217	1,480,579
	20	1,612,434	2,961,159
	30	2,418,652	4,441,738
	40	3,224,869	5,922,318
	50	4,031,086	7,402,897
2	10	954,147	1,708,326
	20	1,908,294	3,416,652
	30	2,862,441	5,124,978
	40	3,816,588	6,833,304
	50	4,770,735	8,541,630
5	10	1,220,421	2,096,571
	20	2,440,841	4,193,143
	30	3,661,262	6,289,714
	40	4,881,682	8,386,286
	50	6,102,103	10,482,857
10	10	1,442,315	2,420,196
	20	2,884,630	4,840,391
	30	4,326,945	7,260,587
	40	5,769,261	9,680,783
	50	7,211,576	12,100,978
100	10	2,440,841	3,516,363
	20	4,881,682	7,032,726
	30	7,322,523	10,549,089
	40	9,763,364	14,065,453
	50	12,204,205	17,581,816

Table 13. Cost estimation of GI for commercial parking lot

GI practices	Area of GI Practice (square feet)	Area disconnected (square feet)	Percentage of disconnected area (%)	Cost ($)
A. pervious pavement	6,345	27,775	8.7	95,175
B. pervious pavement	12,357	65,071	20.3	185,355
C. pervious pavement	5,800	30,362	9.5	87,000
D. pervious pavement	6,811	36,788	11.5	102,165
Total	31,313	159,996	50.0	469,695

Table 14. Cost estimation of GI for elementary school

GI practices	Area of GI Practice (square feet)	Area disconnected (square feet)	Percentage of disconnected area (%)	Cost ($)
A. disconnected playground/ rain garden	2,550	20,427	24.3	25,500
B. rain garden	4,350	26,230	31.2	43,500
Total	6,900	46,657	55.5	69,000

Table 15. Cost estimation of GI for Graceway Bible Church

GI practices	Area of GI Practice (square feet)	Area disconnected (square feet)	Percentage of disconnected area (%)	Cost ($)
A. pervious pavement	590	3,092	4.6	8,850
B. pervious pavement	576	3,058	4.6	8,640
C. rain garden	978	5,780	8.7	9,780
D. pervious pavement	2,910	15,887	23.9	43,650
E. disconnected rooftop/ rain garden	570	4,542	6.8	5,700
Total	5,624	32,359	48.6	76,620

Table 16. Cost estimation of GI for residential area

GI practices	Area of GI Practice (square feet)	Area disconnected (square feet)	Percentage of disconnected area (%)	Cost ($)
A. pervious pavement	739	4,114	6.5	11,085
B. pervious pavement	995	5,005	7.9	14,925
C. pervious pavement	1,197	6160	9.7	17,955
D. downspout disconnection/ rain garden	805	6,452	10.1	8,050
E. downspout disconnection/ rain garden	695	5,572	8.7	6,950
F. downspout disconnection/ rain garden	700	5,585	8.8	7,000
Total	5,131	32,888	51.6	65,965

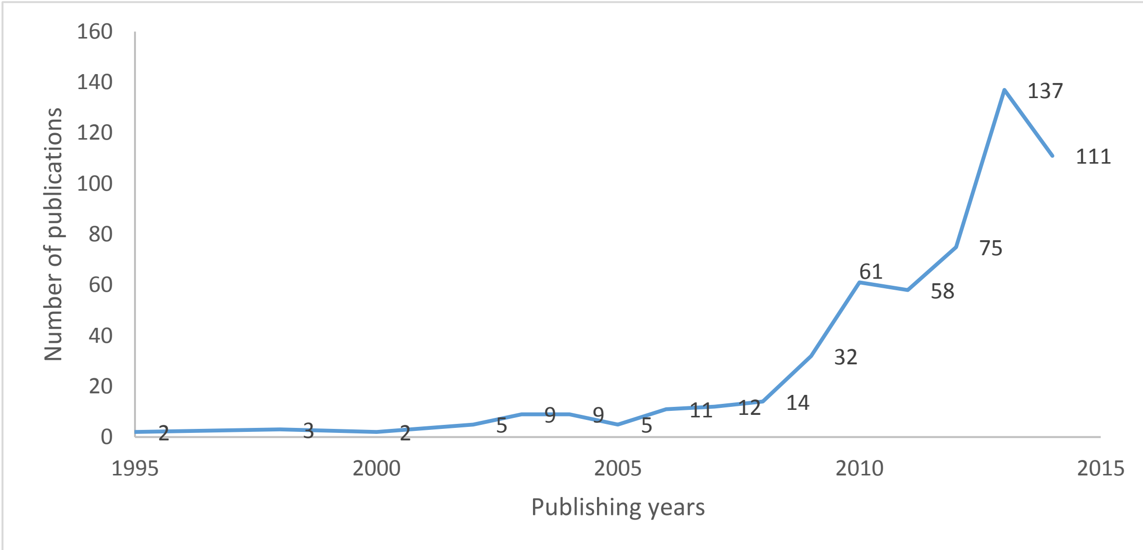

Figure 1. Publication number correlated to publication years on green infrastructure or low impact development

Figure 2. Introduction of the subject watershed

Figure 3. Delineation of the Pond Run Watershed

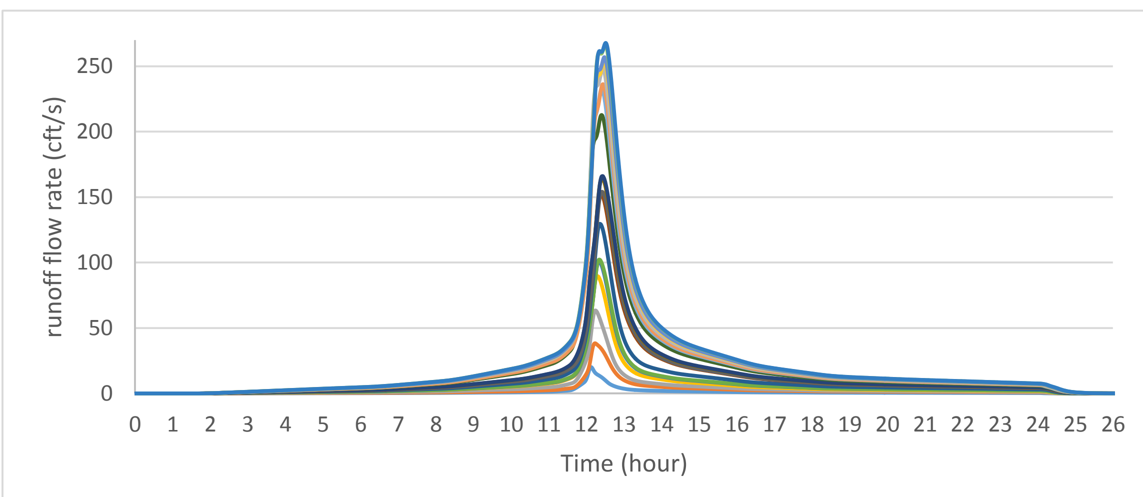

Figure 4. Hydrological result at each discharge point at existing condition, 1-year storm

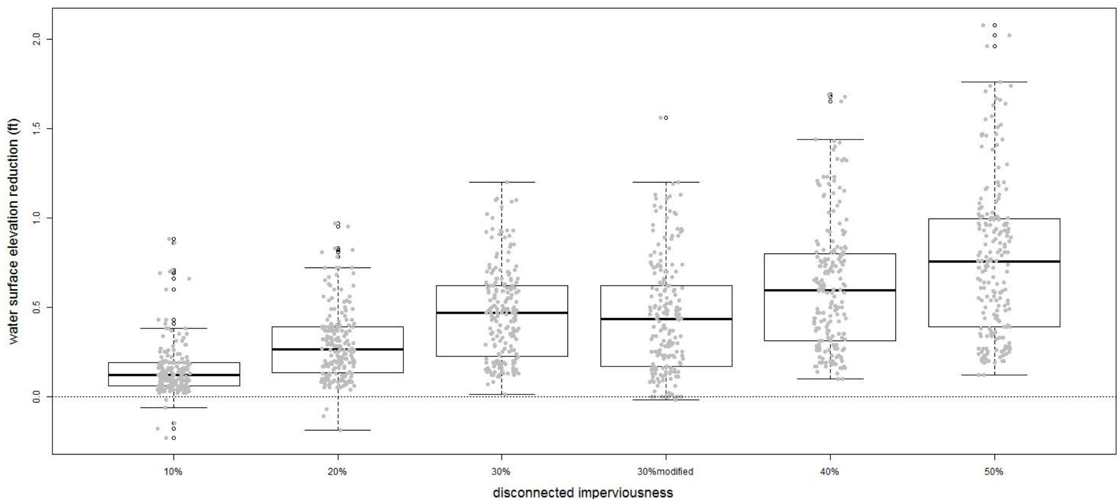

Figure 5. The influence of imperviousness disconnection on water surface

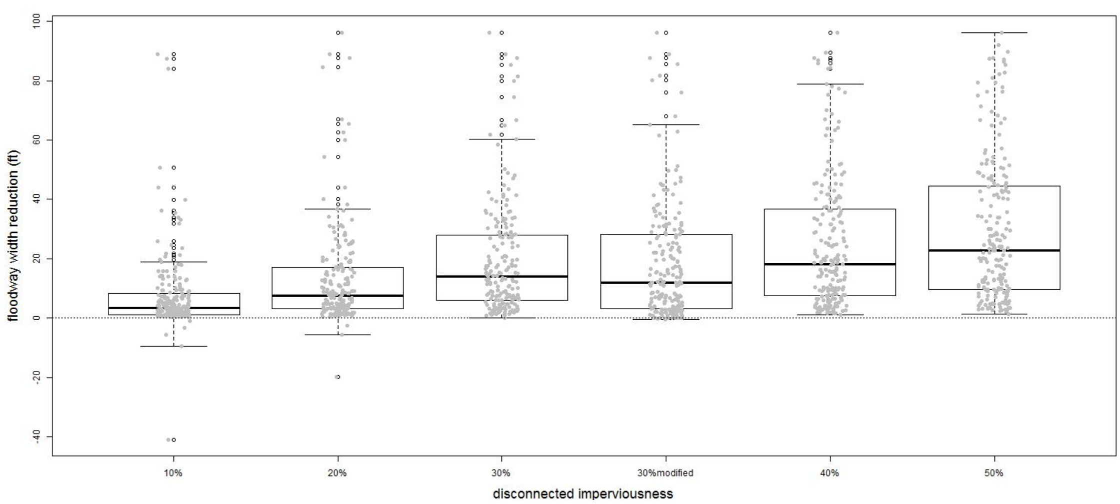

Figure 6. The influence of imperviousness disconnection on floodway width

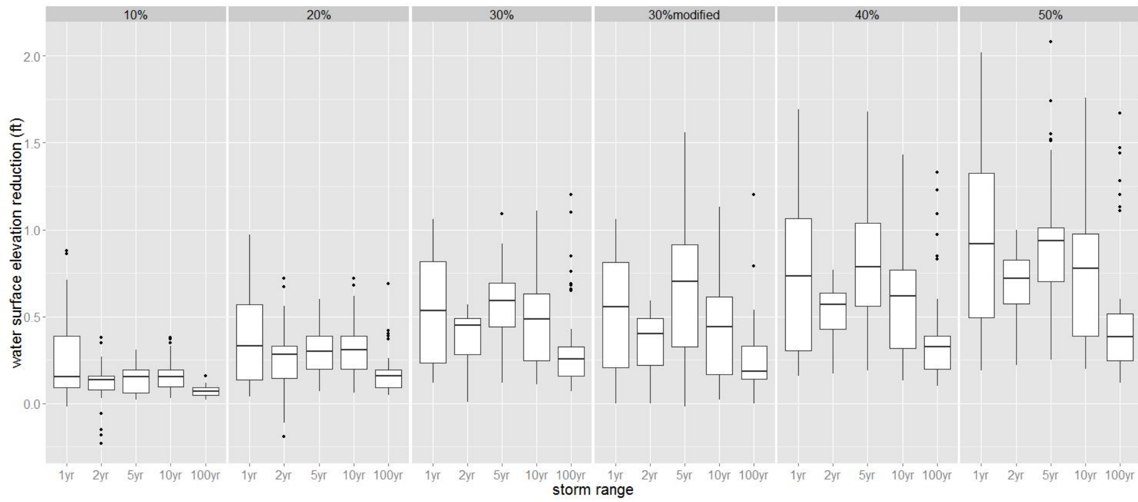

Figure 7. Comparing the influence of imperviousness disconnection on water surface elevation during different storm events

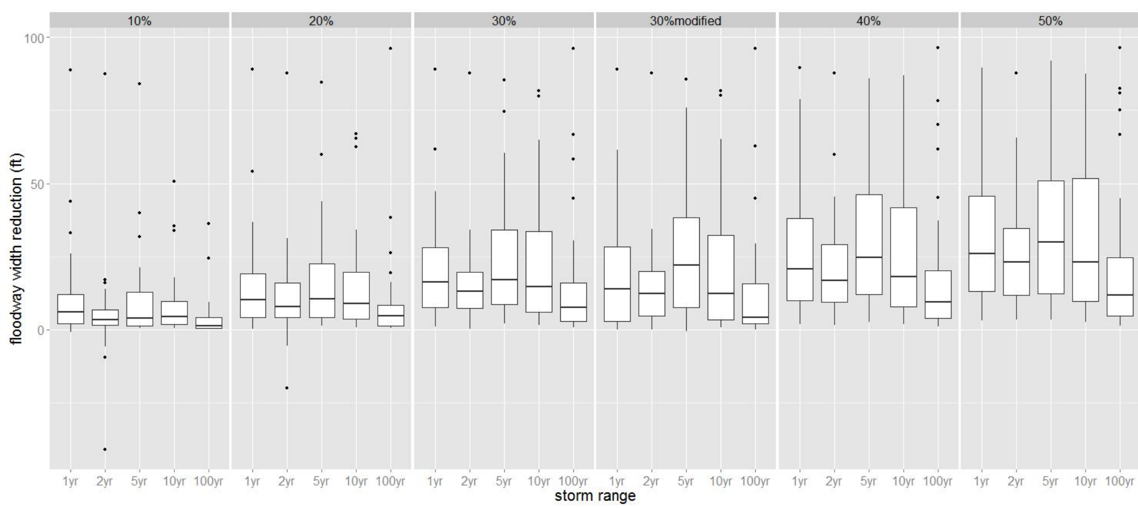

Figure 8. Comparing the influence of imperviousness disconnection on floodway width during different storm events

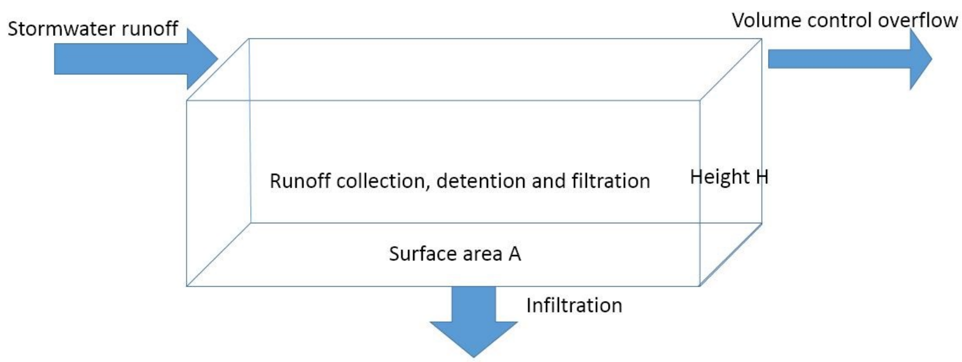

Figure 9. Simplified infiltration GI practices box model

Figure 10. Site example commercial parking lot

Figure 11. Site example Langtree elementary school

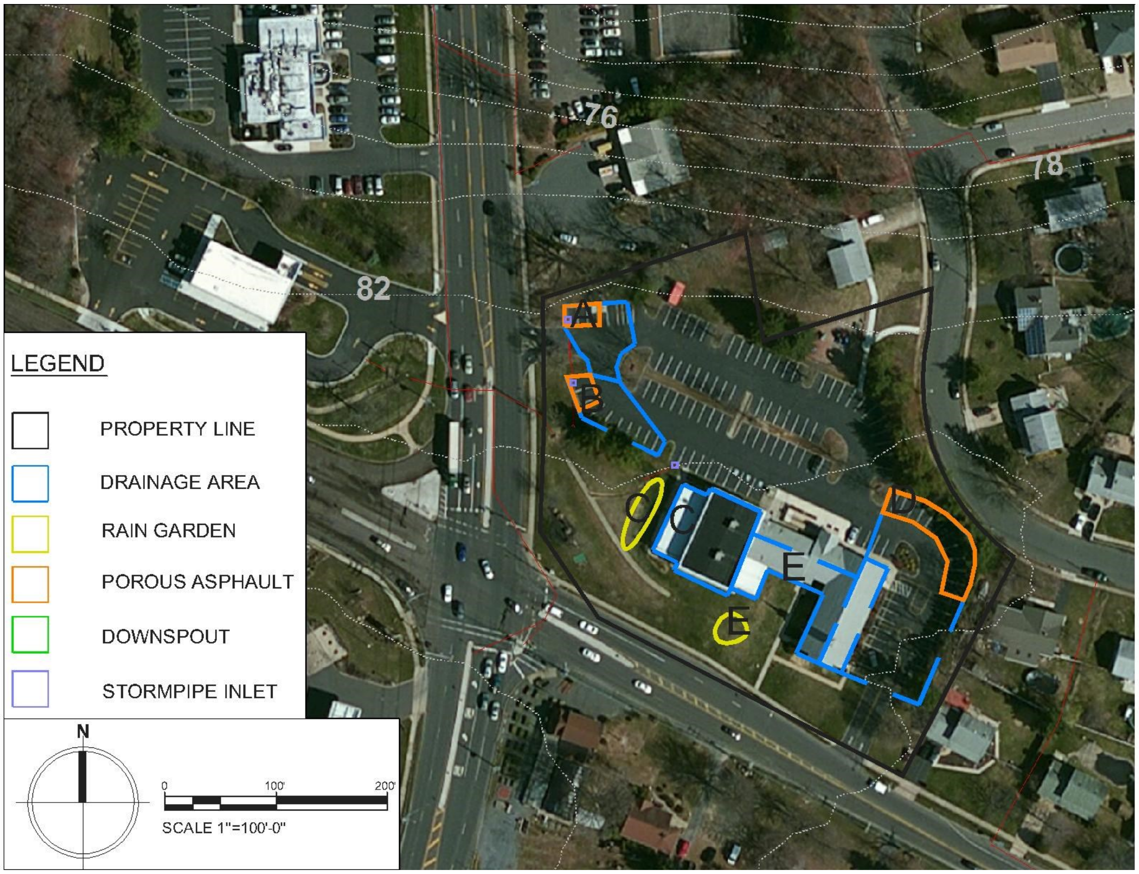

Figure 12. Site example Graceway Bible Church

Figure 13. Site example residential area