Vegetated Swales in Urban Stormwater Modeling and Management

Kyle W. White

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Randel L. Dymond, Chair

Glenn E. Moglen

Kevin D. Young

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ABSTRACT

Despite the runoff reduction efficiencies recommended by various regulatory agencies, minimal research exists regarding the ability of vegetated swales to simultaneously convey and reduce runoff. This study assessed the effect water quality swales distributed among upstream sub-watersheds had on watershed hydrology. The study was also posed to determine how certain design parameters can be dimensioned to increase runoff reduction according to the following modeling scenarios: base, base check dam height, minimum check dam height, maximum check dam height, minimum infiltration rate, maximum infiltration rate, minimum Manning's n, maximum Manning's n, minimum longitudinal slope, and maximum longitudinal slope. Peak flow rate, volume, and time to peak for each scenario were compared to the watershed's existing and predevelopment conditions. With respect to the existing condition, peak flow rate and volume decreased for all scenarios, and the time to peak decreased for most scenarios; the counterintuitive nature of this result was attributed to software error. Overall, the sensitivity analysis produced results contrary to the hypotheses in most cases. The cause of this result can likely be attributed to the vegetated swale design and modeling approaches producing an over designed, under constrained, and/or over discretized stormwater management practice.

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1 Introduction

1.1 Background

Traditional stormwater management focuses on routing excess rainfall away from a developed site, storing it, and conveying it downstream at a reduced flow rate. Sites are typically graded in a manner that promotes sheet flow into "hard" conveyance structures such as gutters and pipes. Conveyances are connected by various node types such as catch basins, manholes, sub-surface junctions, and outfalls – collectively these elements makes up a traditional storm sewer system. The storage devices integrated within these systems are known as Best Management Practices (BMPs). Traditional design objectives include detaining runoff volume and reducing the outflow rate by means of a control structure. At the site scale, this strategy enhances public safety by conveying stormwater off site as quickly as possible. Downstream reaches, however, may experience greater surface flow, less subsurface flow, and reduced water quality.

Low Impact Development (LID) is a collection of practices that seek to minimize the negative hydrological and ecological impacts on the environment caused by the increase in imperviousness resulting from construction. The primary goal of LID from a hydrologic perspective is to match the post development runoff hydrograph to that of the predevelopment condition. The best way to meet this objective is to implement practices which enhance natural hydrologic processes such as infiltration, filtration, evapotranspiration, and biological uptake. Several relatively new BMPs featuring such mechanisms include green roofs, bioretention, rain gardens, rooftop disconnection, and vegetated swales. Considering the novelty of these practices, further investigation is needed to determine their usefulness in mimicking predevelopment hydrologic patterns.

One particular LID practice is designed to minimize the negative hydrologic effects of land development while also maintaining capacity for stormwater conveyance. This practice is generally referred to as a vegetated swale, but variations include grass channels, dry, and wet swales. Each adaptation varies according to runoff reduction potential as well as the implementation constraints and strategies. Generally, vegetated swales are sited to treat sheet flow from roadways or parking lots, but they are also capable of receiving point source discharge from pipe outfalls. Post-development runoff attenuation is realized according to the following hydrologic processes: ponding, infiltration, filtration, and biological uptake. Swales mainly serve as on-line treatment practices, meaning they receive runoff as it follows the primary flow path through a drainage basin. These vegetated swales function most efficiently over time when equipped with pretreatment such as vegetated filter strips, pea gravel diaphragms, or extended side slopes. Pretreatment is intended to regulate the total suspended solids (TSS) entering the swale, which clog soil pores and reduce permeability. Check dams are an optional design feature used to enhance treatment efficiency by means of inducing ponding resulting in an increased residence time. The increase in ponding promotes infiltration through another design variation called engineered soil media. Soil amendment depths range from 18-36 inches beneath the swale bed, and consist of a mixture of sand and clay fines. The soil mixture is underlain by a gravel layer containing a perforated pipe. The pipe collects flow after it percolates through the soil matrix and discharges to downstream reaches of the sewer system.

While vegetated swales are an enticing BMP due to their ability to simultaneously treat and convey runoff, they can prove to be costly. Excavation, media installation, and routine maintenance operations are expensive activities required to implement and sustain functionality. Deciding to increase the stormwater management budget to improve water quality via LID practices such as vegetated swales is a current challenge facing municipalities. Careful consideration must be taken to determine the LID practice(s) offering the most financially efficient treatment, as well as the design approach that optimizes their performance.

A hydrologic model provides localities with a way to analyze the effectiveness of LID on a watershed. A reliable hydrologic model of an existing storm sewer can provide invaluable information such as the system's response to various return frequency events, pollutant loads, and proposed stormwater management strategies. Developing a reliable model requires sound input data, the likes of which offer an accurate representation of the storm sewer elements, land cover, and soil composition. While this information theoretically leads to accurate results, the best means of confirming a model's validity is to perform a calibration using observed flow data. Calibration involves recording rainfall and gauging subsequent flow at the system's outfall. In order to test the model's accuracy, its response to the same rainfall data is compared to the observed flow. Once the model performs to a desired level of accuracy, only then may it be considered reliable enough for forecasting.

Hydrologic models should be used to assess the effectiveness of vegetated swales on a watershed scale (approximately 380 acres). While field and laboratory studies indicate that vegetated swales are a viable pollutant removal and runoff reduction practice, a void exists with respect to studies assessing the attenuation capabilities of water quality swales designed for conveyance. The results from a study of this nature would provide insight into the plausibility of designing swales for simultaneous treatment and conveyance purposes. Positive results would suggest that vegetated swales can be used as a means of storm sewer retro-fit or as the primary treatment and conveyance practice for a new development. This study involves various design approaches in order to assess the runoff reduction potential of a vegetated swale at a watershed scale.

1.2 Problem Statement

The primary focus of this study is to determine the vegetated swale design parameters most effective at reducing runoff. The study incorporates a hydrologic model of an urban watershed in order to evaluate the effect various parameters have on a swale's available surface storage and the resulting runoff attenuation. The parameters are varied through a sensitivity analysis which includes minimum and maximum values for check dam height, infiltration rate, Manning's n, and longitudinal slope. Practical application is also evaluated according to the following constraints: 1) only true sub-basins receive treatment and 2) only swales occupying 10% or less of the total drainage area are implemented. The overall results are intended to provide insight into the design approach and siting strategies appropriate for vegetated swale runoff attenuation. Furthermore, the results can be used to assess the feasibility of vegetated swales as a means of storm sewer retro-fit, or as the primary runoff treatment and conveyance mechanism for new developments.

1.3 Objectives

The study includes three main objectives, outlined as follows:

1. *Investigate vegetated swale design and runoff reduction potential.*

In order to properly design and implement vegetated swales throughout a watershed, siting techniques and parameters relevant to runoff reduction must be analyzed. The cross-sectional dimensions for swales adhering to a rate based design, like the ones in this study, are most greatly affected by Manning's n and longitudinal slope. Also, parameters such as check dam height and infiltration rate have an influence on the storage capacity within a vegetated, open channel.

2. *Design a sensitivity analysis to assess the effect of vegetated swale parameters on design practicality and runoff attenuation.*

Parameters considered influential to swale efficiency (runoff reduction per swale unit surface area) are chosen for variation in a sensitivity analysis. As these parameters are varied, the resulting swale surface area is affected. The designed swales are applied to true sub-basins in a hydrologic model only if their surface area is less than or equal to 10% of the contributing drainage area. Parameters assessed include check dam height, infiltration rate, Manning's n, and longitudinal slope. Increasing check dam height, infiltration rate, and Manning's n is expected to result in the most efficient swales, while decreasing longitudinal slope is also expected to improve swale efficiency.

3. *Recommend the vegetated swale parameters which result in a practical swale capable of runoff attenuation.*

The study plans to show that with careful consideration of design parameters, vegetated swales can be a practical LID practice in at least a partial storm sewer retro-fit or new development conveyance and treatment application. The recommendation will focus upon the design scenario(s) resulting in swales offering the greatest runoff reduction for the least amount of disturbed area.

1.4 Data Development

The study data is the result of a field survey performed by Dr. Dymond's research group within the Via Department of Civil and Environmental Engineering at Virginia Tech. The survey was conducted for an approximately 380 acre watershed in the Town of Blacksburg, Virginia, with the purpose of recording attributes of its storm sewer nodes and conveyances. The study watershed is illustrated in Figure 1.1; it consists of residential (72%), institutional/community (17%), open space (8%), public assembly (2%), and commercial/office/industrial (1%) land uses. The field survey includes data sufficient for mapping, linking, and dimensioning the study watershed's storm sewer system.

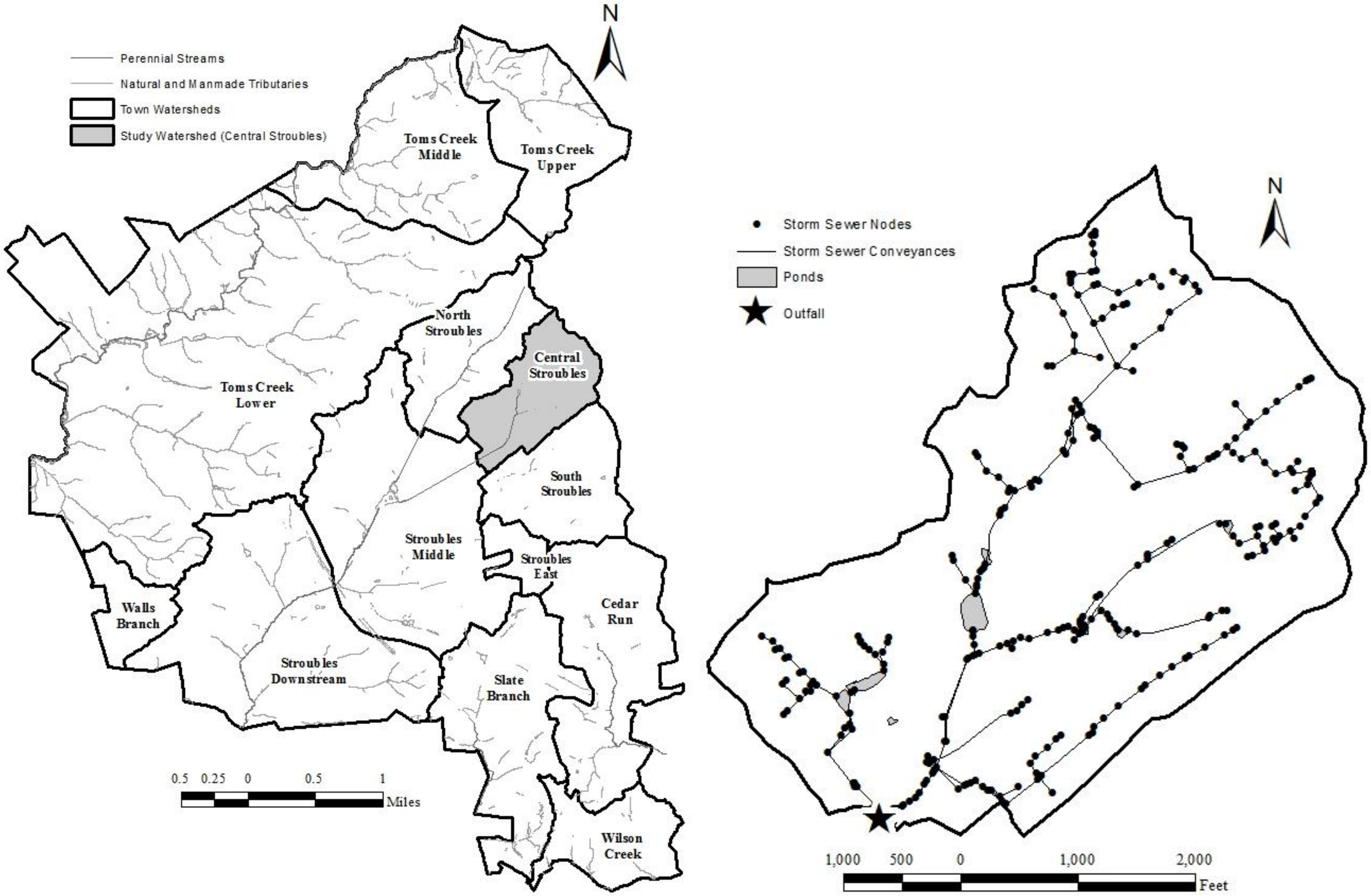


Figure 1.1: Town of Blacksburg Watersheds (left) and Study Watershed (right)

The survey was performed by mapping and recording attributes for storm sewer nodes such as catch basins, manholes, underground junctions, control structures, and outfalls with a handheld global positioning system (GPS) device. Due to their intricacies, pond outlet structures required a more extensive survey noting weir and orifice inverts and dimensions. Data such as pond rating curves were extracted from as-built surveys provided by the town. The attributes recorded using ESRI's ArcPad software included node type, number of conveyances, conveyance shape, material, invert depth, and inflow/outflow azimuth. Data collected in the field were downloaded and compiled in a geospatial information system (GIS) database.

One component of the storm sewer network that was derived from the field survey was open channel conveyances. These elements were more easily developed using a digital elevation model derived from LiDAR data provided by the Town. Channel center lines were delineated across the resulting surface, and three-dimensional analyst tools in ESRI's ArcMap software were used to create a minimum of ten cross-sections per channel. Each cross-section was analyzed for extraneous points, which were subsequently eliminated. An average shape was chosen for each channel and applied to the upstream and downstream nodes. Inverts were adjusted according to channel length and slope.

The final aspect of field data development involved using the information collected for each node as well as the watershed's topography to create a connected storm sewer network. The first step was to determine which nodes were connected; an objective accomplished using conveyance azimuths. A GIS program, or script, was developed to establish a short line in the direction of the measured azimuths. These lines, containing attributes such as conveyance type, shape, diameter, and material, were extended and connected to the next nearest node in the azimuth direction. Another script was developed which assigned invert and rim elevations to nodes based on the LiDAR data. Upstream and downstream nodes for a conveyance were defined using a unique method. Slight variations in LiDAR data did not allow for upstream and downstream nodes to be assigned via invert elevations. Therefore, another script was developed requiring that the outflow node be defined for the entire watershed. With the script assuming gravity flow downstream to the outlet, upstream and downstream nodes were defined for each conveyance working upstream from the overall outlet. At this point, the field data were completely prepared to be transferred into a modeling environment.

1.5 Model Development

The modeling software (Bentley, 2010) chosen for this study included benefits such as smooth integration of GIS data, automatic generation of control structure outflow curves, and the ability to use the Natural Resources Conservation Service (NRCS) TR-55 unit hydrograph method (USDA, 1986). This method is preferable due to the simplicity of the time of concentration and NRCS curve number (CN) inputs which are an effective means of representing the physical make-up of a watershed. Although further delineation and data analysis were required, these inputs were readily derived from LiDAR, aerial imagery, soils data, and land cover.

Developing the model inputs began with sub-basin (catchment) delineation. While sub-watershed discretization can theoretically be reduced to a small enough scale to produce a drainage area for each node, this would be an extremely arduous task. Research has shown, however, that sub-watershed delineation scale has a minimal effect on hydrologic model results (Elliot, et al., 2009), thus it was decided that catchment scale was irrelevant to this study. There was a concerted effort, however, to treat ponds as outlets and to cordon off areas of common land use into separate catchments. This approach was taken to create homogeneity amongst sub-basins, making the NRCS CN as accurate as possible for areas within a catchment.

Catchments were assigned an NRCS CN by combining land cover and soil data. A study compared the National Land Cover Dataset (NLCD) to digitized land cover by assessing their effects on curve number estimation. The resulting CN differences were deemed insignificant (CGIT, 2008), however, due to its availability and level of detail, the digitized land cover was chosen for CN development in this study. Land cover in the detailed data set was digitized using aerial imagery and the following 11 land cover classifications: road/parking, sidewalks, buildings, other asphalt/concrete, gravel, open space, dirt, light forest/tree canopy, dense forest, light bush/mulch, and brush/bush. The soils data used in CN development were derived from the Soil Survey Geographic Database (SSURGO) (USDA 2009). The detailed land cover, SSURGO data, and TR-55 CN tables were combined to develop a catchment area-weighted CN.

Next, the time of concentration was calculated for each catchment through a combination of "longest flow path" delineation and overland flow equations. The surface produced from the LiDAR data was used in concert with the aerial imagery and surveyed infrastructure to delineate several "longest flow paths" for each catchment. The three-dimensional analyst tools in ArcMap were used to calculate the slope for each path. The equations in Table 1.1 were used to compute the travel time along each path, and the one with the greatest total travel time provided the time of concentration.

Table 1.1: Time of Concentration Equations [adapted from (James, 2010)]

Flow Type	Surface	Source	Equation
Sheet	Unpaved	(Seelye, 1945)	$t_{t,sheet} = 0.225 l^{0.42} S^{-0.19} 0.3^{-1.0}$
	Paved		$t_{t,sheet} = 0.225 l^{0.42} S^{-0.19} 0.9^{-1.0}$
Shallow	Unpaved	(USDA, 1986)	$t_{t,shallow} = \frac{l}{16.1345 S^{0.5} \times 60}$
	Paved		$t_{t,shallow} = \frac{l}{20.3282 S^{0.5} \times 60}$
Channel	Unpaved	(Kirpich, 1940)	$t_{t,channel} = 0.00948(H - L)^{-0.38} l^{1.13}$
	Paved		$t_{t,channel} = 0.2[0.00948(H - L)^{-0.38} l^{1.13}]$

$$t_c = t_{t,sheet} + t_{t,shallow} + t_{t,channel}$$

t_t = travel time (min); t_c = time of concentration (min); l = length (ft); S = longitudinal slope (ft/ft); H = highest elevation along segment (ft); L = lowest elevation along segment (ft)

2 Literature Review

2.1 Low Impact Development

Implementation Strategies

When considering LID as a means of runoff mitigation, importance must be placed on the appropriate design and location of stormwater management practice(s). LID practices are an effective way of replicating natural hydrologic patterns as well as retaining pollutants. While it was thought that design techniques were the only important considerations for LID applications in cold climates or tight soils, these factors are actually important in many other situations. LID practices predicated upon infiltration may not be an appropriate treatment mechanism for drainage areas which contribute high contaminant loads. Sites such as recycling centers, gas stations, and brownfields draining to an infiltration BMP can contaminate groundwater. Insuring that a practice has an overall positive effect on effluent quality is an evident goal, but not always a definitive result of LID. The composition of the facility itself can be detrimental to downstream surface waters; according to one study, bioretention cells and green roofs can discharge high concentrations of phosphorus due to their soil media composition (Dietz, 2007).

In addition to the drainage basin's land use, the location of the BMP within the watershed is equally, if not more important to the practice's water quality performance. A spatial analysis performed on BMPs to determine the location which optimizes peak discharge and runoff volume control resulted in guiding principles for BMP citing (Gilroy and McCuen, 2009). Bioretention facilities were discovered to be less effective when sited to treat pervious areas than impervious areas; the reason being that the grassed areas throughout pervious areas partially reduce runoff rates and volume. Another claim made was that BMP storage volume has a variable effect on peak discharge and runoff volume, meaning the BMP should be designed according to the parameter needing to be controlled. Effluent volumes and peak flow rates were shown to be unresponsive to spatial separation of BMPs sited along the same flow path. Ultimately, however, the study concluded that the ability of a BMP to reduce runoff volume and peak flow rate is severely hampered by improper location.

As is the case with all cost minimization exercises, consideration must be given to lifecycle costs. When considering the cost effectiveness of an LID practice, it is important to not only evaluate capital, but operation and maintenance costs as well (Olson, et al., 2009). The maintenance aspect of a BMP's lifecycle is an import, but often overlooked aspect of LID cost assessment. Surveys performed in 1986 and 1990 in Maryland to determine how the effectiveness of infiltration practices varied over time involved evaluating the performance of relatively new facilities (two years old or younger in 1986) (Lindsey, et al., 1992). The practice was deemed effective if water had completely drained within 72 hours of a rainfall event. Practice derivations included infiltration basins, infiltration trenches, dry wells, porous pavement, and vegetated swales. Of the 177 BMPs inspected in 1986 and 1990, 42% failed in both surveys, while 24% that were functioning properly in 1986 failed in 1990. It was initially suspected that insufficient construction techniques lead to the failures in 1986, however, the increase in failure rate after a duration twice as long indicated that the issues were due to insufficient maintenance. When stormwater ordinances came into effect in 1984, minimum standards called for BMP inspection at least once every three years. An apparent, but obvious assertion that the researcher made was the more effective an infiltration practice is at removing contaminants, the more inspection and maintenance attention it should receive.

Performance Assessment

Often LID practices require a commitment from numerous stakeholders, so it is important to have the ability to quantify and present expected long-term benefits. In a study assessing the effectiveness of water quality BMPs (Strecker, et al., 2001), statistical analyses of BMPs showed that percentage removal is not a useful means for judging a practice's water quality efficiency, unless only highly contaminated sites are examined. Part of the theory is that if a BMP is structured to remove a certain percentage of pollutants, then regulations tailored to the reducing pollutant loading at its source will go by the way side. Some BMPs may have been classified as less effective than they actually are due to cleaner influent. A study supporting effluent quality as the best means of BMP treatment efficiency evaluation incorporated a conceptual model representing the storage/infiltration mixing zone in a BMP using the mass balance equation for a series of completely mixed flow reactors (CMFRs) (Wild and Davis, 2009). The study found that variability in pollutant removal performance can be attributed to fluctuating distributions of non-steady-state hydrologic inputs in addition to exponentially decaying influent contaminant loads. The researchers suggested that in reality, variable hydrographs and pollutant load distributions will provide the influent for LID practices, and for those reasons, statistical analyses should be used to judge BMP performance instead of a simple metric like percent removal. Percent removal implies that a BMP should function at the same level of efficiency regardless of the inflow hydrograph and pollutant load. If further data are collected on a BMP's response to a wide distribution of flows and pollutant concentrations and their corresponding response, then future performance can be assessed in terms of statistical significance.

Ultimately, regardless of the type of BMP or the scale at which it is implemented, all practices can be described by a retained storage volume and a rate of recovery from this volume. This generalized approach to BMP characterization provides a way to normalize BMPs, regardless of its treatment mechanisms, for across the board comparison (Clary, et al., 2010).

Storm Sewer Retro-Fit

One method of improving the runoff quality from a watershed to downstream surface water bodies is by retro-fitting the existing storm sewer with LID practices. A study comparing the impacts of traditional stormwater management to LID practices on a watershed under pre- and post-development conditions was performed in Waterford, Connecticut (Bedan and Clausen, 2009). One site was classified as traditional, as it incorporated standard road widths as well as a curb and gutter drainage system. The other site, classified as LID, replaced curbs and gutters with grassed bioretention swales, narrowed roadways, substituted concrete pavers for asphalt, implemented bioretention cul-de-sacs, and equipped individual lots with small bioretention areas (rain gardens). The LID watershed produced a decrease in mean flow depth of 42% for the post-development conditions. Mean peak discharge increased 30 times in the traditional watershed, while showing a decrease in the LID development during post-construction. Overall, the increased runoff from the traditional practices contained higher levels of nitrate and nitrite, ammonia, Total Kjeldahl Nitrogen (TKN), total phosphorus, copper, and zinc. Post-construction, the LID watershed effluent contained greatly decreased concentrations of lead and zinc, however, TSS and total phosphorus (TP) levels were elevated.

The same LID versus traditional stormwater management study in Waterford, Connecticut was further analyzed using regression significance testing (R^2 calculations) to correlate impervious area to various pollutant discharges. The analyses concluded that as imperviousness increased in a watershed, annual stormwater runoff volume increased at an exponential rate. Also, as impervious cover increased in the LID watershed, stormwater runoff volume remained the same. Pollutant discharge from the traditional practices resembled that of an urbanized watershed, while the LID development was comparable to the effluent from forested land cover (Dietz and Clausen, 2008).

A different study observed discharge for two perforated pipe swale systems and one conventionally piped storm sewer. Rainfall events less than 12 mm (0.47 in) produced no discharge 9% discharge relative to rainfall across the two swales, while the conventional system produced 23% discharge relative to rainfall. When considering all rainfall events, the swale systems produced an average of 9%, while the conventional system functioned at 40% (Abida and Sabourin, 2006).

2.2 Hydrologic Modeling

Approaches

Prior to implementing a hydrologic model for flood forecasting or BMP evaluation, it is important to understand the software and methods used to execute the hydrologic and hydraulic calculations. The need for hydrologic modeling is a product of Total Maximum Daily Limits (TMDL) for various pollutants found in stormwater runoff. In a review of hydrologic modeling software incorporating LID practices (Daniel, et al., 2011), it was determined that the majority of software provides insufficient functionality for comprehensive BMP assessment. Such exclusions consist of the contribution of sediment from erosion in streams, as well as the contaminant storage/release cycle in sediments. Another notable exclusion in most software environments was the contribution of baseflow to contaminant concentrations (Elliott and Trowsdale, 2007).

Hydrologic models can be empirically-based, meaning functions (equations) are used to produce results representing the best fit of a system, or physically-based, where mass transfer, momentum, and energy are represented by partial differential equations. In terms of spatial modeling, watersheds can be lumped, semi-distributed, or distributed, where parameters are completely averaged into a single unit, partially averaged into a single unit, or sub-divided among multiple basins respectively. These equations are solved by numerical methods such as the St. Venant equations for surface flow, the Richards equation for unsaturated zone flow, the Penman-Monteith equation for evapotranspiration, and the Boussinesq equation for groundwater flow. A recent addition to hydrologic modeling is a statistical approach called Artificial Neural Networks (ANNs), which are programs capable of learning from previous information. These programs are composed of vast amounts of nodes, networks, and relationships, similar to a biological nervous system (Daniel, Camp, LeBoeuf, Penrod, Dobbins and Abkowitz, 2011).

A decision that stormwater modelers must make is the scale at which the model and its sub-watersheds are to be delineated. A study involving the aggregation of sub-watersheds in a drainage basin showed that catchment delineation scale is insignificant to mean flow, baseflow, and water quality results for a dynamic, hydrologic model (Elliot, Trowsdale and Wadhwa, 2009). Aggregation to a single catchment was found to have a significant impact on peak flow rate, however, and the reason can likely be attributed to aggregation of the conveyance network rather than the sub-watersheds. A similar analysis (Chang, 2009) evaluating the effect of sub-watershed scaling concluded that while catchment delineation scale is relatively insignificant to modeling results, the number of drainage basins used to represent a watershed should be based upon the watershed properties (ie. land cover, concentration of conveyances in the drainage network, detention facilities, topography, etc.). In addition, this study concluded that the relative error for runoff prediction gradually increased as the number of sub-divisions decreased.

Inputs and Data

A basic principle states that a model's forecasting performance will only be as accurate as the data used to develop it. For this reason, modelers should understand the methods behind data development as well as the influence input variability has on the model's results.

Precipitation, flow, land cover and other physical data are limited over the span of an entire watershed, thus limiting the model's accuracy. Also, a sensitivity analysis should be conducted to determine the appropriate number of parameters or variables used to represent a system (Daniel, Camp, LeBoeuf, Penrod, Dobbins and Abkowitz, 2011). In terms of data detail, higher resolution (increased level of detail) is optimal when peak runoff is the parameter of interest, while lower resolution (lower level of detail) can be used if the model's purpose is to assess sediment concentration in the effluent (Kalin, et al., 2003).

A study of time of concentration flow path discretization showed that as the number of segments used to delineate the longest flow path increased, the resulting time of concentration provided an overestimate of the actual time. In addition, the single segment longest flow path approach provided an underestimate of the actual time (Pavlovic and Moglen, 2008). Another time of concentration study (McCuen, 2009) assessed calculation accuracy for instances where the watershed's principal flow path, or the path extending from the watershed outlet to the drainage divide, was used to determine time of concentration. Results showed that time of concentration was consistently under predicted when calculations incorporated the principal flow path. Reasons for the discrepancy included greater resistance through flow paths incorporating storage as well as underestimation of sheet flow lengths.

Applications

Engineers and localities considering hydrologic models should take into account the scenarios in which such models have been applied and the conclusions that have resulted. One modeling study (Yang and Li, 2011) evaluated the effects of soil permeability, development density, and development location on curve numbers and runoff. Compared to the forested condition, runoff increased by 35% and 85% among high- and low-density developments respectively. Also, the sediment concentration in effluent increased by 30% and 80% for high- and low-density development respectively. While development density proved to have a greater impact on a watershed's runoff hydrograph than the development's proximity to the outlet, the most comprehensive planning approach would be to consider both the site's density and proximity to the outlet.

Another modeling study (Schneider and McCuen, 2004) designed to develop a procedure for determining the amount and type of rainfall data required to validate a BMP determined that cisterns required four years' worth of rainfall data to be validated. The researcher noted that while four years may seem like an excessive amount of data for a BMP as simple as a cistern, it is often necessary to acquire up to 25 years of data to develop peak discharge regression equations.

2.3 Vegetated Swales

Expected Performance

The most accurate hydrologic models designed to forecast BMP performance are calibrated using observed results from a physical representation of the practice. The study presented in this thesis is greatly influenced by the vegetated swale design parameters and approaches outlined below.

In the Virginia Department of Conservation and Recreation (DCR) specifications for dry swale design, the level 1 design (18-36 inch media depth, less than or equal to 2% slope, and underdrain conveyance) is reportedly capable of reducing 40% of annual runoff volume, while 60% efficiency is reported for the level 2 design (24-36 inches, less than 1% slope, and underdrain if necessary) (VADCR, 2011). Swales constructed with underdrains in Austin, Texas were tested for their ability to remove pollutants via infiltration. The total runoff reduction realized was 90% (Barrett, et al., 1997). Similarly, a biofilter (dry swale) retro-fit study resulted in 30-80% runoff reduction due to infiltration among eight storm events sampled (Schueler and Holland, 2000). A swale equipped with a grass filter strip pretreatment area used to treat highway runoff in Maryland was studied for 22 rainfall events over a 1.5 year period. The swale resulted in a range of 46-54% runoff volume reduction over the study period (Stagge, 2006). Further studies assessing the runoff reduction potential of vegetated swales produced the following results: 40% (average) (Strecker, et al., 2004), 50% (drainage swales, semi-arid climate with permeable soils and low initial moisture content) (Barrett, 2005), and 33-80% (contributing drainage area imperviousness of 90-95%) (CALTRANS, 2004).

A study (Liptan and Murase, 2000) included two identically sized swales with identical soils, one consisting of grass turf and the other of native grass, were subjected to runoff volumes from a 50 acre urban area. The study involved six events over a two year period, and the turf swale was mowed regularly, while the other swale contained native vegetation allowed to grow naturally. The native grassed swale produced runoff reduction of 41% and the turf swale performed at 27%. A similar comparison based study (Schueler and Brown, 2004) paralleled the performance of a wet and dry swale sited to treat highway runoff. The wet swale was vegetated and intercepted groundwater flow, while the dry swale had sparse grass cover and a relatively high infiltration rate. The dry swale reduced runoff by 80%, and performed better than the wet swale overall.

The study (Schueler and Holland, 2000) also evaluated three swales in different locations, each of approximately equal length but consisting of different longitudinal slope, vegetated cover, and soils. The swale located in Florida resulted in the best pollutant removal, and it consisted of sandy soils, high grass, and a relatively flat slope. The Maryland swale used short grass and a moderate slope (3.2%), but the bed media eroded causing the swale to export sediment and remove fewer pollutants. The swale sited in Virginia removed a moderate level of pollutants, and it consisted of the steepest slope (4.7%), better grass cover, and minor erosion. The study indicated that higher and denser grass cover, flatter slopes, and more permeable soils are contributors to greater pollutant removal performance. The researchers suggested enhancing swale design through sand layers, check dams, and underdrains since slope and soil type cannot always be controlled.

Other vegetated field studies (Barber, et al., 2003) have been performed to assess total runoff attenuation parameters, including peak flow rate, volume, and time to peak (or lag time). A physical model of a v-shaped ditch was subjected to a Type I-A Soil Conservation Survey (SCS) 24-hour design rainfall event in order to measure the peak flow and peak delay time reductions. Peak flow and delay time decayed exponentially as a function of storm size. Peak flow reduction was found to be a function of the soil media, input hydrograph distribution, and ditch size, while the time to peak was dependent upon soil media and ditch size. The recommended upper layer is sand, with a saturated hydraulic conductivity between 0.03-0.08 centimeters per second (42.5-113 in/hr), and coarse gravel for the pipe bedding. The cross-sectional ditch dimensions had no significant impact on the effluent hydrographs, however, shallower side slopes improved peak flow rate reductions modestly. A separate BMP study (Davis, 2008) evaluated outflow for three existing bioretention cells over the course of 49 runoff producing rainfall events. The storms ranged from 0.044-8.1 m^3/m^2 (runoff volume per unit bioretention surface area), and the resulting average runoff flow rate was equal to 0.021 $m^3/h/m^2$, or 2.1 cm/h. More specifically, 18% of events resulted in no effluent from the underdrain, and for the events producing outflow, peaks were reduced between 44-63%. Also, times to peak were delayed by a factor of two or more.

Further studies investigated individual vegetated swale design parameters and their effects on runoff attenuation. In one field study (Abida and Sabourin, 2006), total rainfall and the resulting discharge was observed for two perforated pipe swale systems and one conventionally piped storm sewer. The same study involved infiltration tests using the constant head method for five separate grassed swales in Ontario, Canada. The soils at each site were sandy silt, and infiltration rates varied from three to 13 centimeters per hour (1.2-5.1 in/hr). After a 15-20 minute testing period, constant infiltration rates of one to three centimeters per hour were observed (0.4-1.2 in/hr). The study suggested a conservative rate of one centimeter per hour (0.4 in/hr) be used for similar soil conditions due to the infiltration rate degradation over time.

In a laboratory study (Claytor and Schueler, 1996) evaluating hydraulic resistance of various types of grasses commonly used for swale bed cover, Manning's n ranges were derived for Centipede, Kentucky Bluegrass, and Zoysia (average densities of 6,253, 5,019, and 4,772 stems/m^2 respectively). The Manning's n values fell within 0.27-0.95 (flow depth: 27-50 mm), 0.26-0.56 (flow depth: 24-60 mm), and 0.28-1.35 (flow depth: 24-60 mm) respectively. The research also concluded that Manning's equation for open channel flow should be strictly limited to turbulent flows (Kirby, et al., 2004). Another aspect of hydraulic resistance to consider is that the Manning's n roughness coefficient varies with flow depth. For shallow depths of approximately four inches or less, where vegetation height is equal to flow depth, n is equal to 0.15. When flow depth increases above four inches, n decreases linearly to a value of 0.03 at a depth of 12 inches (for grass cover).

An example of vegetated swale modeling (Ackerman and Stein, 2008) includes a calibrated, continuous model involving the hydrologic simulation program-Fortran (HSPF), used to study the sensitivity of design parameters on BMP performance. One of the modeled BMPs was a dry swale, designed with 3:1 side slopes, 0.5% longitudinal slope, 400 foot length, 6 foot bottom width, and a Manning's n of 0.15. For all of the BMPs modeled, volumetric reductions decreased as storm size increased over the nearly ten year period. The swale's peak flow reduction decreased linearly as storm size increased. In the sensitivity analysis, the swale was most sensitive to volumetric changes (width, length, and infiltration rate). The effectiveness of other, retention based BMPs tested greatly varied according to the infiltration rate of underlying soils. Design parameter values derived from literature must only be applied to comparable antecedent conditions. Also, the swale results were based upon runoff from a one acre catchment and siting adjacent to roads, parking lots, and other similar impervious areas. The greater conclusion from the study was that since swales and bioretention are most sensitive to variation in their water balance, objectives relating to design and performance must be determined prior to BMP implementation.

Design and Siting

Finally, prior to implementing a BMP, optimal design and siting techniques should be investigated. In a study (Palhegyi, 2010) performed to create curves of required BMP surface area (as a percentage of the total drainage area) versus the percentage of imperviousness to be treated, researchers were able to apply the resulting methodology to size bioretention and a one foot deep infiltration swale. The infiltration swale required the most surface area per unit impervious area. The study concluded that area requirements can be proportionally reduced if facilities are designed to be deeper, or if LID is combined with basin storage.

Virginia Department of Conservation and Recreation (VADCR) requires that dry swales be non-erosive for the 2- and 10-year design storms, while maintaining adequate capacity to pass the 10-year event. Dry swales are applicable on all Hydrologic Soil Group (HSG) types, however, groups C and D require an underdrain. The contributing drainage area should be five acres or less, because larger areas typically create excessive channel velocities. Check dams can be used to dissipate energy, however, appropriate downstream toe protection is required (VADCR, 2011). The VADCR minimum standard for grassed and water quality swales states that grassed swales are applicable in low- to moderate-density (16-21% imperviousness) developments, while water quality swales can be used to treat higher densities (16-37% imperviousness). The standard specifies that vegetation used should be water-tolerant, deeply rooted to prevent scouring, and have the ability to recover following inundation. Check dams should be constructed from non-erosive materials such as water-resistant wood or gabion baskets (VADCR, 1999).

2.4 Summary

Given careful consideration for implementation and design, LID practices can be an effective means of pollutant removal and runoff attenuation. This approach to stormwater management, however, is not sustainable without placing an emphasis on facility inspection and maintenance. The modeling and field studies conducted on LID BMPs, such as bioretention and vegetated swales, support their use in practice. With these BMPs, and all LID practices for that matter, it is important to consider the conditions for which they will be applied as well as the design parameters serving to enhance their performance. Hydrologic modeling provides a means of testing and predicting LID attenuation and contaminant removal. This method of performance assessment is precluded by non-comprehensive data and model development.

3 Vegetated Swales in Urban Stormwater Modeling and Management

3.1 Introduction

The primary goal of Low Impact Development (LID) is to minimize the negative environmental effects associated with post-development runoff quantity and quality by preserving the predevelopment conditions. This goal is typically achieved by replicating natural processes such as infiltration, filtration, evaporation, and biological uptake. Such processes reduce runoff volumes, peak flow rates, and pollutant concentrations. LID is becoming an increasingly popular means of adhering to the most current stormwater management objectives which include runoff volume, peak flow rate, and pollutant load reduction. One practice addressing each of these goals is the vegetated swale. While traditional swales were considered a superficial means of stormwater conveyance, vegetated swales can be equipped with an engineered soil matrix to induce infiltration, vegetation for biological pollutant uptake and settling, and check dams to decrease velocity and promote infiltration. The combination of its traditional and contemporary uses results in a means of addressing water quality concerns without the need for additional infrastructure. Prior to siting swales throughout a watershed to convey and attenuate flow, consideration must be given to the following: 1) practical implementation due to the erosive potential of concentrated stormwater flows on natural surfaces as well as the surface area required for construction, and 2) dimensions of the various design parameters affecting flow attenuation.

Several studies have investigated the effect of parameters significant to swale design on runoff reduction. Vegetated swales covered by natural vegetation reduced runoff by approximately 15% more than a swale covered by mowed turf (Liptan and Murase, 2000). With respect to soils and longitudinal slope, a study comparing vegetated swales located in different areas of the United States found that the swale with the flattest slope and sandy soils performed best on the basis of pollutant removal (Schueler and Holland, 2000). Cross-sectional dimensions are an aspect of swale design deemed relatively insignificant in terms of runoff reduction (Barber, King, Yonge and Hathhorn, 2003). The Manning's n roughness coefficient which applies to open channel flow must also be considered in vegetated swale design. The coefficient is variable with depth, decreasing linearly from the swale bed to the top of bank in channel with approximately four inches of grass cover (Claytor and Schueler, 1996).

Vegetated swale studies have also considered other design aspects such as the siting strategies capable of optimizing runoff reduction and water quality performance. A study (Palhegyi, 2010) investigating swale surface area found that deeper channels and swale BMPs combined with detention require less surface area to construct. Also a regulatory agency (VADCR, 2011) suggests that swales should not be used to treat drainage areas greater than five acres. The same agency also recommended that, depending on design enhancements, drainage areas being treated by vegetated swales contain no greater than 37% impervious area. Design enhancements can include engineered subsurface media, an augmentation suggested when existing soils fall within the hydrologic soil groups (HSG) C and D. Engineered soil media is equipped with a perforated pipe underdrain, designed to convey filtered flow to downstream reaches of the storm sewer system. Another design improvement is the check dam, which retards flow, induces ponding, and increases hydraulic head and residence time. In slowing flow velocities and increasing depth, check dams increase hydraulic residence time and hydrostatic pressure, producing increased infiltration rates.

This study assesses the watershed-wide impacts of vegetated swales serving to convey, detain, and treat runoff via attenuation. The results can be used to evaluate vegetated swales as a potential storm sewer retro-fit solution, or as the primary source of infrastructure in new developments. This type of vegetated swale study is unique because the practice incorporates conveyance, detention, and water quality design methodology. Further, the study involves distributing vegetated swales throughout sub-watersheds to determine their cumulative attenuation effects on the watershed. This aspect of the study was also expanded to include a sensitivity analysis of vegetated swale design parameters in order to determine their effect on watershed hydrology.

3.2 Methods

The watershed modeled in this study is within the Town of Blacksburg, Virginia, and consists of a mixture of residential, commercial, municipal, and open space land use. Natural drainage in the approximately 380-acre watershed occurs through a branch of Stroubles Creek which spans the length of the basin. Sub-watersheds (catchments) were delineated according to common land uses and to the confluences of dense reaches of storm sewer. Due to this delineation approach, the catchment scale was arbitrary, ranging in area from approximately 1-30 acres. The watershed's elevations range from approximately 2,060-2,320 feet. A rain gauge exists just outside the extents of the watershed, and a flow gauge was installed at the outlet indicated in Figure 3.1. Data collected from the rain and flow gauges were used to validate a model of the study watershed.

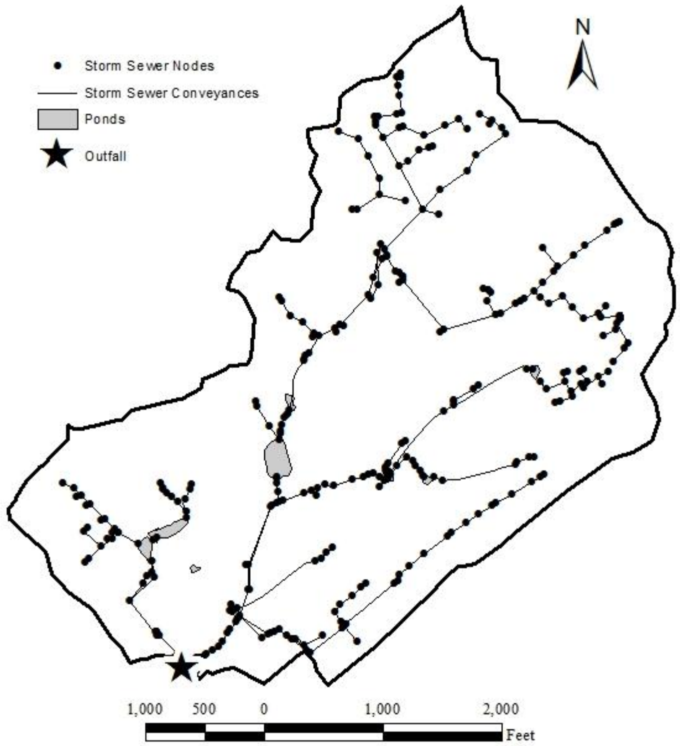


Figure 3.1: Study Watershed

The study incorporated ten modeling scenarios which varied vegetated swale parameter dimensions to determine their effect on watershed-scale runoff attenuation. Each scenario was subjected to various return frequency rainfall events, and their results were compared to models of the watershed's existing and predevelopment conditions. The predevelopment and existing watershed conditions provided a means of comparison for the sensitivity analysis models, showing the attenuation achieved with respect to the existing condition and the progress made with respect to matching the predevelopment condition. The vegetated swales in this study were assumed to serve as the primary conveyance, detention, and treatment practice for a sub-watershed. The swales were also assumed to mimic implementation as it would occur in a new development or widespread urban retro-fit, and the results will provide insight into their effects on the study watershed's hydrology.

The following models were developed to compare to the existing and predevelopment watershed conditions on the basis of volume, peak flow rate, and time to peak attenuation:

1. *Base model*

   The model incorporated vegetated swales with check dam enhancements sized according to the 2-year normal depth.

2. *Base check dam height model*

   The model followed a similar vegetated swale design procedure to the base scenario; however, the check dam enhancements were maintained at a constant height.

3. *Minimum/maximum check dam height models*

   The models were derived from the base check dam height model, and swale design was varied to include decreased and increased check dam heights respectively.

4. *Minimum/maximum infiltration rate models*

   The models were derived from the base model, and swale design was varied to include decreased and increased infiltration rates respectively.

5. *Minimum/maximum Manning's n models*

   The models were derived from the base model, and swale design was varied to include decreased and increased Manning's n roughness coefficients respectively.

6. *Minimum/maximum longitudinal slope models*

   The models were derived from the base model, and swale design was varied to include decreased and increased longitudinal slopes respectively.

3.2.1 Existing Condition

A model of the existing stormwater network in the study watershed was produced in order provide a control for the vegetated swale integrated models. The model includes the regional ponds, open channels, closed conduits, and nodes, with elevations and inverts representative of the actual storm sewer.

The infrastructure within the existing model was derived using field, as-built, and aerial survey data. Topography within the watershed was developed using LiDAR data, and physical attributes were collected in a field survey for manholes, catch basins, subsurface junctions, outfalls, pipes, open channels, and ponds. The data were compiled and manipulated in a geographic information system (GIS). After simplifying the data, the model was imported to into a modeling environment (Bentley, 2010), which was used to perform the hydrologic and hydraulic calculations in this study.

Sub-watersheds (catchments) were delineated to the confluences of dense branches of storm sewer within the study watershed, and an effort was made to maintain land use homogeneity within each catchment. Catchments were delineated to the confluences of developments such as apartment complexes, commercial developments, and residential subdivisions (Elliot, Trowsdale and Wadhwa, 2009). LiDAR aerial survey data provided the elevations required to determine drainage divides, slopes, and inverts. Runoff was only introduced at the downstream most node in a catchment, where the NRCS TR-55 (USDA, 1986) equations were the basis of runoff calculations for various return frequency rainfall events. Once the flow was introduced to the system at the catchment's outfall, the implicit computational engine (Bentley, 2010) performed hydraulic routing to the watershed outfall.

Another model input parameter was an NRCS area-weighted curve number (CN) assigned to each catchment on the basis of land use and hydrologic soil group (HSG). Detailed land cover polygons provided an area for land uses such as buildings, asphalt and concrete pavement, meadow, light, and dense forest. The land cover polygons were intersected with Soil Survey Geographic Database (SSURGO) soil data (USDA, 2011), then land use and hydrologic soil groups were used to derive a CN from TR-55 (USDA, 1986) tables.

The final catchment property accounted for was time of concentration. Times of concentration were developed using the sheet (Seelye, 1945), shallow concentrated (USDA, 1986), and channel (Kirpich, 1940) flow equations in conjunction with slopes calculated from the LiDAR survey data. Flow paths were delineated for several possible hydraulically most distant points, and the one resulting in the greatest combined travel time provided the time of concentration.

The existing conditions model was calibrated using a rain gauge just beyond the study watershed's boundary and a flow gauge located at the watershed's outfall. Eight 24-hour rainfall events totaling between 0.85 and two inches each were used to evaluate the model. Metrics used to compare the observed and modeled hydrographs included the Nash-Sutcliffe "goodness of fit" coefficient (R_{NS}^2), peak flow rate, and volume differences. The storms and corresponding observed hydrographs with the lowest lag times exhibited the best R_{NS}^2 values (0.80-0.93; goodness of fit increases as R_{NS}^2 approaches one). Peak flow rate differences ranged from 1-49%, while the volume difference range was 3-99%. Differences between measured and modeled flows were likely due to one or a combination of the following: 1) spatial variance in rainfall depth and/or intensity across the watershed, 2) location of the rain gauge with respect to the watershed, and/or 3) debris build-up and/or inadequate maintenance and calibration leading to instrumentation error at the outlet flow gauge. Figure 3.2 provides an example of results for a calibration event occurring on July 4, 2011 which totaled 0.92 inches.

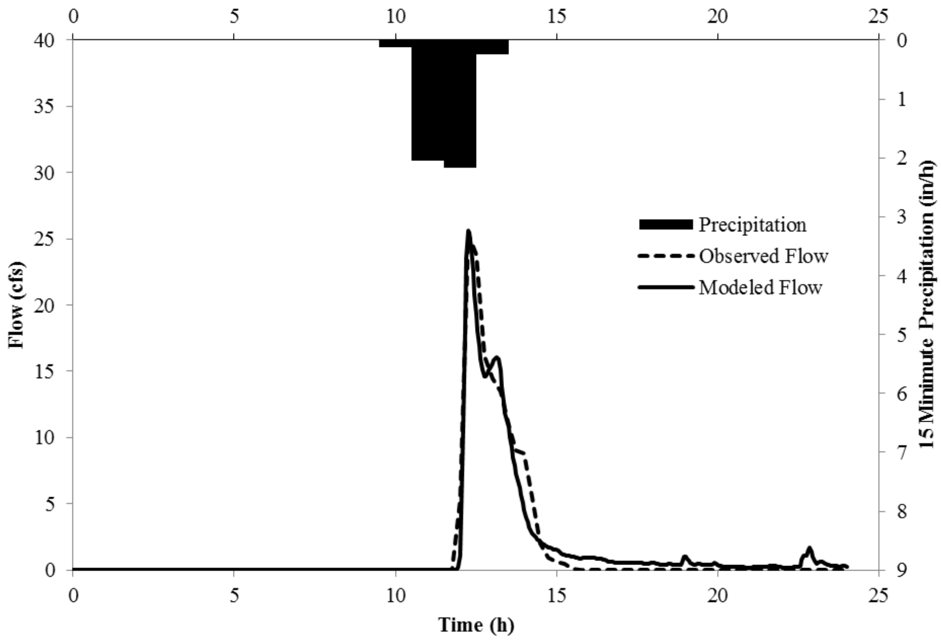


Figure 3.2: July 4, 2011 Storm Event

3.2.2 Predevelopment Condition

A model of the study watershed in its predevelopment conditions was produced in order to understand the effects of land development and the relative impacts of LID. This model consists of a single catchment with a CN and time of concentration based on forested land cover in good condition (Table A.1). Elevations along the time of concentration flow path were assumed to be the same as the existing condition.

3.2.3 Base Scenario Models

The base models involved locating a vegetated swale at the outlet of each catchment meeting feasibility constraints. Feasibility constraints introduced some degree of practicality to an otherwise theoretical model. As a measure of feasibility, vegetated swales were only implemented in true sub-basins and when design resulted in a swale requiring 10% or less of the catchment for construction (a swale's required surface area is the product of its top width and total length). A true sub-basin is defined as a catchment devoid of discharge from an upstream catchment. The true sub-basin (Figure 3.3) constraint was established due to the erosive vulnerability vegetated swales exhibit for the high velocity, concentrated flows common in downstream reaches. While channels can be armored to counteract the effects of erosion, it was decided that the scope of this study would only include swale implementation in true sub-basins. Other siting constraints found in regulatory standards for dry swales included a maximum treatment area of five acres as well as a maximum drainage area impervious coverage of 37%

(VADCR, 1999). Taking into consideration the other siting constraints as well as the increased level of design used in this study, the area and imperviousness constraints were not included. The vegetated swales meeting the constraining criteria represented the primary conveyance and treatment practice within the catchment. Aside from swale implementation, the base models retained the same hydrologic and hydraulic features as the existing model.

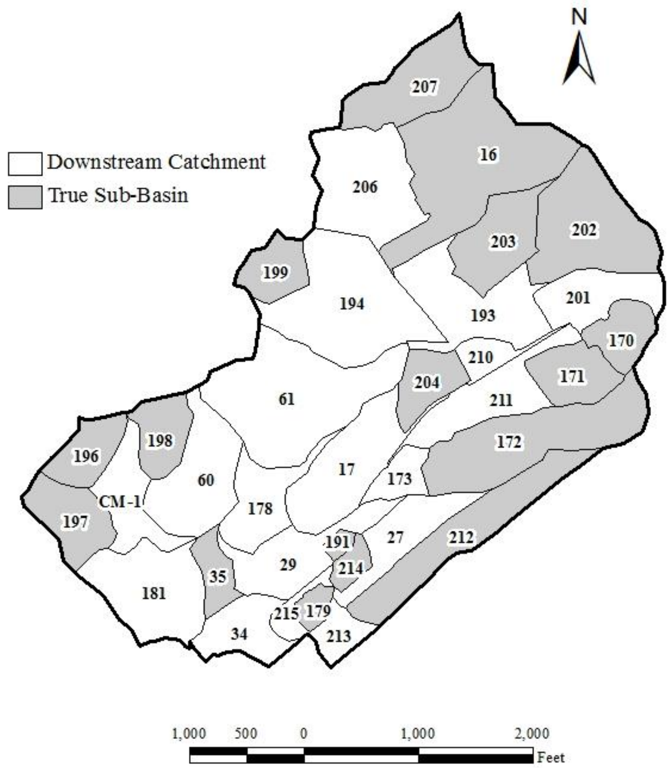


Figure 3.3: Study Watershed True Sub-basins

The vegetated swales modeled in this study account for surface storage and infiltration losses. Sub-surface storage was considered by means of assigning an infiltration rate to the swale bed area. The infiltrated volume was subtracted from the system and never re-introduced back to the system. Neglecting to re-introduce infiltration volumes in downstream reaches is a reasonable assumption as sub-surface travel times far exceed the duration of the model simulation (24-hours).

The base scenario model involved a hybrid design process, incorporating the traditional rate-based open channel design as well water quality and detention basin routing methodology. The base model check dam height is a function of the 2-year normal depth, while the maximum channel depth is the 10-year normal depth plus an additional freeboard factor (f_b) (VADCR, 2011). The design parameters represented in Figure 3.4, including the cross-sectional shape (trapezoidal), base width (w_b), side slope horizontal factor (z:1, horizontal:vertical), freeboard, weir notch depth (d_n), and weep hole diameter (d_{wh}) are constant throughout the study. The dimensions assigned to these parameters are outlined in Table 3.1.

Table 3.1: Swale Design Constants

Parameter	Units	Value
Base Width, w_b	ft	5
Side Slope (H:V), z:1	ft	3:1
Weir Notch Depth, d_n	-	$0.25(H_d)$
Freeboard, f_b	in	6
Weep Hole Diameter, d_{wh}	in	2

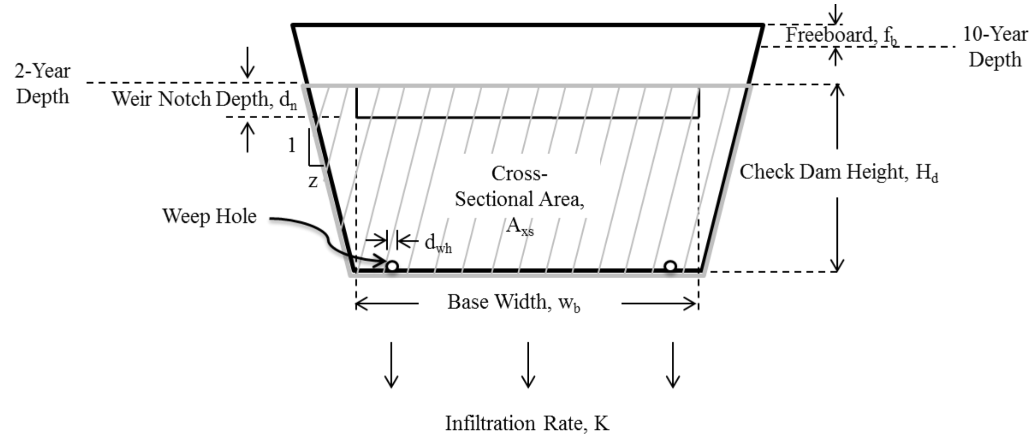


Figure 3.4: Vegetated Swale Section View

All channels are assumed to be dry swales equipped with an engineered soil media matrix, perforated pipe underdrain, and check dams. This assumption was required due to the low permeability of the soils found throughout the study watershed (Table 3.2) (VADCR, 2011).

Table 3.2: Study Watershed Soil Types [adapted from (James, 2010)]

Soil Type	Area (acres)	% Watershed Area
B	17.3	4.6
C	277	72.7
D	86.5	22.7

The check dams assume a position in the channel as indicated in Figure 3.5, where the toe of the upstream dam is equal in elevation to the top of the downstream dam. The 2- and 10-year normal depths (y_n) resulting from the corresponding peak runoff rate (q_p) calculated using the NRCS TR-55 (USDA, 1986) approach, were solved for using the Manning's equation for open, trapezoidal channel flow (Equation 3.1). Other parameters in Equation 3.1 include Manning's n roughness coefficient (n), longitudinal slope (S_0), base width (w_b), and side slope width factor (z). The depth was used to determine an average cross-sectional area (A_{avg}, Equation 3.2) to be applied to all swale segments (SS_N, Figure 3.5). The average cross-sectional area and segment length (L_{SS}, Equation 3.3) provided a segment volume (V_{ss}, Equation 3.4) used to calculate the number of segments (N) and overall swale length (L) to meet design specifications.

$$\frac{nq_p}{1.49S_0^{1/2}} = \frac{(w_b y_n + z y_n^2)^{5/3}}{\left[w_b + 2y_n(1+z^2)^{1/2}\right]^{2/3}} \qquad \text{Equation 3.1}$$

$$A_{avg} = 0.5(w_b H_d + zH_d^2) \text{ [ft}^2\text{]} \qquad \text{Equation 3.2}$$

$$L_{ss} = \frac{H_d}{S_0} \text{ [ft]} \qquad \text{Equation 3.3}$$

$$V_{ss} = A_{avg} L_{ss} \text{ [ft}^3\text{]} \qquad \text{Equation 3.4}$$

In order to determine the required storage volume (V_s) for the swales, the detention basin routing methodology from NRCS TR-55 (USDA, 1986) was implemented. Detention basin routing was used to compute swale storage volume for two reasons: 1) the swales were assumed to be the only conveyance and treatment practice available in the catchment, serving as the primary stormwater infrastructure in a new development or widespread retro-fit, and 2) the vegetated swale check dam systems were modeled as a single pond element. Also, the 2-year storage volume was chosen as the design level due to the regulations set forth by the locality housing the study watershed (Town of Blacksburg, 2008). The 2-year storage volume for the catchment of interest gave rise to the number of required swale segments and total swale length (L) (Equations 3.5 and 3.6).

$$N = ROUND\left[\frac{V_S}{V_{SS}}, 0\right] + 1 \qquad \text{Equation 3.5}$$

$$L = NL_{SS} \qquad \text{Equation 3.6}$$

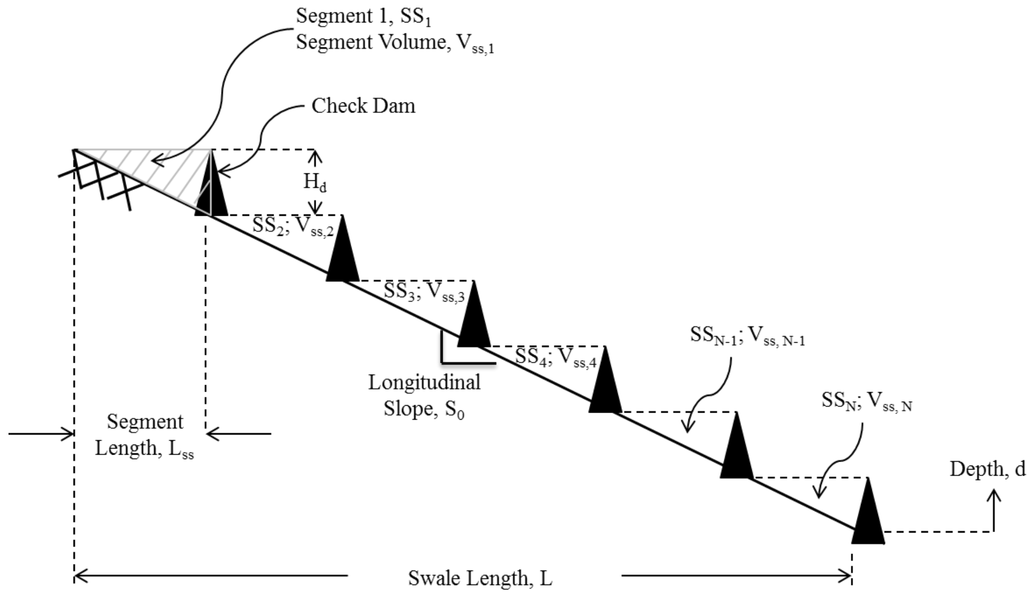


Figure 3.5: Vegetated Swale Profile View

While in reality, the vegetated swale check dam systems would be divided into multiple linear reaches, appropriately sited adjacent to roadways and parking lots throughout the catchment, they were represented in the model by a single storage element with a composite outlet structure. The composite outlet structure represents the check dam in SS_N, with depth (d) for the storage element's rating curve originating at the toe (Figure 3.5). At the origin, the swale's surface area (A_s, Equation 3.7 and Figure 3.6) is the sum of surface areas in all segments upstream of SS_N assuming flow levels have reached the top of dam. Only in segment SS_N does the area become a function of depth (second term, Equation 3.7). The outlet control structure represents the check dam in SS_N, since in the assumed condition when all upstream segments are spilling over the top of dam, this would be the only dam controlling outflow. The outlet structure contains weep holes (two, 2-inch diameter orifices at a depth of zero), a notch weir (rectangular contracted weir with a length equivalent to the base width at depth $0.75(H_d)$), and the top of dam (a rectangular suppressed weir with a length and depth proportional and equal to the 2-year flow depth respectively).

$$A_s = \left[\frac{H_d(N-1)}{S_0}(w_b + 2zH_d)\right] + \frac{d}{S_0}(w_b + 2zd)\ [\text{ft}^2] \qquad \text{Equation 3.7}$$

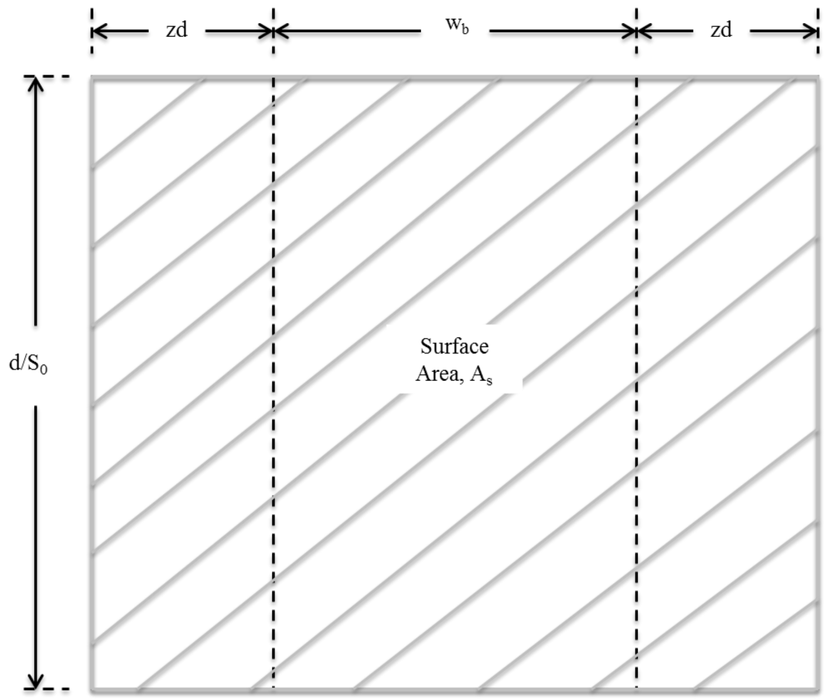


Figure 3.6: Vegetated Swale Plan View

Another base modeling scenario was developed incorporating a fixed check dam height of one foot above the swale bed. The previously outlined design procedure was kept the same, however, check dam height was not a function of normal depth. All constraints were maintained, but the check dam height was fixed at one foot for all swales, regardless of the flows they were treating. Only the number of segments and total swale length varied according to the required storage volume.

3.2.4 Sensitivity Analysis

The remaining scenarios are variations of the base and base check dam height models, diverse only in the dimensions outlined in Table 3.3. The minimum and maximum check dam height models were derived from the base check dam height scenario, while all other scenarios descend from the base. These scenarios were intended to gauge the effect of increasing and decreasing certain vegetated swale parameters on the impact of watershed hydrology.

Table 3.3: Model Sensitivity Analysis

Parameter	Units	Minimum Dimension	Base Dimension	Maximum Dimension
Check Dam Height, H_d	ft	0.5	1	2
Average Infiltration Rate, K	in/hr	0.5	1.5	3
Manning's n	unitless	0.03	0.11	0.15
Longitudinal Slope, S_0	ft/ft	0.005	0.02	0.04

These models are the result of an analysis performed on Equation 3.1, to determine which parameters most greatly influence normal depth. Since normal depth directly correlates to segment storage, the sensitivity analysis could be most effectively designed by understanding which parameters were most significant to normal depth. The Manning's n value and longitudinal slope were the only parameters with a direct correlation to segment volume; the remaining parameters (Table 3.1) relate to cross-sectional area, where the variation of any one parameter necessitates variation of the others in order to produce the same cross-sectional area. Infiltration rate was another parameter included in the sensitivity analysis due to its expected impact on a swale's water balance.

3.3 Results and Discussion

The models were subjected to the 1-, 2-, 5-, and 10-year return frequency rainfall events for Blacksburg, Virginia. Rainfall depths were derived using the NOAA Atlas-14, Volume 2 (Bonnin, et al., 2004) partial duration series. Table 3.5 outlines the attributes of each of the resulting hydrographs.

Outfall hydrographs exhibited attenuation in all models, and volume reduction and rainfall recurrence interval correlated positively (Figure 3.7). Positive correlation between volume attenuation and rainfall recurrence interval is evidenced by the fact that most scenarios approach the predevelopment condition line as rainfall recurrence interval increases in Figure 3.7. All scenarios reduced volumes considerably as compared to the existing condition, with median volume reduction for all events ranging from 22-38% (approximately 26,400-46,500 ft^3 of reduction). The maximum longitudinal slope model exhibited the greatest median volume reduction across all events (38%), while the minimum check dam height scenario reduced the least volume (22%). In terms of median volume reduction for all rainfall events, the following results were obtained:

1. Base check dam height (34%) reduced more volume than the base scenario (33%) (difference of approximately 1,200 ft^3)

2. Base check dam height (34%) reduced more volume than minimum (22%) and maximum (25%) check dam height
3. Base and maximum infiltration rate reduced more volume (33%) than minimum infiltration rate (27%)
4. Base and minimum Manning's n reduced more volume (33%) than maximum Manning's n (30%)
5. Maximum longitudinal slope (38%) reduced more volume than minimum longitudinal slope (26%) and base (33%)

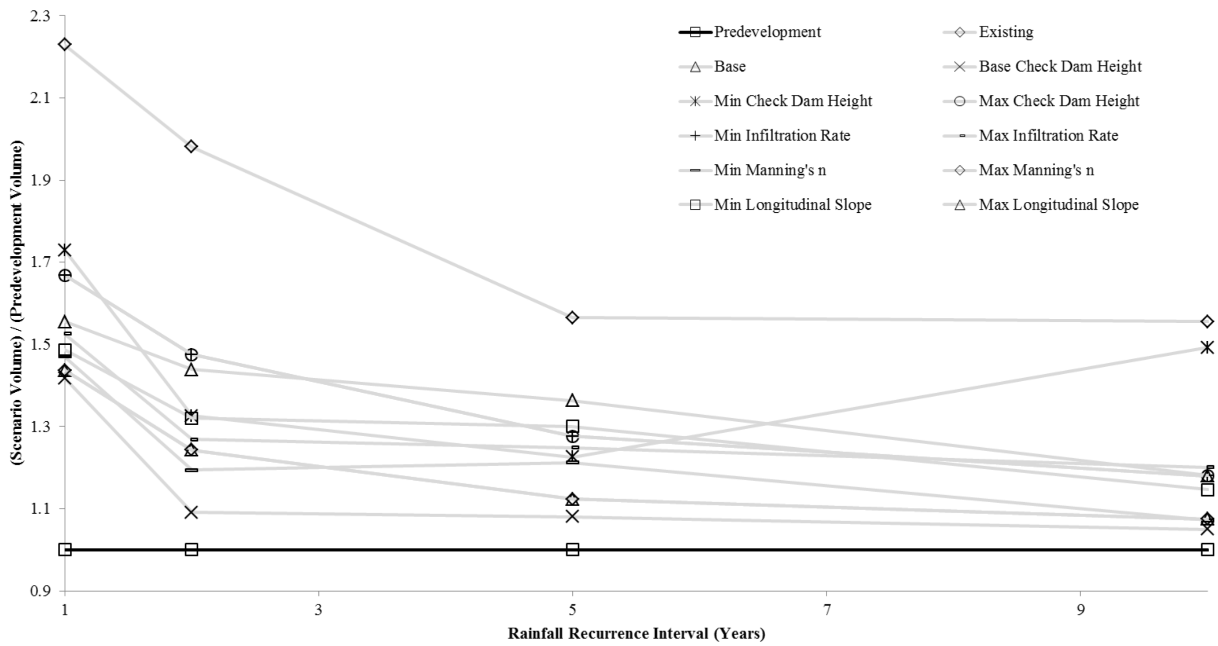


Figure 3.7: Normalized Outfall Volumes
Note: The lines connecting points are for visual clarity, not as an indication of trends.

Peak flow rate attenuation exhibited similar results to volume reduction, with peak flow reduction increasing as rainfall event recurrence interval increased. The range of median peak flow rate reduction among all rainfall events was 21-34% (or 71-115 ft^3/s of peak flow rate attenuation). All scenarios proved capable of reducing peak flow rate, but only the 10-year maximum longitudinal slope model (28% peak flow reduction) exhibited reduction below that of the predevelopment condition. The maximum longitudinal slope model (34%) had the greatest positive impact on peak flow rate attenuation, while the minimum and maximum check dam height scenarios (21% each) had the least positive effect on peak flow rate attenuation. The following results for median peak flow rate reduction among all rainfall events were observed:

1. Base check dam height (32%) more greatly decreased peak flow rate than the base scenario (31%) (difference of approximately 3 ft^3/s)

2. Base check dam height reduced peaks (32%) by more than minimum and maximum check dam height (21% each)
3. Base and maximum infiltration rate (31% each) more greatly decreased peak flow rate than minimum infiltration rate (29%)
4. Base and minimum Manning's n (31% each) more greatly decreased peak flow rate than maximum Manning's n (27%)
5. Maximum longitudinal slope (34%) reduced more peak flow than base (31%) and minimum longitudinal slope (23%)

Time to peak was also taken into consideration; however, the results were inconclusive. By adding detention volume in upstream reaches of the watershed, it was hypothesized that an increase in time to peak would be observed at the outfall. While time to peak was extended for the 1-year event among the majority of the models, results from lower frequency events exhibited no extension of time to peak.

BMP efficiency was measured in terms of volume reduction per unit area routed to the swale (E_{RA}, Table 3.4) and volume reduction per unit surface area required for swale construction (E_{SA}, Table 3.4). Like the peak flow rate and volume reduction metrics, efficiency results positively correlated to rainfall frequency interval. The minimum Manning's n model produced the greatest median E_{RA} (0.74%) among all rainfall events. The minimum longitudinal slope scenario produced the lowest median E_{RA} (0.32%). With respect to E_{SA}, the maximum check dam height model (12.4%) was most efficient. Conversely, the minimum check dam height (5.6%) resulted in the least E_{SA} of all the scenarios.

Table 3.4: Median Swale Efficiencies For All Rainfall Events

Scenario	E_{RA}	E_{SA}
Base	0.56%	10.1%
Base Check Dam Height	0.61%	11.7%
Min Check Dam Height	0.50%	5.6%
Max Check Dam Height	0.35%	12.4%
Min Infiltration Rate	0.45%	8.4%
Max Infiltration Rate	0.50%	9.3%
Min Manning's n	0.74%	9.4%
Max Manning's n	0.40%	8.6%
Min Longitudinal Slope	0.32%	6.8%
Max Longitudinal Slope	0.67%	11.5%

When evaluating the results derived from this study, it is important to consider the assumptions made when the vegetated swales were designed and modeled. One critical assumption was that vegetated swales equipped with check dams could be discretized to a single pond element and outlet control structure. While this approach accounted for stage-storage and stage-discharge relationships, it neglected the flow routing associated with linear conveyance systems such as vegetated swales. The assumption was that while the detention-based approach may be overly simplistic, it may accurately represent the performance of a vegetated swale check dam system for the low frequency rainfall events modeled in this study. It is quite possible, however, that due to the design approach taken, the rainfall events in this study would not actually inundate the BMP. In that case, a more representative approach to modeling the BMP would require flow routing throughout the length of the practice.

Another primary assumption of the study was that the vegetated swales that were implemented represented the only stormwater infrastructure in the catchment. The practices were sited to serve the conveyance, detention, and water quality treatment needs in the catchment. For these reasons, a hybrid design approach was taken, incorporating traditional rate-based capacity and detention basin routing methods. Storage was provided for the 2-year runoff volume due to widespread BMP implementation and lack of supporting infrastructure within the catchment. It is possible that this approach produced over-designed BMPs, considering water quality swales are frequently designed to capture runoff volume resulting from a one-inch rainfall event (for comparison, the 2-year event in this study was 2.75 inches). Over-designing the swales in this study would result in a model with an excessive amount of detention in the upstream reaches, likely causing attenuation results to be greater with minimized variation due to changes in model inputs.

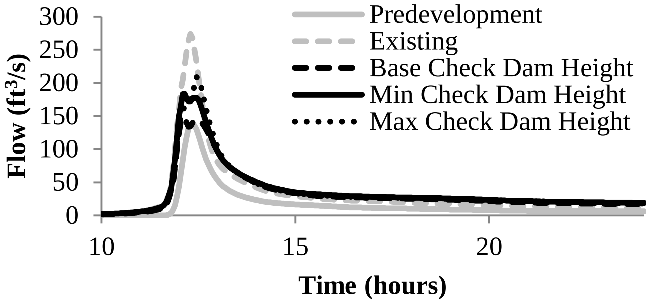
While constraints were implemented to limit swale siting in downstream catchments and where required surface area was not feasible, the practice could have also been limited by the level of imperviousness and total drainage area of each catchment. Regulatory guidance recommends using vegetated swale to treat areas consisting of no greater than five acres and/or 37% impervious cover (VADCR, 1999). It could be that by not considering these constraints, vegetated swales were somewhat carelessly sited to treat certain catchments. The concern over ignoring these constraints lies in the potential limited effectiveness of swales sited to treat large areas and dense, impervious land uses.

Any one or combination of the previously mentioned assumptions/justifications could have led to the following unexpected results:

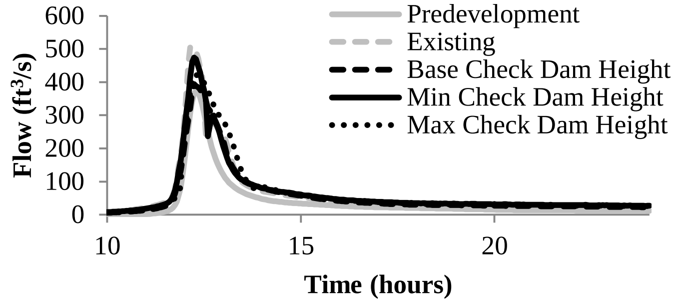
1. The time to peak assessment metric provides counterintuitive results for the 2-, 5-, and 10-year rainfall events. Increasing detention in a watershed causes the timing of peak flows to occur at an earlier time than before detention was introduced. The only explanation for why times to peak decreased as detention increased in the parametric models is software routing error.
2. Peak flow rate and volume reduction, as well as BMP efficiency, positively correlated to rainfall recurrence interval. The swales were either over-designed, excessively

implemented, or the design and modeling methodologies were overly simplistic. Over-design may have been the result of sizing all swales to account for the 2-year storage volume from the contributing drainage area, whereas accounting for the storage of a smaller water quality volume is common practice. The swales may have also been excessively implemented by neglecting to account for a maximum contributing drainage area and/or level of contributing imperviousness. Finally, over simplicity may have been a factor of not considering the length properties of the swale practices in the model, but rather representing them as a pond and outlet control structure element.

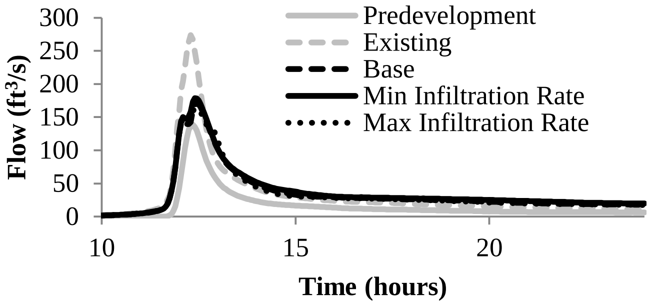
The sensitivity analysis produced a narrow range of attenuation and efficiencies. The rationale behind the positive correlation between attenuation and rainfall recurrence interval may also be applied to explain the inconclusive nature of the sensitivity analysis. Results provided little evidence supporting the significance of the parameters tested on the attenuation impacts of vegetated swales.



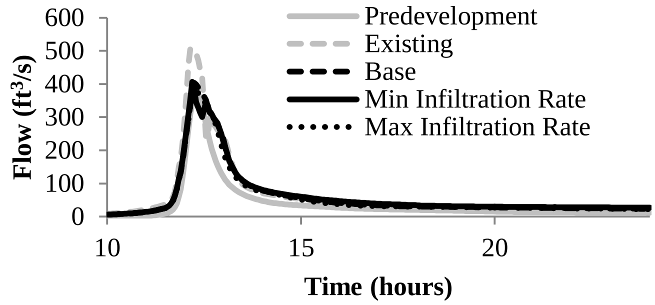
a) Check Dam Height Scenarios – 2-Year



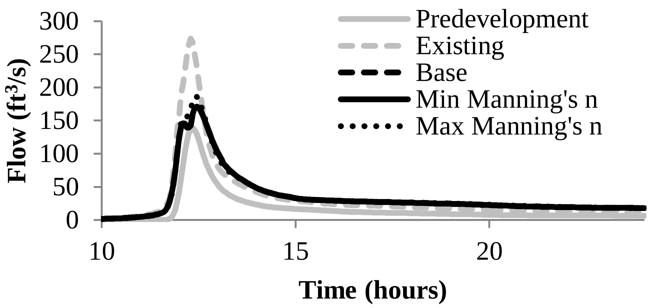
b) Check Dam Height Scenarios – 10-Year



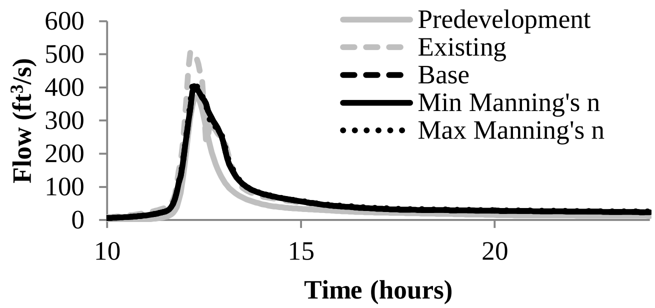
c) Infiltration Rate Scenarios – 2-Year



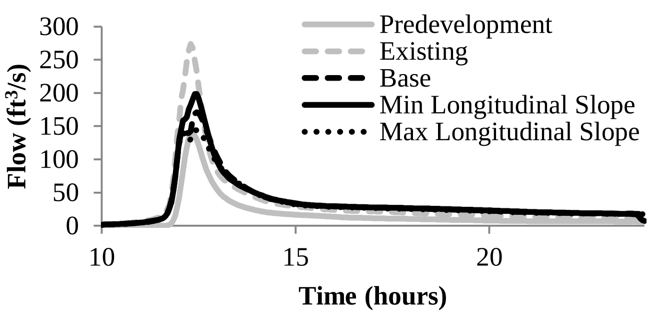
d) Infiltration Rate Scenarios – 10-Year



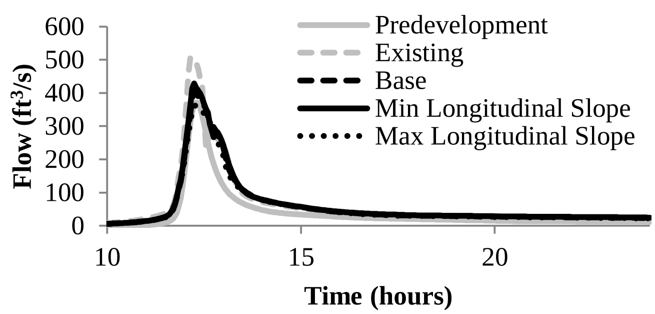
e) Manning’s n Scenarios – 2-Year



f) Manning’s n Scenarios – 10-Year



g) Longitudinal Slope Scenarios – 2-Year



h) Longitudinal Slope Scenarios – 10-Year

Figure 3.8: Outfall Hydrographs

Table 3.5: Outfall Hydrograph Attributes

24-Hour Event	*1-Year (2.27 inches)*			*2-Year (2.75 inches)*			*5-Year (3.49 inches)*			*10-Year (4.09 inches)*		
Scenario Model	Peak Flow Rate (ft^3/s)	Volume (ft^3)	Time to Peak (hours)	Peak Flow Rate (ft^3/s)	Volume (ft^3)	Time to Peak (hours)	Peak Flow Rate (ft^3/s)	Volume (ft^3)	Time to Peak (hours)	Peak Flow Rate (ft^3/s)	Volume (ft^3)	Time to Peak (hours)
Predevelopment	73.7	25,830	12.4	138.6	48,816	12.4	258.6	91,498	12.4	370.7	131,449	12.3
Existing	159.9	57,598	12.4	273.8	96,698	12.3	405.0	143,170	12.2	507.3	204,525	12.2
Base	102.6	37,116	12.4	171.0	60,683	12.3	295.5	102,787	12.2	404.0	141,244	12.2
Base Check Dam Height	103.6	36,614	12.1	146.8	53,237	12.2	284.6	98,849	12.4	394.5	138,002	12.3
Min Check Dam Height	126.3	44,678	12.1	183.6	64,742	12.2	321.0	112,100	12.3	474.5	196,168	12.3
Max Check Dam Height	115.6	43,090	12.3	209.1	72,032	12.5	330.1	116,739	12.5	423.1	155,057	12.3
Min Infiltration Rate	106.2	39,409	12.1	178.6	61,945	12.4	308.6	114,248	12.4	406.3	157,901	12.2
Max Infiltration Rate	104.6	37,978	12.1	163.7	58,300	12.5	297.1	110,914	12.4	402.3	140,926	12.3
Min Manning's n	102.6	37,116	12.1	171.0	60,683	12.5	295.5	102,787	12.4	404.0	141,244	12.3
Max Manning's n	104.7	38,410	12.1	186.6	64,462	12.4	313.2	118,987	12.4	417.8	150,673	12.3
Min Longitudinal Slope	108.1	40,181	12.1	198.5	70,238	12.5	326.2	124,724	12.4	428.4	155,259	12.3
Max Longitudinal Slope	101.6	36,254	12.1	143.8	53,577	12.5	279.5	96,934	12.4	363.1	126,790	12.3

3.4 Conclusions

This study assessed the effects of watershed-wide vegetated swale implementation on stormwater quantity. A calibrated model served as the foundation from which various swale-centric models were derived. These models included two standard design approaches as well as a sensitivity analysis which increased and decreased four vegetated swale design parameters. The study concluded the following:

1. Vegetated swale systems equipped with check dams and distributed amongst upstream drainage basins provided flow attenuation at a watershed scale. The practice offered between 21-34% median peak flow reduction as well as 22-38% median volume reduction across all rainfall events for the various design parameters tested. Performance, however, is largely a function of the design, modeling, and constraining methodology used to develop and implement the swales.
2. The design and modeling processes produced over-designed, inaccurately discretized, and/or under constrained vegetated swale check dam systems. This conclusion was reached due to the positive correlation between attenuation and rainfall recurrence interval, narrow range in attenuation results, and inexplicable performance of parametric models relative to one another.
3. Including routing effects over the length of a swale may provide a more representative model of the BMP. The approach taken in this study is for length to only factor into the storage capacity of the swale, without having much of an effect on the timing and reduction of flow pulses traveling through the practice.
4. Increasing the check dam height in a vegetated swale check dam system reduces the practice's required surface area. The maximum check dam height scenario efficiency results were among the lowest in E_{RA} (volume reduction per area routed to the swale) and the highest in E_{SA} (volume reduction per surface area required for swale construction). The implication of these results is that by increasing check dam height, a swale's required surface area decreases, directly increasing its E_{SA}. The results also indirectly affect E_{RA}, as decreasing the surface area required to construct a swale lends its availability to more drainage basins, increasing the area that can be routed to the practice.

4 Conclusions

4.1 Implications

This study provided insight into the significance the design and modeling approaches have on a BMP's performance. The vegetated swale check dam systems were possibly over-sized in design or over-simplified in the model which likely produced the inconclusive results. Due to possibly over-sizing the practice in the design process or over simplifying it in the modeling process, the study designed to evaluate vegetated swale check dam systems as a means of conveyance and runoff reduction in an urban watershed produced inconclusive results. It is also possible that the constraints put in place to insure practical swale siting did not adequately limit BMP distribution.

4.2 Future Work

Considering the inconclusive nature of this study, it is important to consider evolving and/or altering the parameters of this research. The results derived from this study were highly dependent upon the design and modeling approaches taken to represent the vegetated swale check dam systems. The assumption was that dividing the open channel with check dams would create a linear series of reservoirs, the culmination of which was modeled in a single detention element. This approach considered routing to the swales by means of a time of concentration to the sub-watershed's outlet; however, no consideration was given to routing down the longitudinal length of the swale. Also, in reality, swales would be distributed throughout a drainage basin, sited alongside roadways and parking lots. A modeling approach considering the effects of routing through a swale as well as the physical distribution of the practices throughout the watershed may yield more conclusive results.

Future researchers may also consider investigating other hydrologic modeling software packages to explore the elements available for simulating LID practices. The software (Bentley, 2010) used in this study contained open channel elements, but they simply served as a means of connecting nodes to one another. The open channel element did not allow for check dam installation at a particular interval, nor did it provide any sort of infiltration/sub-surface storage options. Other software packages may contain more holistic open channel modeling which could provide more conclusive results.

One area of this research worth paying closer attention to is the design of the sensitivity analysis. The study was comprised of an analysis of four design parameters which included check dam height, infiltration rate, Manning's n, and longitudinal slope. These parameters were assigned base values by consulting a range of dimensions provided in regulatory standards (VADCR, 2011) and choosing an approximate median value. The base values were increased by approximately 50% where possible, or to the high and low ends of the suggested ranges when such increase was not a possibility. The sensitivity analysis could be expanded to include dimensions for a greater number of intervals from the base value (ie. ±5%, ±10%, ±25%, ±50%, ±75%, and ±100% of the base dimension) in order to provide further insight into the effects of increasing and decreasing certain parameters on attenuation results. This type of analysis could also result in dimensions that optimize flow attenuation in vegetated swales. This type of approach could be expanded even further to include other vegetated swale attributes.

While pollutant removal via runoff reduction is important to quantify for a stormwater management practice, it is also helpful to know how much direct pollutant removal it is capable of achieving. With the total maximum daily loads (TMDLs) being proposed for a growing number of United States surface water bodies, engineers must be able to show water quality options for site plans where there is proposed land disturbance. In order to assign the appropriate removal efficiencies to the various BMPs, engineers need reliable efficiencies for direct pollutant removal and removal by runoff reduction. While preliminary BMP efficiencies have been assigned by various regulatory agencies, further research is needed to support these values. Phosphorus, nitrogen, heavy metals, total suspended solids, and hydrocarbons are among the most highly regulated pollutants. Models simulating the build-up and washoff cycles of these pollutants and the efficiency with which vegetated swales and other BMPs provide treatment would be greatly beneficial to engineers performing water quality calculations for land development projects.

The theoretical nature of this study should also be considered, and further research could investigate ways of making vegetated swale siting and analysis more practical. As was previously stated, municipalities would find it difficult to implement vegetated swale check dam systems to the level proposed in this study. Future work should consider the location of planimetric features throughout drainage basins, choosing to implement swales where it is practical and considering other BMPs where swales are an impractical option. It should be noted, however, that any approach involving practical swale distribution according to planimetric features would require increasing the discretization of sub-watersheds, complicating models past the scope of this study. It is possible, however, that this level of detail is required to produce more accurate results for the vegetated swale design approach and model representation taken in this study.

4.3 Final Words

Mankind's proliferated awareness of the environment within the last half century has given rise to a paradigm shift in the approach to stormwater management from strictly quantity-based design to an equal level of concern for water quantity and quality. The concern for downstream receiving channels, water bodies, and ecosystems has led to the development of LID. The primary concern with these practices as of late has been gathering data to gauge the efficiency at which they are operating. This research has proposed a unique way in which vegetated swale check dam systems can be designed and modeled. Various tests were designed to evaluate this uniquely designed practice as it is distributed across an urban watershed model. Regardless of what was learned in this study and new questions that were raised, ideas were presented that can be expanded upon by future researchers in order to further the understanding of BMP performance.

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Appendix A

Model Input Data

This section summarizes the catchments used to develop each of the scenario models in the study. The predevelopment model is described in Table A.1 and does not include an associated figure due to the fact that it consists of only a single catchment. Table A.2 shows the catchment attributes used in all models except the predevelopment. Table A.3 and Figure A.1 depict the true sub-basins which receive dry swale treatment in the parametric scenario models. The following tables contain attributes of the designed swales for each parametric scenario model.

Table A.1: Predevelopment Model Catchment

CN	Area (Ac)	T_c (hours)
71	381.8	0.67

Table A.2: Existing Conditions Model Catchments

ID	CN	Area (Ac)	T_c (hours)
16	82	30.3	0.22
17	86	15.3	0.24
27	86	8.9	0.32
29	87	11.3	0.23
34	85	7.7	0.33
35	85	4.2	0.18
60	83	14.5	0.24
61	79	27.4	0.36
170	84	6.0	0.18
171	86	7.6	0.12
172	81	21.5	0.29
173	84	4.0	0.20
178	84	9.1	0.24
179	93	2.1	0.15
181	88	15.9	0.24
191	90	1.1	0.16
193	78	16.5	0.21
194	79	24.9	0.22
196	86	6.4	0.21
197	88	8.8	0.23
198	85	6.6	0.20
199	82	6.1	0.22
201	80	9.2	0.19
202	79	19.4	0.29
203	80	10.8	0.23
204	76	6.2	0.23
206	83	18.2	0.24
207	78	13.3	0.26
210	82	3.4	0.20
211	80	11.4	0.17
212	84	16.3	0.32
213	87	3.9	0.18
214	90	2.4	0.21
215	79	2.4	0.20
CM-1	83	8.8	0.24

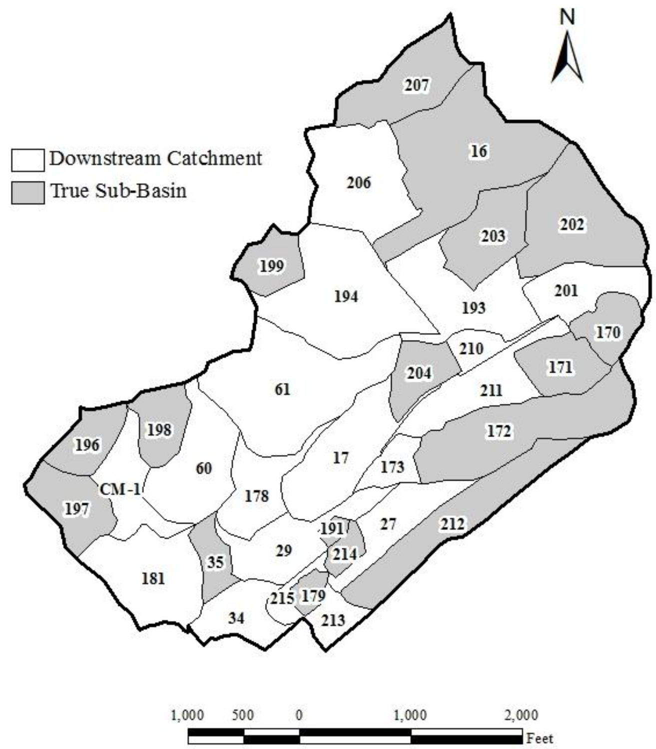

Figure A.1: Study Watershed Catchments

Table A.3: Parametric Model True Sub-basins

ID	Pre-Development CN	Existing Condition CN	Area (Ac)	Impervious Cover	T_c (hours)	$q_{p,2}$ (ft^3/s)	$q_{p,10}$ (ft^3/s)	Storage Volume, V_s (ft^3)
16	72	82	30.3	35%	0.22	35.1	81.3	32200
35	73	85	4.2	26%	0.18	6.3	14.0	5600
170	67	84	6.0	40%	0.18	8.4	19.0	10200
171	70	86	7.6	46%	0.12	13.8	30.4	12500
196	76	86	6.4	31%	0.21	9.6	20.8	8000
172	72	81	21.5	16%	0.29	21.1	49.8	21000
179	71	93	2.1	73%	0.15	5.6	10.8	5900
191	70	90	1.1	65%	0.16	2.3	4.8	2500
197	77	88	8.8	39%	0.23	14.7	30.6	12700
198	73	85	6.6	28%	0.20	9.5	21.0	8800
199	70	82	6.1	22%	0.22	7.0	16.3	7300
202	67	79	19.4	23%	0.29	16.5	40.7	22000
203	70	80	10.8	24%	0.23	11.0	26.4	10800
204	70	76	6.2	12%	0.23	4.7	12.4	4000
207	68	78	13.3	17%	0.26	11.2	28.3	12600
212	73	84	16.3	24%	0.32	18.4	40.8	20100
214	72	90	2.4	59%	0.21	4.6	9.2	5100

Table A.4: Base, Minimum Infiltration Rate, and Maximum Infiltration Rate Model Catchments and Swales

Catchments		*Swales*					*Area Constraint*
ID	Area, A (Ac)	Check Dam Height (ft)	Depth (ft)	Length, L (ft)	Top Width (ft)	Area, A_s (Ac)	A_s/A
191	1.1	0.41	1.11	1994	11.6	0.27	24.9%
179	2.1	0.66	1.44	2553	13.6	0.40	19.4%
214	2.4	0.59	1.36	2575	13.2	0.39	16.6%
170	6.0	0.83	1.76	3343	15.5	0.60	10.0%
35	4.2	0.71	1.58	2231	14.5	0.37	8.8%
199	6.1	0.75	1.66	2730	15.0	0.47	7.8%
171	7.6	1.07	2.09	2884	17.5	0.58	7.7%
198	6.6	0.88	1.82	2641	15.9	0.48	7.3%
196	6.4	0.88	1.82	2387	15.9	0.44	6.8%
197	8.8	1.10	2.09	2813	17.6	0.57	6.5%
203	10.8	0.95	1.98	2936	16.9	0.57	5.3%
204	6.2	0.60	1.51	1957	14.1	0.32	5.1%
212	16.3	1.24	2.34	3773	19.0	0.82	5.1%
202	19.4	1.17	2.33	4452	19.0	0.97	5.0%
207	13.3	0.96	2.04	3361	17.2	0.66	5.0%
172	21.5	1.33	2.52	3583	20.1	0.83	3.8%
16	30.3	1.71	3.04	3754	23.2	1.00	3.3%

Table A.5: Base Check Dam Height Model Catchments and Swales

Catchments		*Swales*					*Area Constraint*
ID	Area, A (Ac)	Check Dam Height (ft)	Depth (ft)	Length, L (ft)	Top Width (ft)	Area, A_s (Ac)	A_s/A
179	2.1	1.00	1.75	1500	15.5	0.27	12.9%
191	1.1	1.00	1.75	650	15.5	0.12	10.8%
214	2.4	1.00	1.75	1300	15.5	0.23	9.8%
170	6.0	1.00	1.75	2600	15.5	0.46	7.7%
171	7.6	1.00	1.75	3150	15.5	0.56	7.4%
197	8.8	1.00	1.75	3200	15.5	0.57	6.5%
198	6.6	1.00	1.75	2200	15.5	0.39	5.9%
35	4.2	1.00	1.75	1400	15.5	0.25	5.9%
196	6.4	1.00	1.75	2000	15.5	0.36	5.6%
212	16.3	1.00	1.75	5050	15.5	0.90	5.5%
199	6.1	1.00	1.75	1850	15.5	0.33	5.4%
202	19.4	1.00	1.75	5500	15.5	0.98	5.0%
16	30.3	1.00	1.75	8050	15.5	1.43	4.7%
203	10.8	1.00	1.75	2700	15.5	0.48	4.4%
172	21.5	1.00	1.75	5250	15.5	0.93	4.3%
207	13.3	1.00	1.75	3150	15.5	0.56	4.2%
204	6.2	1.00	1.75	1000	15.5	0.18	2.9%

Table A.6: Minimum Check Dam Height Model Catchments and Swales

Catchments		*Swales*					*Area Constraint*
ID	Area, A (Ac)	Check Dam Height (ft)	Depth (ft)	Length, L (ft)	Top Width (ft)	Area, A_s (Ac)	A_s/A
179	2.1	0.50	1.25	3650	12.5	0.52	25.3%
191	1.1	0.50	1.25	1550	12.5	0.22	20.8%
214	2.4	0.50	1.25	3150	12.5	0.45	19.2%
170	6.0	0.50	1.25	6300	12.5	0.90	15.1%
171	7.6	0.50	1.25	7725	12.5	1.11	14.7%
197	8.8	0.50	1.25	7800	12.5	1.12	12.8%
198	6.6	0.50	1.25	5425	12.5	0.78	11.8%
35	4.2	0.50	1.25	3425	12.5	0.49	11.7%
196	6.4	0.50	1.25	4925	12.5	0.71	11.1%
212	16.3	0.50	1.25	12400	12.5	1.78	10.9%
199	6.1	0.50	1.25	4500	12.5	0.65	10.7%
202	19.4	0.50	1.25	13525	12.5	1.94	10.0%
16	30.3	0.50	1.25	19825	12.5	2.84	9.4%
203	10.8	0.50	1.25	6650	12.5	0.95	8.8%
172	21.5	0.50	1.25	12925	12.5	1.85	8.6%
207	13.3	0.50	1.25	7750	12.5	1.11	8.3%
204	6.2	0.50	1.25	2450	12.5	0.35	5.7%

Table A.7: Maximum Check Dam Height Model Catchments and Swales

Catchments		*Swales*					*Area Constraint*
ID	Area, A (Ac)	Check Dam Height (ft)	Depth (ft)	Length, L (ft)	Top Width (ft)	Area, A_s (Ac)	A_s/A
179	2.1	2.00	2.75	600	21.5	0.15	7.2%
191	1.1	2.00	2.75	300	21.5	0.07	6.9%
214	2.4	2.00	2.75	500	21.5	0.12	5.3%
170	6.0	2.00	2.75	1000	21.5	0.25	4.1%
171	7.6	2.00	2.75	1200	21.5	0.30	3.9%
35	4.2	2.00	2.75	600	21.5	0.15	3.5%
197	8.8	2.00	2.75	1200	21.5	0.30	3.4%
196	6.4	2.00	2.75	800	21.5	0.20	3.1%
198	6.6	2.00	2.75	800	21.5	0.20	3.0%
212	16.3	2.00	2.75	1900	21.5	0.47	2.9%
199	6.1	2.00	2.75	700	21.5	0.17	2.9%
202	19.4	2.00	2.75	2000	21.5	0.49	2.5%
16	30.3	2.00	2.75	3000	21.5	0.74	2.4%
172	21.5	2.00	2.75	2000	21.5	0.49	2.3%
203	10.8	2.00	2.75	1000	21.5	0.25	2.3%
207	13.3	2.00	2.75	1200	21.5	0.30	2.2%
204	6.2	2.00	2.75	400	21.5	0.10	1.6%

Table A.8: Minimum Manning's n Model Catchments and Swales

Catchments		*Swales*					*Area Constraint*
ID	Area, A (Ac)	Check Dam Height (ft)	Depth (ft)	Length, L (ft)	Top Width (ft)	Area, A_s (Ac)	A_s/A
191	1.1	0.19	0.79	4694	9.7	0.53	49.2%
179	2.1	0.32	0.96	6197	10.8	0.77	37.1%
214	2.4	0.28	0.92	6168	10.5	0.75	31.8%
170	6.0	0.40	1.13	8155	11.8	1.10	18.5%
35	4.2	0.34	1.04	5389	11.2	0.69	16.5%
199	6.1	0.36	1.08	6601	11.5	0.87	14.4%
171	7.6	0.53	1.32	7153	12.9	1.06	14.0%
198	6.6	0.43	1.17	6463	12.0	0.89	13.5%
196	6.4	0.43	1.17	5820	12.0	0.80	12.6%
197	8.8	0.55	1.32	6936	12.9	1.03	11.7%
203	10.8	0.47	1.26	7222	12.5	1.04	9.6%
204	6.2	0.29	1.00	4673	11.0	0.59	9.5%
212	16.3	0.62	1.45	9436	13.7	1.49	9.1%
202	19.4	0.59	1.45	11071	13.7	1.74	9.0%
207	13.3	0.47	1.29	8279	12.7	1.21	9.1%
172	21.5	0.67	1.56	8901	14.4	1.47	6.8%
16	30.3	0.88	1.86	9573	16.2	1.78	5.9%

Table A.9: Maximum Manning's n Model Catchments and Swales

Catchments		*Swales*					*Area Constraint*
ID	Area, A (Ac)	Check Dam Height (ft)	Depth (ft)	Length, L (ft)	Top Width (ft)	Area, A_s (Ac)	A_s/A
191	1.1	0.48	1.22	1623	12.3	0.23	21.5%
179	2.1	0.78	1.60	2078	14.6	0.35	16.9%
214	2.4	0.70	1.52	2069	14.1	0.33	14.2%
170	6.0	0.97	1.97	2673	16.8	0.52	8.6%
35	4.2	0.84	1.76	1798	15.6	0.32	7.6%
199	6.1	0.88	1.86	2207	16.2	0.41	6.8%
171	7.6	1.25	2.35	2316	19.1	0.51	6.7%
198	6.6	1.04	2.04	2123	17.3	0.42	6.4%
196	6.4	1.04	2.04	1924	17.2	0.38	6.0%
197	8.8	1.29	2.35	2261	19.1	0.50	5.7%
203	10.8	1.11	2.23	2336	18.4	0.49	4.5%
204	6.2	0.71	1.69	1569	15.1	0.27	4.4%
212	16.3	1.45	2.63	3036	20.8	0.72	4.4%
202	19.4	1.37	2.63	3564	20.8	0.85	4.4%
207	13.3	1.13	2.29	2707	18.7	0.58	4.4%
172	21.5	1.55	2.84	2865	22.0	0.72	3.4%
16	30.3	1.98	3.43	2972	25.6	0.87	2.9%

Table A.10: Minimum Longitudinal Slope Model Catchments and Swales

Catchments		*Swales*					*Area Constraint*
ID	Area, A (Ac)	Check Dam Height (ft)	Depth (ft)	Length, L (ft)	Top Width (ft)	Area, A_s (Ac)	A_s/A
191	1.1	0.60	1.38	1318	13.3	0.20	18.8%
179	2.1	0.96	1.84	1729	16.1	0.32	15.4%
214	2.4	0.86	1.74	1722	15.4	0.30	13.0%
170	6.0	1.18	2.27	2133	18.6	0.46	7.6%
35	4.2	1.02	2.03	1432	17.2	0.28	6.7%
199	6.1	1.08	2.14	1725	17.9	0.35	5.8%
171	7.6	1.52	2.72	1820	21.3	0.45	5.9%
198	6.6	1.26	2.36	1764	19.2	0.39	5.9%
196	6.4	1.27	2.35	1518	19.1	0.33	5.2%
197	8.8	1.56	2.73	1877	21.4	0.46	5.2%
203	10.8	1.35	2.57	1892	20.4	0.44	4.1%
204	6.2	0.88	1.94	1225	16.6	0.23	3.8%
212	16.3	1.75	3.05	2443	23.3	0.65	4.0%
202	19.4	1.66	3.04	2982	23.3	0.80	4.1%
207	13.3	1.37	2.65	2191	20.9	0.53	3.9%
172	21.5	1.87	3.29	2239	24.7	0.64	3.0%
16	30.3	2.37	3.98	2374	28.9	0.79	2.6%

Table A.11: Maximum Longitudinal Slope Model Catchments and Swales

Catchments		*Swales*					*Area Constraint*
ID	Area, A (Ac)	Check Dam Height (ft)	Depth (ft)	Length, L (ft)	Top Width (ft)	Area, A_s (Ac)	A_s/A
191	1.1	0.33	1.00	2514	11.0	0.32	29.7%
179	2.1	0.55	1.28	3249	12.7	0.47	22.9%
214	2.4	0.49	1.22	3249	12.3	0.46	19.5%
170	6.0	0.69	1.55	4247	14.3	0.70	11.7%
35	4.2	0.59	1.40	2814	13.4	0.43	10.3%
199	6.1	0.62	1.47	3440	13.8	0.55	9.0%
171	7.6	0.89	1.84	3676	16.0	0.68	9.0%
198	6.6	0.73	1.61	3347	14.7	0.56	8.5%
196	6.4	0.73	1.60	3012	14.6	0.51	7.9%
197	8.8	0.92	1.84	3546	16.1	0.65	7.4%
203	10.8	0.79	1.75	3724	15.5	0.66	6.1%
204	6.2	0.50	1.34	2460	13.1	0.37	5.9%
212	16.3	1.04	2.05	4815	17.3	0.96	5.9%
202	19.4	0.98	2.05	5656	17.3	1.12	5.8%
207	13.3	0.80	1.79	4277	15.8	0.77	5.8%
172	21.5	1.11	2.21	4533	18.3	0.95	4.4%
16	30.3	1.44	2.66	4820	21.0	1.16	3.8%

Appendix B

Model Calibration

The 2009 calibration events were assessed in a previous modeling study (Hixon, 2009). The 2010 and 2011 rainfall event analysis provides further assessment of the existing conditions model. The events used to calibrate the model were chosen on the basis of approximately 1-2 inches of cumulative rainfall over a 24-hour duration. This level of rainfall accumulation was chosen to minimize node surcharging while also avoiding relatively low flow conditions. Modeled and observed hydrographs were assessed by comparing peak flow rate, volume, and the Nash-Sutcliffe "goodness of fit" coefficient (Nash and Sutcliffe, 1970). The results of these comparisons are summarized in Table B.1, and the modeled and measured flows are outlined in the following tables. Graphical comparisons between the measured and modeled flows are depicted in the following figures.

The events exhibiting the best Nash-Sutcliffe coefficient (R_{NS}^2) were those with the shortest lag time, or the time between the center of mass of rainfall excess and the time of peak discharge (McCuen 2005). Differences between measured and modeled flows were likely due to one or a combination of the following: 1) spatial variance in rainfall depth and/or intensity across the watershed, 2) location of the rain gauge with respect to the watershed, and/or 3) debris build-up and/or inadequate maintenance and calibration leading to instrumentation error at the outlet flow gauge.

Table B.1: Model Calibration Summary

Storm Date	5/15/2009	5/26/2009	7/17/2009	8/5/2009
Precipitation Depth (in)	0.91	1.17	1.05	1.04
Modeled Volume (cf)	136,800	222,100	199,300	197,500
Measured Volume (cf)	160,200	200,200	192,800	99,100
Modeled Peak (cfs)	25.6	23.7	47.2	35.3
Measured Peak (cfs)	38.0	24.6	46.6	26.4
Volume Difference	14.6%	-11%	-3%	-99%
Peak Difference	49%	4%	-1%	-25%
R_{NS}^2	0.80	0.85	-0.64	-0.70

Storm Date	10/27/2010	12/1/2010	5/22/2011	7/4/2011
Precipitation Depth (in)	0.85	1.81	0.86	0.92
Modeled Volume (cf)	96,000	701,900	114,300	127,200
Measured Volume (cf)	132,100	393,000	158,800	123,300
Modeled Peak (cfs)	23.7	47.2	35.3	29.8
Measured Peak (cfs)	25.3	41.4	28.7	25.4
Volume Difference	27%	-79%	28%	-3%
Peak Difference	7%	-12%	-19%	-15%
R_{NS}^2	-0.30	-0.71	0.82	0.93

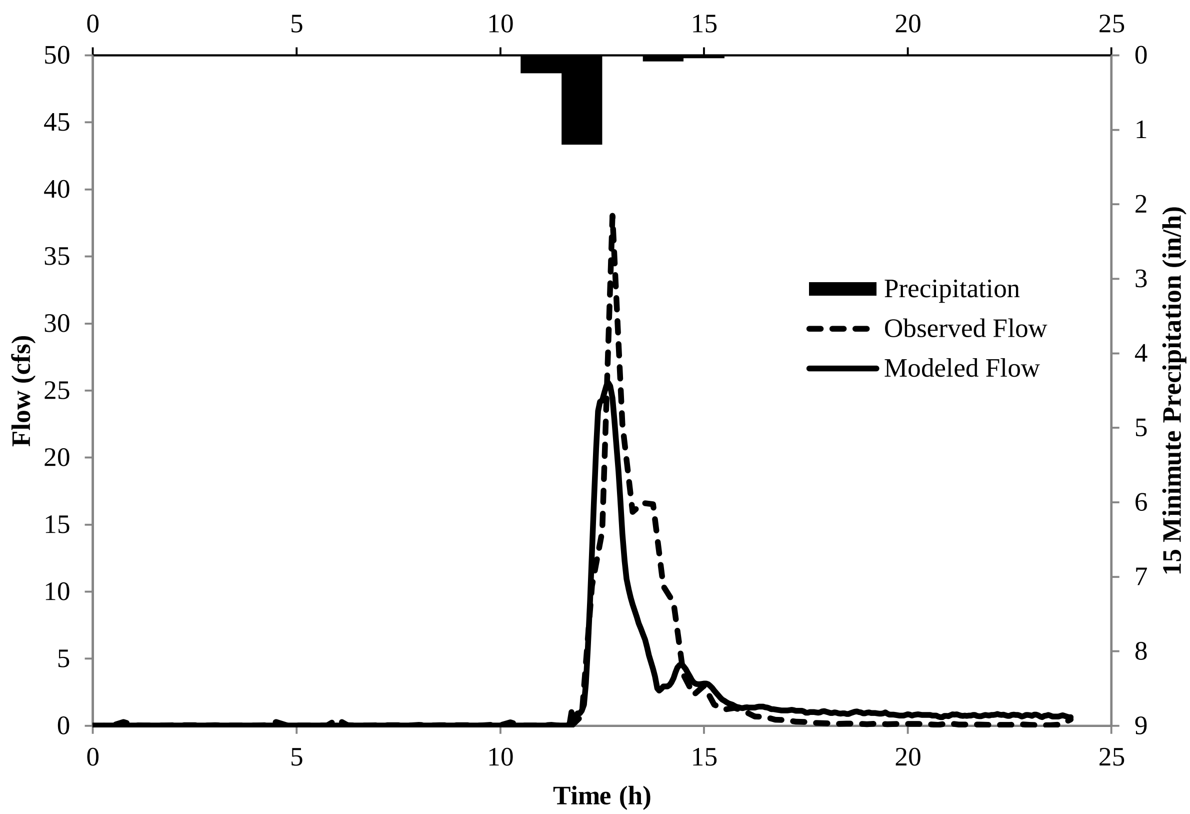

Figure B.1: May 15, 2009 Event

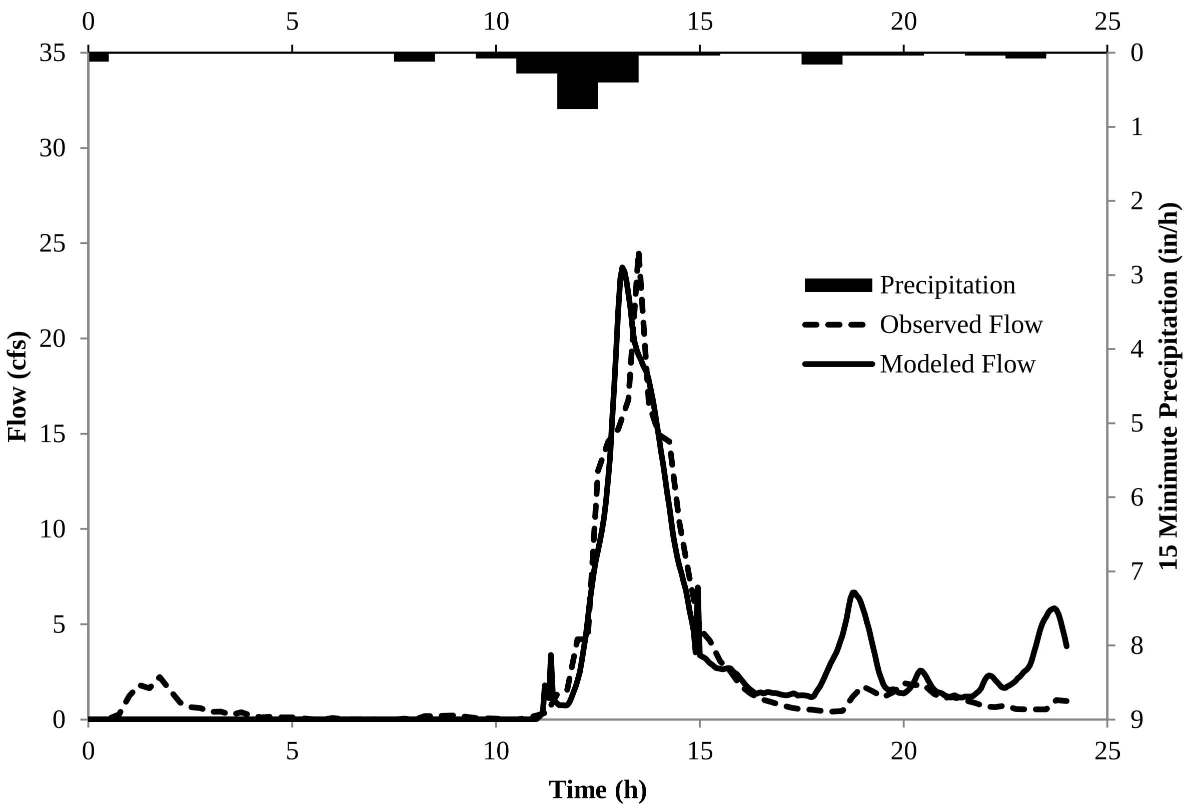

Figure B.2: May 26, 2009 Event

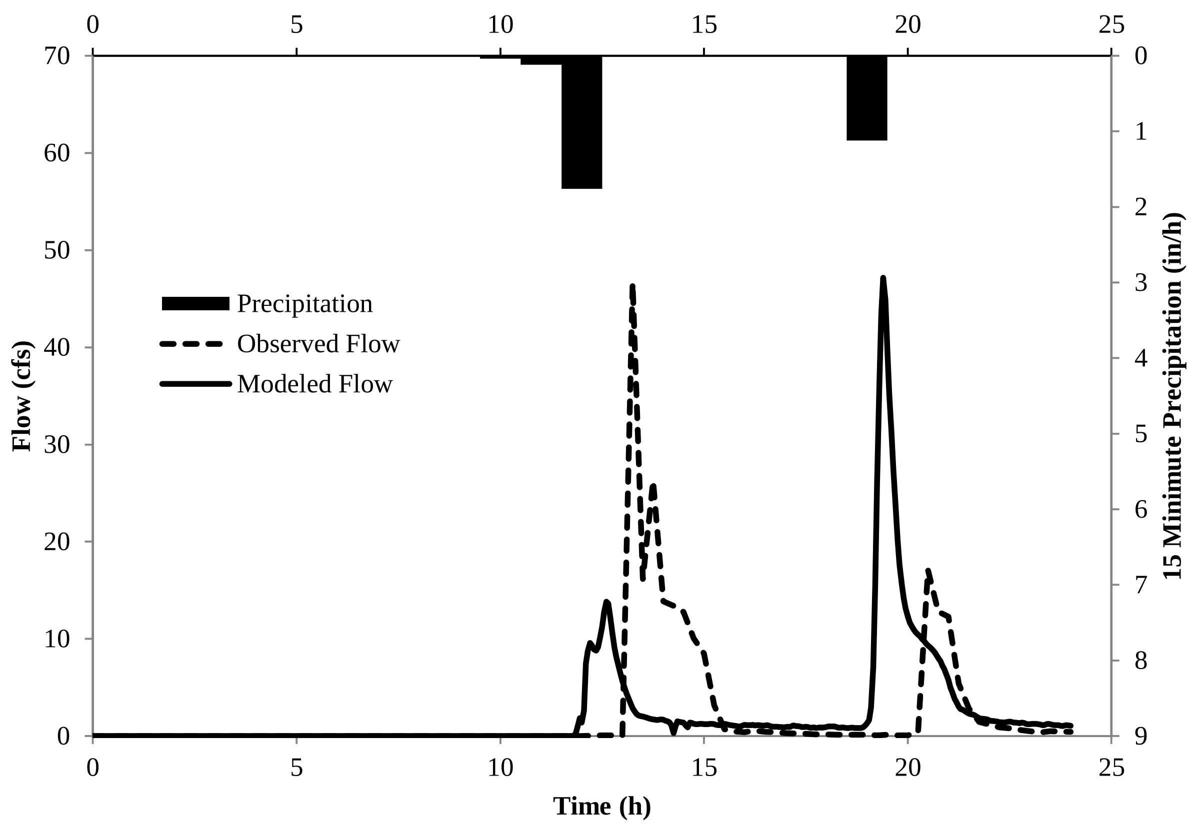

Figure B.3: July 17, 2009 Event

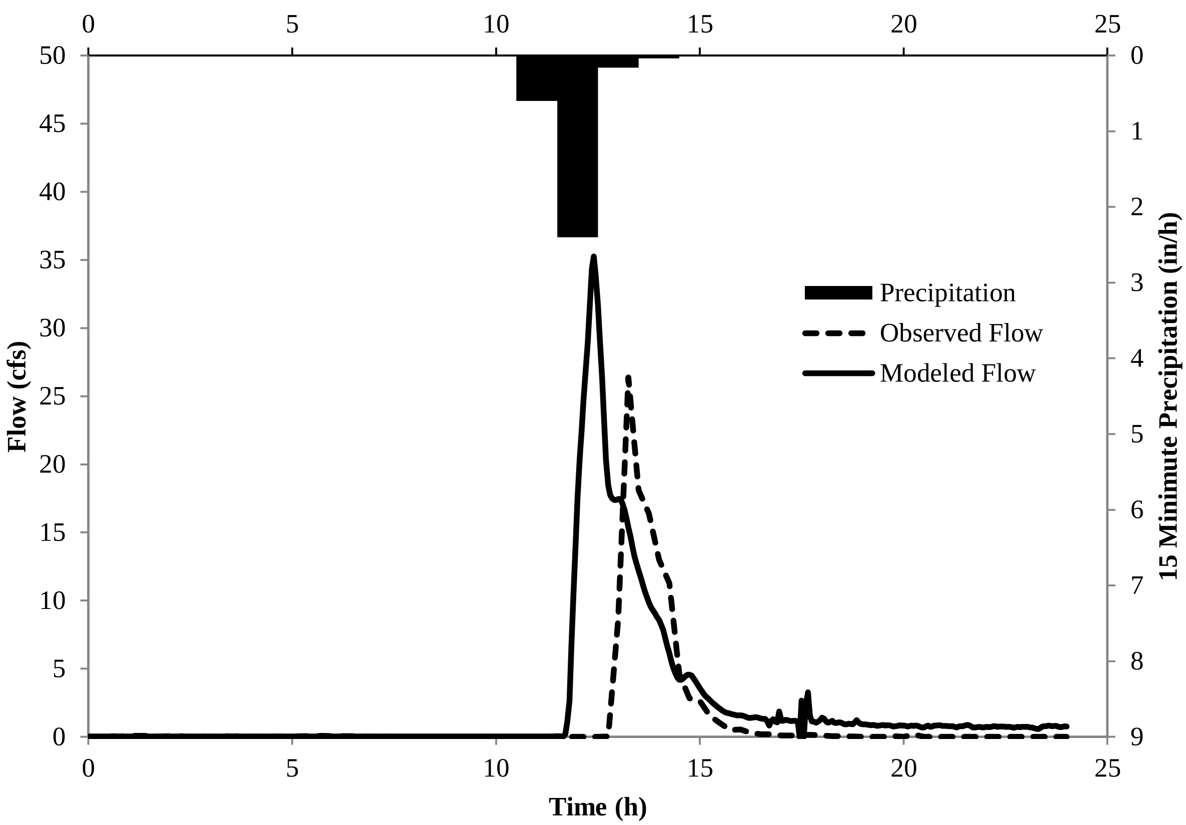

Figure B.4: August 5, 2009 Event

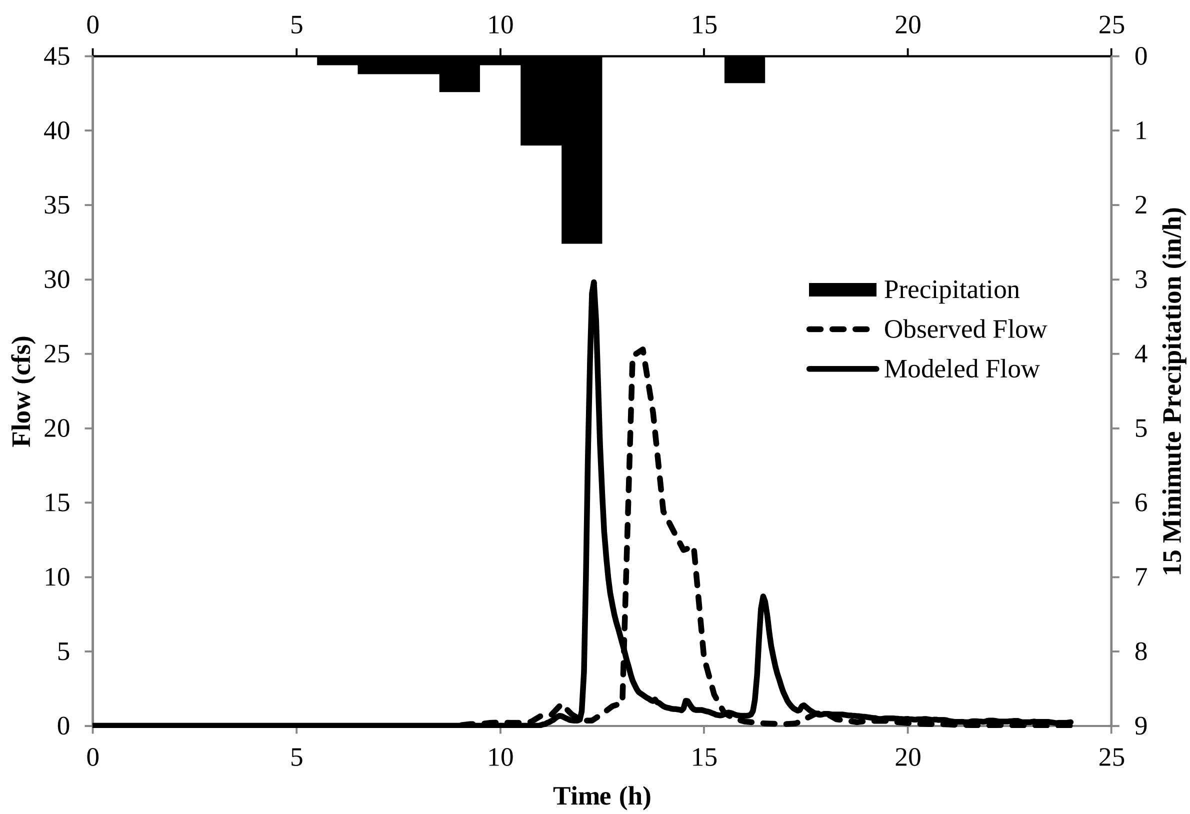

Figure B.5: October 27, 2010 Event

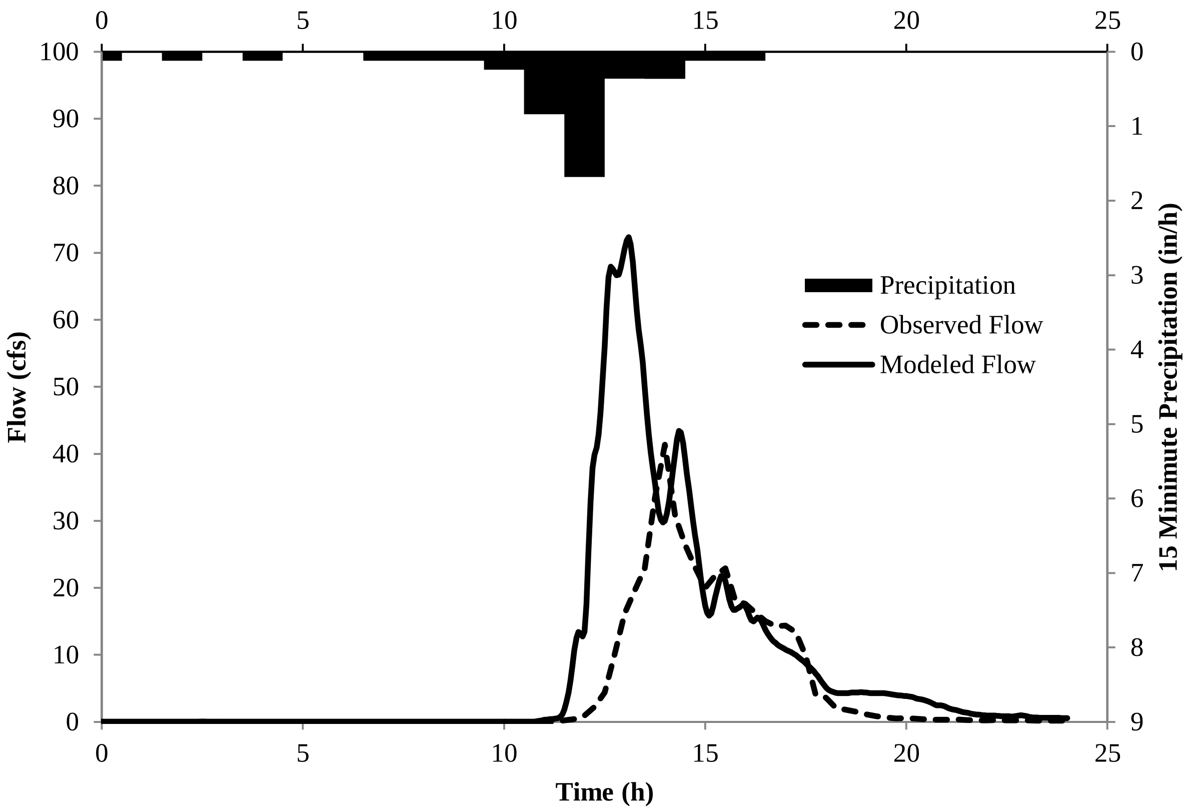

Figure B.6: December 1, 2010 Event

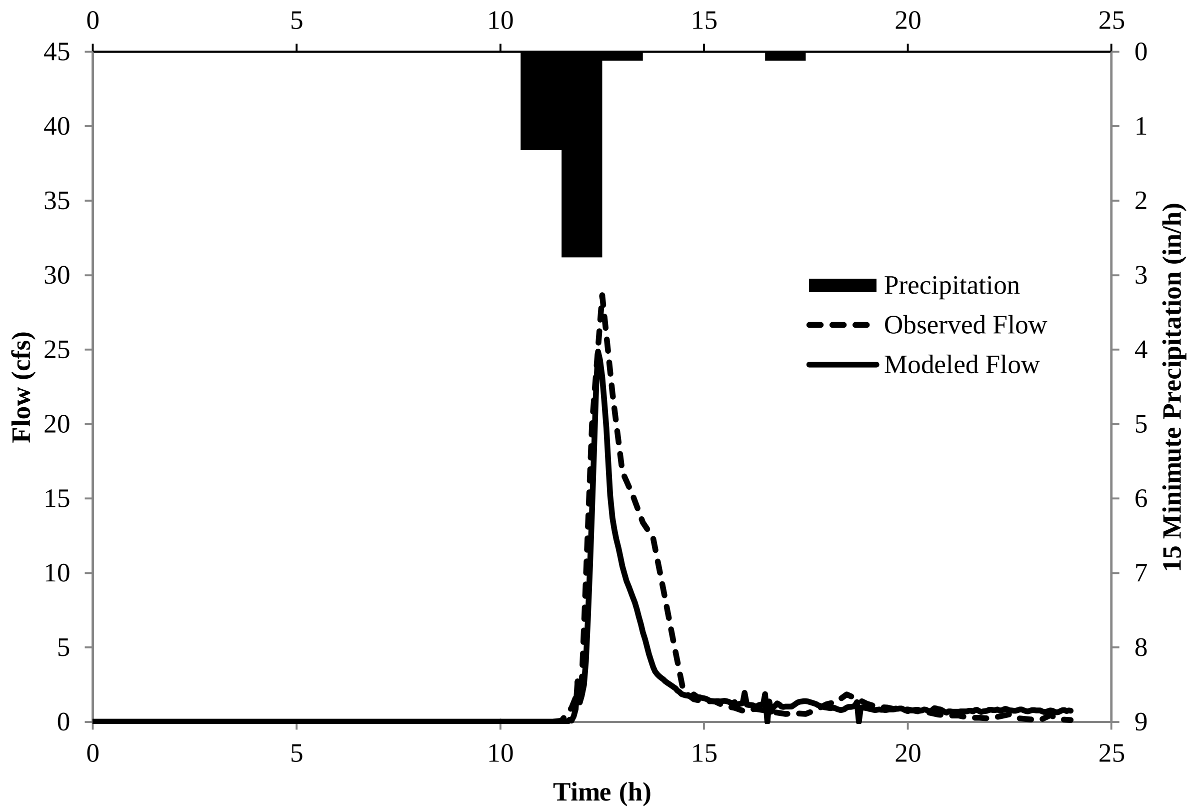

Figure B.7: May 22, 2011 Event

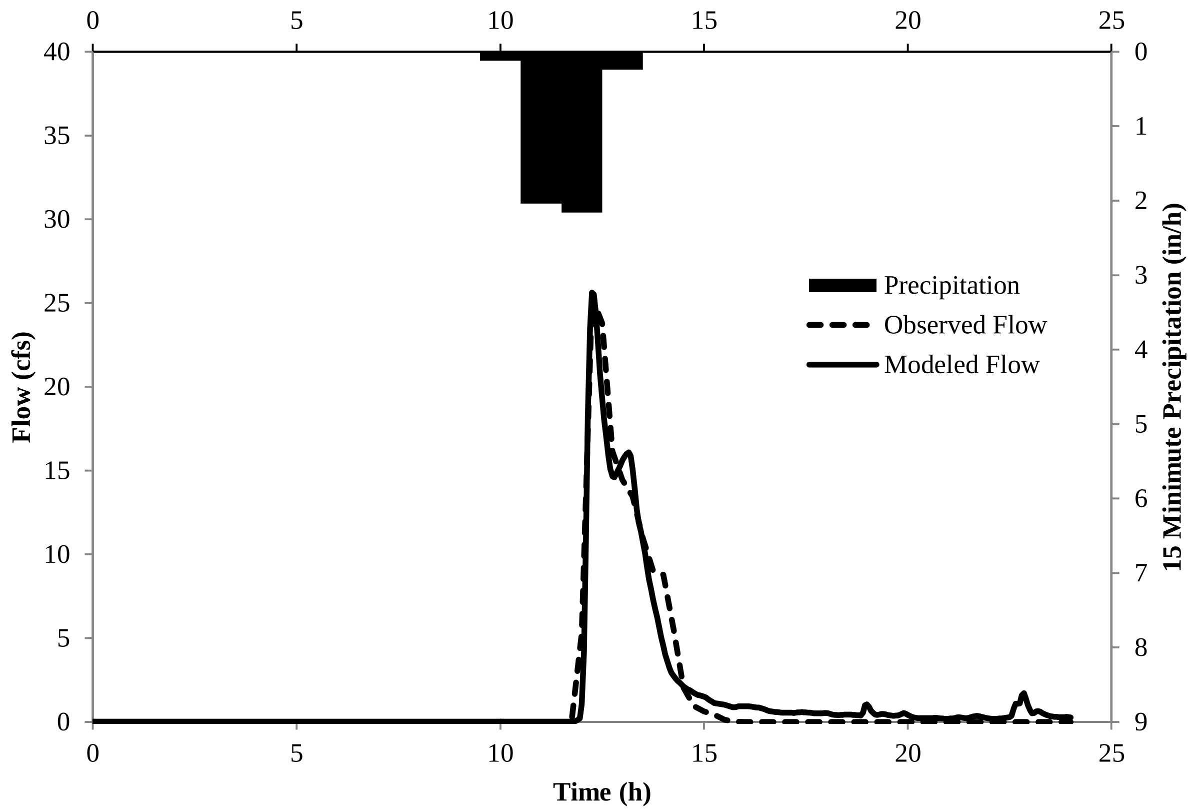

Figure B.8: July 4, 2011 Event

Table B.2: 2009 Event Flow Data (cfs)

Storm Date	5/15/09		5/26/09		7/17/09		8/5/09	
Time (hours)	*Meas.*	*Mod.*	*Meas.*	*Mod.*	*Meas.*	*Mod.*	*Meas.*	*Mod.*
0.00	0.00	0.02	0.00	0.02	0.00	0.02	0.00	0.02
0.25	0.00	0.02	0.00	0.02	0.00	0.02	0.00	0.02
0.50	0.05	0.02	0.04	0.02	0.00	0.02	0.04	0.02
0.75	0.29	0.02	0.27	0.02	0.00	0.02	0.02	0.02
1.00	0.05	0.02	1.25	0.02	0.00	0.02	0.07	0.02
1.25	0.02	0.02	1.80	0.02	0.00	0.02	0.07	0.02
1.50	0.02	0.02	1.65	0.02	0.00	0.02	0.07	0.02
1.75	0.02	0.02	2.23	0.02	0.00	0.02	0.02	0.02
2.00	0.05	0.02	1.54	0.02	0.00	0.02	0.04	0.02
2.25	0.05	0.02	0.89	0.02	0.00	0.02	0.04	0.02
2.50	0.05	0.02	0.65	0.02	0.00	0.02	0.00	0.02
2.75	0.02	0.02	0.60	0.02	0.00	0.02	0.04	0.02
3.00	0.05	0.02	0.40	0.02	0.00	0.02	0.00	0.02
3.25	0.02	0.02	0.42	0.02	0.00	0.02	0.07	0.02
3.50	0.02	0.02	0.25	0.02	0.00	0.02	0.02	0.02
3.75	0.02	0.02	0.38	0.02	0.00	0.02	0.04	0.02
4.00	0.02	0.02	0.20	0.02	0.00	0.02	0.00	0.02
4.25	0.05	0.02	0.11	0.02	0.00	0.02	0.02	0.02
4.50	0.29	0.02	0.16	0.02	0.00	0.02	0.02	0.02
4.75	0.05	0.02	0.11	0.02	0.00	0.02	0.02	0.02
5.00	0.02	0.02	0.11	0.02	0.00	0.02	0.02	0.02
5.25	0.02	0.02	0.07	0.02	0.00	0.02	0.04	0.02
5.50	0.02	0.02	0.02	0.02	0.00	0.02	0.04	0.02
5.75	0.02	0.02	0.00	0.02	0.00	0.02	0.07	0.02
6.00	0.47	0.02	0.09	0.02	0.00	0.02	0.04	0.02
6.25	0.05	0.02	0.02	0.02	0.00	0.02	0.04	0.02
6.50	0.02	0.02	0.02	0.02	0.00	0.02	0.04	0.02
6.75	0.02	0.02	0.02	0.02	0.00	0.02	0.02	0.02
7.00	0.05	0.02	0.02	0.02	0.00	0.02	0.00	0.02
7.25	0.05	0.02	0.00	0.02	0.00	0.02	0.02	0.02
7.50	0.05	0.02	0.00	0.02	0.00	0.02	0.00	0.02
7.75	0.02	0.02	0.04	0.02	0.00	0.02	0.00	0.02
8.00	0.07	0.02	0.00	0.02	0.00	0.02	0.00	0.02
8.25	0.02	0.02	0.18	0.02	0.00	0.02	0.00	0.02
8.50	0.05	0.02	0.18	0.02	0.00	0.02	0.00	0.02
8.75	0.05	0.02	0.20	0.02	0.02	0.02	0.00	0.02

Table B.2 - Continued

Storm Date	5/15/09		5/26/09		7/17/09		8/5/09	
Time (hours)	*Meas.*	*Mod.*	*Meas.*	*Mod.*	*Meas.*	*Mod.*	*Meas.*	*Mod.*
9.00	0.05	0.02	0.22	0.02	0.00	0.02	0.00	0.02
9.25	0.05	0.02	0.16	0.02	0.00	0.02	0.00	0.02
9.50	0.02	0.02	0.09	0.02	0.00	0.02	0.00	0.02
9.75	0.07	0.02	0.07	0.02	0.00	0.02	0.00	0.02
10.00	0.05	0.02	0.04	0.02	0.00	0.02	0.00	0.02
10.25	0.27	0.02	0.02	0.02	0.00	0.02	0.00	0.02
10.50	0.02	0.02	0.00	0.02	0.00	0.02	0.00	0.02
10.75	0.02	0.02	0.09	0.02	0.00	0.02	0.00	0.02
11.00	0.02	0.02	0.22	0.02	0.00	0.02	0.00	0.02
11.25	0.07	0.02	0.38	0.83	0.00	0.02	0.00	0.02
11.50	0.00	0.02	1.34	1.45	0.00	0.02	0.04	0.02
11.75	0.00	0.25	1.56	0.76	0.00	0.02	0.00	0.27
12.00	0.74	0.81	4.21	1.44	0.00	0.89	0.00	10.20
12.25	10.43	6.54	4.21	3.76	0.04	7.55	0.00	24.73
12.50	14.48	21.85	13.06	7.60	0.07	9.65	0.00	33.32
12.75	38.03	25.10	14.59	10.81	0.07	12.62	0.02	23.50
13.00	22.53	18.76	15.24	17.57	0.07	7.32	8.58	17.49
13.25	15.95	10.40	16.78	23.16	46.61	3.87	26.38	16.53
13.50	16.62	7.67	24.60	20.08	16.18	2.18	18.09	13.34
13.75	16.51	5.29	16.58	18.38	26.27	1.83	16.40	10.74
14.00	10.41	2.95	14.95	16.09	13.86	1.69	12.99	9.06
14.25	9.20	3.14	14.59	12.62	13.41	1.19	11.27	7.24
14.50	3.79	4.38	10.38	9.13	12.77	1.35	4.61	4.77
14.75	2.32	3.69	7.44	6.76	10.05	1.20	2.83	4.42
15.00	2.99	3.12	4.77	4.73	8.51	1.24	2.63	4.05
15.25	1.58	2.93	4.12	3.15	3.16	1.24	1.54	2.99
15.50	1.20	2.12	3.05	2.74	0.69	1.18	1.00	2.29
15.75	1.34	1.62	2.52	2.67	0.47	1.13	0.49	1.80
16.00	1.07	1.36	1.78	2.37	0.40	1.05	0.56	1.61
16.25	0.69	1.37	1.36	1.77	0.56	1.13	0.29	1.46
16.50	0.67	1.42	1.07	1.41	0.42	1.10	0.20	1.40
16.75	0.45	1.27	0.91	1.41	0.40	1.02	0.20	1.16
17.00	0.42	1.16	0.76	1.36	0.31	0.93	0.11	1.31
17.25	0.31	1.16	0.62	1.30	0.27	1.01	0.11	1.20
17.50	0.29	1.09	0.53	1.30	0.22	0.96	0.16	1.23
17.75	0.20	1.01	0.51	1.22	0.18	0.89	0.16	1.64
18.00	0.18	1.04	0.45	1.58	0.18	0.89	0.09	1.18

Table B.2 - Continued

Storm Date	5/15/09		5/26/09		7/17/09		8/5/09	
Time (hours)	*Meas.*	*Mod.*	*Meas.*	*Mod.*	*Meas.*	*Mod.*	*Meas.*	*Mod.*
18.25	0.13	0.98	0.42	2.66	0.13	0.98	0.04	1.15
18.50	0.16	0.92	0.45	3.83	0.13	0.86	0.04	1.04
18.75	0.16	0.99	1.20	5.82	0.13	0.84	0.02	0.94
19.00	0.11	0.97	1.74	6.31	0.13	0.99	0.00	1.04
19.25	0.16	0.96	1.47	4.68	0.09	10.64	0.00	0.89
19.50	0.11	0.92	1.18	2.54	0.13	42.26	0.00	0.84
19.75	0.13	0.82	1.45	1.60	0.09	27.50	0.04	0.81
20.00	0.13	0.79	1.92	1.43	0.09	14.59	0.00	0.82
20.25	0.13	0.82	1.80	1.66	0.40	11.00	0.18	0.80
20.50	0.11	0.81	1.80	2.43	17.09	9.78	0.00	0.75
20.75	0.07	0.75	1.34	1.89	12.81	8.62	0.00	0.79
21.00	0.18	0.69	1.11	1.39	12.28	6.78	0.00	0.83
21.25	0.09	0.83	1.27	1.18	5.41	3.95	0.00	0.78
21.50	0.09	0.75	1.00	1.17	2.85	2.56	0.00	0.77
21.75	0.09	0.77	0.87	1.23	1.45	2.08	0.00	0.77
22.00	0.07	0.76	0.69	1.70	1.18	1.73	0.00	0.71
22.25	0.07	0.83	0.65	2.23	0.89	1.51	0.00	0.75
22.50	0.07	0.78	0.74	1.76	0.80	1.43	0.00	0.75
22.75	0.11	0.80	0.56	1.87	0.62	1.39	0.00	0.71
23.00	0.07	0.76	0.53	2.37	0.51	1.29	0.00	0.73
23.25	0.05	0.78	0.53	3.23	0.36	1.24	0.00	0.67
23.50	0.05	0.74	0.53	4.93	0.51	1.20	0.00	0.71
23.75	0.09	0.70	1.02	5.76	0.47	1.12	0.00	0.80
24.00	0.47	0.70	0.98	4.73	0.45	1.08	0.00	0.74

Table B.3: 2010 and 2011 Event Flow Data (cfs)

Storm Date	10/27/10		12/1/10		5/22/11		7/4/2011	
Time (hours)	*Meas.*	*Mod.*	*Meas.*	*Mod.*	*Meas.*	*Mod.*	*Meas.*	*Mod.*
0.00	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
0.25	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
0.50	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
0.75	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
1.00	0.00	0.01	0.01	0.01	0.00	0.02	0.00	0.01
1.25	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
1.50	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
1.75	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
2.00	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
2.25	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
2.50	0.00	0.01	0.01	0.01	0.00	0.02	0.00	0.01
2.75	0.00	0.01	0.02	0.01	0.00	0.02	0.00	0.01
3.00	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
3.25	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
3.50	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
3.75	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
4.00	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
4.25	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
4.50	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
4.75	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
5.00	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
5.25	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
5.50	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
5.75	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
6.00	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
6.25	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
6.50	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
6.75	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
7.00	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
7.25	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
7.50	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
7.75	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
8.00	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
8.25	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
8.50	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01
8.75	0.00	0.01	0.00	0.01	0.00	0.02	0.00	0.01

Table B.3 - Continued

Storm Date	10/27/10		12/1/10		5/22/11		7/4/2011	
Time (hours)	*Meas.*	*Mod.*	*Meas.*	*Mod.*	*Meas.*	*Mod.*	*Meas.*	*Mod.*
9.00	0.04	0.01	0.00	0.01	0.00	0.02	0.00	0.01
9.25	0.11	0.01	0.00	0.01	0.00	0.02	0.00	0.01
9.50	0.11	0.01	0.00	0.01	0.00	0.02	0.00	0.01
9.75	0.22	0.01	0.00	0.01	0.00	0.02	0.00	0.01
10.00	0.22	0.01	0.00	0.01	0.00	0.02	0.00	0.01
10.25	0.22	0.01	0.00	0.01	0.00	0.02	0.00	0.01
10.50	0.22	0.01	0.00	0.01	0.00	0.02	0.00	0.01
10.75	0.27	0.01	0.00	0.02	0.00	0.02	0.00	0.01
11.00	0.68	0.02	0.01	0.18	0.00	0.02	0.00	0.01
11.25	0.74	0.20	0.03	0.40	0.00	0.02	0.00	0.01
11.50	1.47	0.58	0.21	0.95	0.08	0.02	0.00	0.01
11.75	0.79	0.47	0.39	6.47	0.93	0.06	0.00	0.01
12.00	0.36	0.50	0.94	13.07	2.64	1.40	5.30	0.28
12.25	0.36	17.01	2.21	30.67	19.96	7.58	25.36	16.47
12.50	0.81	22.94	4.40	47.36	28.65	22.54	23.80	22.47
12.75	1.31	10.26	10.06	66.21	22.08	17.54	16.14	16.13
13.00	1.54	6.48	16.15	68.16	16.78	11.67	14.44	15.08
13.25	24.86	3.98	19.62	69.82	15.25	9.12	13.44	15.78
13.50	25.30	2.34	22.86	55.94	13.38	7.05	10.96	12.11
13.75	21.05	1.81	33.49	40.48	12.37	4.58	9.02	8.60
14.00	14.43	1.53	41.35	30.89	8.90	3.09	8.78	5.65
14.25	13.09	1.19	30.74	35.28	5.35	2.53	5.53	3.30
14.50	11.82	1.11	26.46	41.95	2.04	2.00	2.03	2.35
14.75	12.05	1.43	23.11	32.27	1.52	1.76	0.96	1.87
15.00	4.59	1.06	20.00	21.34	1.37	1.65	0.62	1.58
15.25	2.11	0.91	21.83	16.83	1.39	1.44	0.44	1.28
15.50	0.85	0.74	22.92	21.10	1.08	1.39	0.13	1.06
15.75	0.49	0.85	18.11	17.73	0.93	1.32	0.01	0.92
16.00	0.29	0.69	17.63	17.21	0.71	1.33	0.00	0.92
16.25	0.24	0.98	16.26	15.58	0.88	1.12	0.00	0.91
16.50	0.17	6.84	15.05	14.82	0.76	1.28	0.00	0.81
16.75	0.15	5.53	14.27	12.41	0.63	0.83	0.00	0.63
17.00	0.11	2.69	14.36	11.14	0.52	1.09	0.00	0.55
17.25	0.16	1.34	13.37	10.32	0.57	1.08	0.00	0.54
17.50	0.49	1.22	9.80	9.25	0.53	1.35	0.00	0.57
17.75	0.82	0.97	4.03	7.85	0.82	1.28	0.00	0.52
18.00	0.83	0.78	3.61	5.96	1.17	1.03	0.00	0.51

Table B.3 - Continued

Storm Date	10/27/10		12/1/10		5/22/11		7/4/2011	
Time (hours)	*Meas.*	*Mod.*	*Meas.*	*Mod.*	*Meas.*	*Mod.*	*Meas.*	*Mod.*
18.25	0.45	0.77	2.10	4.56	1.34	0.92	0.00	0.45
18.50	0.35	0.75	1.80	4.28	1.85	0.85	0.00	0.42
18.75	0.25	0.69	1.49	4.36	1.54	1.08	0.00	0.42
19.00	0.33	0.62	1.15	4.40	1.23	0.76	0.00	0.67
19.25	0.34	0.54	0.85	4.28	1.01	0.83	0.00	0.59
19.50	0.33	0.48	0.67	4.27	0.95	0.81	0.00	0.45
19.75	0.26	0.49	0.51	4.11	0.82	0.85	0.00	0.37
20.00	0.18	0.46	0.56	3.90	0.83	0.82	0.00	0.46
20.25	0.15	0.44	0.47	3.67	0.71	0.78	0.00	0.27
20.50	0.13	0.45	0.38	3.29	0.65	0.78	0.00	0.22
20.75	0.12	0.42	0.31	2.75	0.49	0.83	0.00	0.23
21.00	0.10	0.38	0.30	2.38	0.42	0.72	0.00	0.19
21.25	0.06	0.29	0.35	1.88	0.42	0.69	0.00	0.23
21.50	0.06	0.27	0.27	1.50	0.28	0.71	0.00	0.24
21.75	0.04	0.31	0.22	1.19	0.27	0.75	0.00	0.32
22.00	0.04	0.31	0.24	1.00	0.22	0.73	0.00	0.25
22.25	0.05	0.34	0.28	0.93	0.36	0.80	0.00	0.19
22.50	0.04	0.30	0.22	0.85	0.50	0.81	0.00	0.24
22.75	0.04	0.32	0.23	0.83	0.24	0.77	0.00	0.88
23.00	0.03	0.25	0.23	0.92	0.16	0.75	0.00	1.27
23.25	0.03	0.28	0.16	0.67	0.13	0.77	0.00	0.58
23.50	0.02	0.27	0.16	0.60	0.44	0.71	0.00	0.43
23.75	0.02	0.21	0.16	0.59	0.16	0.69	0.00	0.29
24.00	0.02	0.22	0.19	0.56	0.12	0.76	0.00	0.28