AN ANALYSIS OF THE SCS METHOD IN THE SIMULATION OF STORMWATER DISCONNECTION IN AN URBAN WATERSHED

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ABSTRACT OF THE THESIS

An Analysis of the SCS Method in the Simulation of Stormwater Disconnection in an Urban Watershed

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Small storms can have adverse affects on downstream water quality in urbanizing watersheds because impervious surfaces convey greater volumes of runoff and lead to larger storm flows than under natural conditions (Booth 1990; Beard and Chang 1979). Therefore management of the water quality storm (1.25" of rain over 2 hours) has been targeted in water quality initiatives. This study examined whether reducing the effective impervious area that contributes runoff during the water quality storm by disconnecting it from the stormwater conveyance system could be a viable stormwater management solution in existing residential areas. Disconnection was examined in the Pompeston Creek Watershed, Burlington County, New Jersey on the lot, subdivision, and watershed scale. A calibrated HEC-HMS model of the watershed was used for the watershed scale analysis. The SCS equations to were applied to simulate disconnection by routing runoff from the disconnected impervious surface over an adjacent impervious surface. The 2, 10, and 100-year storms were examined in addition to the water quality storm. Three primary conclusions were made: 1) the composite curve number method, and therefore the composite curve numbers

given in TR-55, under-predicts small storm runoff when compared to the volume weighted method because the composite curve number does not account for the runoff conveyed by directly connected impervious surfaces in urbanizing areas; 2) by disconnecting the runoff from the impervious areas by routing it over the pervious area, the runoff volume can be reduced for the water quality storm; 3) the effectiveness of disconnection in mitigating the runoff volumes relies on the infiltration capacity of the pervious area. Both the extent of the under-prediction of the composite curve number and the relative volume reduction achieved by disconnection decrease as storm depth increases. The adjustment of the basin curve number during model calibration for small observed storm events suggested that the original composite curve number method was inadequate in predicting runoff in the watershed scale model for small storms. Basin-wide reductions in runoff volumes with the application of basin-wide disconnection were consistent with the reductions predicted on the smaller scales. These results have a direct application to regional stormwater management planning as disconnection can be a suitable retrofit for the management of the small storm in existing residential areas.

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INTRODUCTION

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Urban development induces a variety of changes on a watershed that have profound impacts on local hydrologic processes, water quality, and downstream aquatic ecosystems. As the degree of urbanization increases in a watershed, watershed health diminishes. Wherever the natural landscape is converted to rooftops and roads and other paved surfaces for human use, the transport of water across the land surface is radically altered. The relationship between the impervious surface cover introduced by these land use alterations and the unintended adverse impacts on water quality, aquatic communities, habitat, and stream flow is a result of this disruption in runoff processes (Klein 1979; Schueler 1994; Arnold and Gibbons 1996; May et al. 1997). Urban areas with high levels of impervious surfaces tend to contribute a greater volume of runoff and elevated peak discharges, have shorter travel times, reduce groundwater recharge and baseflow, and at times, contribute more severe pollutant loadings than less urbanized areas.

The primary stormwater management concerns in urban areas are the downstream physical and biological effects of an increase in the peak rate and volume of runoff from the introduction of impervious surfaces and improved drainage systems (Leopold 1968; Klein 1979; Booth and Jackson 1997). Ecological systems are closely related to the hydrologic system and thus are very sensitive to changes in hydrology (Klein 1979; Booth 1991; May et. al 1997). One of the most detrimental hydrologic changes from an increase in urban stream flows is a decline in channel stability. Channels that receive increased stormwater loads tend to experience widening or incision. The erosion of the unstable banks jeopardizes riparian habitat and increases sediment loads into the stream system (Wolman 1967; Hammer 1972;

Booth 1990). Ecosystem damage occurs when downstream aquatic communities are buried by excess sediment (May et al. 1997). Urbanization increases the frequency and number of runoff producing events that stress stream banks (Hammer 1972; Hawkins 1975; Booth 1991).

Many urban areas throughout New Jersey are already highly developed and experience a multitude of water quantity and quality issues due to a lack of adequate stormwater management. Many of these areas are residential areas in need of stormwater management retrofits. There are also situations where downstream rehabilitation efforts, such as bank stabilization, will not be successful unless the stormwater contributions of upstream areas are addressed (Booth 1991). To address these issues, much effort has been put forth in the areas of watershed management, planning, and stormwater engineering to mitigate the downstream affects of increasing urbanization. A recent trend is to evaluate the stormwater management picture from a regional perspective. These regional plans are developed to address flooding and water quality problems and focus on fixing the causes of the problems on the watershed scale, instead of simply repairing the symptoms of the problems on the individual reach scale. The regional stormwater management plans described in the New Jersey Stormwater Management Rules (N.J.A.C. 7:8-3, 2004) typically involve preparation of a characterization study of the drainage area, establishment of stormwater goals and objectives, and development of site-specific objectives to address the stormwater issues of the watershed.

In the regional stormwater management process, hydrologic models are often employed to simulate the runoff conditions of the area, to identify areas of concern, and to test management solutions. Hydrologic models can be very powerful in simulating runoff from rainfall events over a watershed provided the model can accurately simulate the system and conditions. Imperviousness is an important parameter in hydrologic modeling and can be described differently by different models. The most widely used models rely on the Soil Conservation Service (SCS) equations that, in practice, define imperviousness in terms of a runoff curve number (SCS 1972). Variations in the method of curve number designation, as well as variations in the application of the curve number, affect the utility of SCS methods for use in effective stormwater management (Rawls, Shalaby, and McCuen 1981; Rallison and Miller 1982; Garen and Moore 2005).

Disconnection as a BMP

Successful achievement of the stormwater management goals outlined in regional planning initiatives in existing urban areas will require some degree of on-site retrofits to manage stormwater from residential areas. While there are a variety of structural best management practices (BMPs) available to treat stormwater (i.e. extended detention basins, stormwater wet ponds, stormwater wetlands, bioretention systems, vegetated swales, prefabricated treatment devices, and riparian stream buffers), many of these options are not available to the individual homeowner, or there is not sufficient land available to install a BMP to drain an entire subdivision (NJDEP 2004). Another issue is the cost of design, installation, and maintenance of a structural stormwater BMP. In these situations, on-site disconnection of stormwater from the stream drainage system can be a novel, cost effective solution to manage urban stormwater.

Disconnection of impervious surfaces can be used as a stormwater management strategy in residential areas for the small storm event. The effectiveness of disconnection in mitigating runoff volumes relies on the infiltration capacity of the pervious areas. While all stormwater BMPs use the concept of disconnection to reduce stormwater volume, decrease peak flows, and remove pollutants, disconnection in its simplest form involves routing runoff from an impervious surface over a pervious surface as sheet flow to promote infiltration. A connected impervious surface is defined as a surface that discharges stormwater runoff through piping or over another impervious surface until it reaches the receiving water; at no time along its travel from the surface to the stream does the runoff come in contact with a pervious surface. A disconnected impervious surface is one that discharges stormwater runoff as sheet flow over a pervious surface such as a lawn area, thereby providing the runoff an opportunity to be filtered and infiltrate. Although many BMPs are designed to promote storage or infiltration of large volumes of water, a pervious, vegetated area, such as a lawn or garden, can achieve the same effect for small storms when stormwater travels over these surfaces as sheet flow.

There are different hydrologic effects on a receiving stream when stormwater is directly conveyed from an impervious surface to the receiving stream than when the conveyance chain is disconnected somewhere along the way. When impervious surfaces are directly connected, runoff is quickly transported from impervious surface to the receiving stream with little or no losses. On many lots rooftop runoff is conveyed through a gutter and downspout, to a driveway, along a curbed street, into a catch basin, through the storm drainage network, and ultimately discharged to the receiving stream. In this directly connected example, any opportunity for reduction in volume by infiltration is bypassed.

In the mid-Atlantic region 90% of the storm events are less than 1.25" in total rainfall depth (Claytor and Schueler 1996). Due to their frequency, small storms are responsible for contributing a large amount of nonpoint source pollution (Leopold 1968; May et al. 1997).

Therefore, the design rainfall depth used in New Jersey is 1.25" over two hours for water quality purposes. It is called the water quality design storm. Since the small, frequent storm is of greatest water quality concern in urban watersheds, disconnection will be evaluated for its effectiveness at reducing the runoff contributions from these smaller storm events.

Total impervious area (TIA) is used in many models as an input parameter to describe imperviousness. However, this term can be misleading because it would also include impervious surfaces that are disconnected from the drainage network (Hill, Botsford, and Booth 2003). Rainfall falling on a disconnected surface may run off onto a pervious surface, and is prevented from entering the storm drainage system and ultimately the receiving stream. If the area of total impervious surface is used in runoff calculations, the result would not be accurate. A better representation of imperviousness throughout the drainage area would be to calculate the entire impervious surface that is hydraulically effective in conveying stormwater to the receiving water. This area can be referred to as the effective impervious area (EIA) (Beard and Chang 1979; Alley and Veenhuis 1983; Hill, Botsford, and Booth 2003). The effective impervious area does not include areas that are disconnected from the drainage area. By reducing the EIA of a watershed through disconnection, while the TIA remains the same, local and basin-wide stormwater discharges may be reduced.

Disconnection can be utilized by individual homeowners by simply rerouting rooftop downspouts to pervious areas instead of onto an impervious driveway. Splash blocks can be installed at the bottom of the downspout to dissipate energy and prevent erosion. Rooftop runoff can either be routed over a lawn area with pervious soils or into a rain garden (NJDEP 2004). A rain garden is a shallow landscaped depression that is designed to capture and infiltrate runoff from the water quality storm (1.25” over two hours). It is also possible to divert driveway runoff into a rain garden or over a pervious area. Streets account for the largest percent of impervious surface in most medium density residential areas (Lee and Heaney 2003). While stormwater contributed by streets cannot be managed by the individual homeowner, it is still possible to use disconnection to mange the stormwater contribution from residential streets. Tree boxes, which consist of an infiltration chamber that discharges to the ground, can be installed in urban areas to reduce stormwater volumes. Street runoff is routed to the tree box where it is infiltrated into the soil or is treated by the soil matrix within the box and is released slowly through an under drain system to the drainage network (www.lid.net, UNH Stormwater Center).

Purpose

The goal of this study was to devise a method for modeling disconnection for planning purposes and to examine whether disconnection could be an effective management strategy to control stormwater for the water quality storm in a residential setting. Disconnection was examined in three scenarios: first on the individual lot scale, then on the subdivision scale, and results from these scenarios were then used in a calibrated HEC-HMS model on the watershed scale. The lot scale scenario followed the method in the NJBMP manual which separates the lot into impervious and pervious sections. Disconnection was simulated by routing all the runoff generated from the impervious area over the pervious area. A new curve number was established to describe the disconnected runoff response of the lot. The SCS method was also investigated by comparing the volume predicted using the composite curve numbers included in the TR-55 model for urban areas to the volume predicted using the disconnection method on all three scales. A table of curve numbers based on land use, effective impervious area, and rainfall depth was developed for each study area. The relationship between EIA and disconnection was applied to the urban land use table in TR-55 to create new curve number tables for urban areas based on disconnection and rainfall depth. Streamflow and runoff data from the Pompeston Creek watershed were used to calibrate a HEC-HMS model. The model was used to compare the predicted volumes using the original composite curve numbers, to current observed volumes and the predicted volumes when the new disconnection curve numbers are used. The disconnection curve numbers generated on the subdivision scale were used to evaluate the effectiveness of disconnection in reducing the small storm runoff volumes on the watershed scale.

Calculated volumes and model generated hydrographs were used in the analysis to determine the effects disconnection may have on the overall runoff of the lot, subdivision, and watershed areas. The relationship between the percent curve number reduction and percent volume reduction for different residential land uses and degree of disconnection was examined on the three scales. Previous research shows that the curve number decreases as storm size increases (Bonelid, McCuen, and Jackson 1982; Hjlmfelt 1991; Hawkins 1993; Grove, Harbor, and Engel 1998; Kottegoda, Natale, and Raiteri 2000), and that the composite curve number does not accurately represent runoff for the smaller storm event (Grove, Harbor, and Engel 1998). Therefore, the new curve numbers generated for small storm situations may provide a better tool for the analysis and management of small storm runoff hydrology in urban areas.

Disconnection has the potential to play a role in the future of stormwater management as a low-cost alternative. Regional stormwater management planning has gained popularity in recent years as a tool to holistically look at basin-wide stormwater issues and pinpoint areas for rehabilitation that would have the greatest effect on improving water quality and reducing water quantity issues. An evaluation of disconnection as a management strategy should present another option for effective stormwater management in urban areas and improve regulatory guidance for the future.

LITERATURE REVIEW

Research on Watershed Impacts of Impervious Surface Cover

Research has been published that emphasizes the direct relationship between urbanization and a decline in hydrologic conditions. Literature highlights that the alteration of the natural drainage basin through increased urbanization can impose dramatic changes in the movement and storage of water which has adverse effects on many factors that contribute to ecologic function.

In 1968 the USGS published Circular 554 – Hydrology for Urban Land Planning –A Guidebook on the Hydrologic Effects of Urban Land Use (Leopold 1968). It was published in response to a need for guidance in applying current knowledge about the effects of urbanization on hydrology to test alternatives in making land use decisions. The document is a review of the current literature at the time regarding the effects of urbanization on the hydrologic conditions of a multitude of basins. It suggests that management alternatives that affect the hydrologic function of drainage basins should be expressed in terms of the hydrologic parameters that are affected by land use. Four interrelated, yet separable land-use changes on the hydrology of a basin are highlighted: 1) changes in peak flow characteristics; 2) changes in total runoff; 3) changes in water quality and; 4) changes in hydrologic amenities, which can be defined as an aesthetic value to an individual.

Leopold (1968) cites a number of studies that suggest that urbanization tends to increase the flood potential from a given basin. Studies are also included that find a direct relationship between the number of annually occurring bankfull flows and the extent of urbanization. It is stated that a direct impact of upstream urbanization is an observed increase in stormwater volumes and more frequent flooding. Degree of urbanization is measured as both a value of percent watershed imperviousness and a value of percent watershed area served by storm sewers. Leopold (1968) performed a frequency analysis of bankfull flows of the Brandywine Creek basin in Pennsylvania and predicted that the capacity of the channel would be exceeded more than an average of every 1.5 to 2 years. Flood-frequency curves were established for different degrees of urbanization throughout the basin. The number of flows for the original 1.5 year recurrence interval increased twofold when urbanization was increased from natural conditions to 20% sewered and 20% impervious area. For comparison, the number of bankfull flows either equal or exceeding bankfull capacity from an area that was 50% sewered and 50% impervious surface increased by a factor of 4.

Leopold (1968) also described the effects of urbanization and increases in sediment yield. Wolman (1967) documents that the sediment yield for small urbanizing, developed, or industrial basins can be 10 to 100 times the annual yield for rural areas. Wolman (1967) explores the changes in sediment yield and river channel behavior and their successive changes in land use. The concept of equilibrium was useful in dealing with the response of channel systems to significant changes in discharge and sediment load over short intervals of time. When larger volumes of water are introduced to a channel, the channel may enlarge until a new condition of equilibrium is achieved. However, Wolman (1967) points out that it is difficult to determine whether a new equilibrium will be established, or whether a condition of disequilibrium will persist. Urbanization affects the sediment loading of a stream channel in three stages: 1) initial, stable or equilibrium condition where the landscape is undeveloped, 2) period of construction during which bare land is exposed to erosion, 3) final stage where a new urban landscape is dominated by impervious surfaces such as rooftops and streets that are connected by gutters and sewers. The quantity of sediment from each stage was estimated from literature on drainage basins throughout the Middle Atlantic region of the United States. The increase in runoff volume and number of peak flows expected from urban areas coupled with the absence of sediment in urban runoff contributes to more rapid channel erosion and a subsequent increase in channel width.

Hammer (1972) studied the effect of urbanization on stream channel enlargement. The empirical study of 78 small watersheds near Philadelphia related increases in cross sectional areas to land use data. Large channel enlargement effects were observed for sewered streets and areas of large impervious surfaces such as parking lots. Smaller effects occurred in areas with unsewered streets and residential areas with detached houses. The influence of impervious development on channel size was significantly related to topographic watershed characteristics, the location of imperviousness, and man-made drainage alterations. However, the most critical determinant of channel enlargement in urban areas was due to slope. In the determination of impervious surface area, it was assumed that all streets with curbs were serviced by storm sewers. Both channel cross sectional area and ratio of channel width to depth were measured. An analysis of the channel enlargement ratio (empirical relationship between channel cross sectional area and watershed size for unurbanized areas) indicated that the channels of urbanized streams had enlarged relative to their pre-urbanized state. A special category of houses was given separate treatment in the study. These houses had direct, underground connections between gutter downspouts and the storm sewerage system. This only occurred within the city of Philadelphia, not in the surrounding suburbs. The estimated effect of channel enlargement in these areas was very high and similar to the effect of more impervious areas. Age of urban area was also significant in predicting the channel enlargement ratio. Houses between 15 and 30 years old had the greatest effect.

Hollis (1975) examined the effect of urbanization on floods of different recurrence intervals. Published streamflow data results were synthesized to show the general relationship between the increase in flood flows following urbanization and both the percent of the basin that is paved and the flood recurrence interval. Hollis (1975) came to four general conclusions: 1) floods with a return period of a year or longer are not affected by a 5% paving of the drainage area; 2) small floods may increase by 10 times as a result of urbanization; 3) floods with a return period of 100 years may be doubled in size by a 30% paving of the basin; and 4) the relative effect of urbanization declines as flood recurrence intervals increase. The effect of small modest storms that would otherwise be completely absorbed by soil storage in pervious areas was significant. The ratio of the pre-development peak discharge to the post-development discharge was plotted against the percent imperviousness of the basin and the recurrence interval of the pre-development flood. The effect of urbanization is seen to decline as the flood magnitude increases because of the reduced importance of interception, depression storage, and infiltration of undisturbed basins during severe rainfall events.

Booth (1991) emphasizes that the effect of watershed alterations on the stream channel and hydrology will be determined by the prevailing hydrologic regime. Where overland flow predominates in pre-urban conditions, excess precipitation reaches the stream channel under all storm events regardless of the level of urbanization. In this situation the underlying runoff processes have not been drastically altered. On the other hand, urbanization does alter the underlying runoff processes where subsurface flow predominates in the pre-urban condition. Before urbanization, most of the rainfall is lost to evaporation and transpiration, and the remaining volume slowly moves down gradient through the soil where most of it is retained in long-term storage. Most runoff never reaches the stream channel, instead, the stream is fed by baseflow. However, when the land surface is modified so precipitation can no longer reach the subsurface soil layer, overland flow begins to dominate, transport rates will increase many fold, and intervening storage is vastly reduced. Thus, in areas where subsurface flow prevails under predevelopment conditions, urbanization has a particularly dramatic effect on increasing the magnitude of runoff because the fundamental process of runoff generation is changed (Booth 1991).

May et al. (1997) identified linkages between landscape level conditions and instream environmental factors as they affect aquatic biota in the Puget Sound Lowland Eco-region. The goal was to provide a set of stream quality indices for local resource managers to use in managing urban streams and to minimize resource degradation from development pressures. Fine sediment sampling in the study areas indicated that urbanization can result in the degradation of streambed habitat. Higher than normal percentages of fine sediment were found in urbanized basins (May et al., 1997). When the total percent of impervious surface was related to the biological condition of the benthic macroinvertebrate community, a rapid decline in biotic integrity was seen with the onset of urbanization.

An earlier study by Booth (1990) explored the alterations on stream morphology following drainage basin alterations in rapidly urbanizing basins in King County, Washington. The study focuses on stream expansion and incision as a result of increases on peak flows and flow durations downstream of urban areas. Stream channel changes from an increased influx of runoff include increased channel width or depth, bank and side-hill mass failures, increased downstream sediment loading and loss of aquatic habitat. Booth (1990) modeled basins in the study area using the Hydrologic Simulation Program Fortran (HSPF), to display the hydrologic differences between relatively low and high levels of urbanization. The model is a deterministic, continuous, hydrologic model that simulates runoff and streamflow by keeping a running account of the amount of water within various hydrologic storage zones, both surface and subsurface. The model was calibrated with data from four basins gauged for streamflow and precipitation over a two year period. Two conclusions were drawn from the run of the model. 1) In highly urbanized basins, not only were major peaks amplified, but many smaller storms generated substantial flows. These smaller storms did not produce runoff in the predevelopment condition. 2) Discharge duration was extended. Durations in the urbanized basins were on an average from 30 to 100 times longer than in predevelopment conditions.

The effects of urbanization on moderate channel expansion were investigated by plotting a simple regression of channel area against the two year discharge modeled in HSPF. Modeled 2-year discharges were used to calculate changes in stream discharge from past land use changes in the watershed. Bankfull width and depth measurements along with vegetation, floodplain height, historic flooding inundation, and slope break observations were used to determine a cross sectional area and indicate the active channel dimensions. Then the channel enlargement at build-out could be estimated using the regression relationship. Booth (1990) also examined the case of catastrophic downcutting by a process of channel incision that is more aggressive than channel expansion. Channel incision is dependent on the amount of sediment that the flow can transport relative to the influx into the channel. Channel slope affects the shear stress equation of sediment transport, and grain size determines whether transport and erosion can occur, therefore, these two parameters dominate whether a channel is susceptible to incision. Booth (1990) screened basins for a potential to experience incision by identifying steep slopes and erodible soils. Urbanized areas contribute a high frequency of flows which can induce incision in these sensitive areas. Study results indicated that incision would not be prevented if only peak discharges from lower-frequency storms (10 to 100-year events) were controlled, which implies the most frequent storms would also need to be mitigated in urban areas (Booth 1990).

Booth (1990) also determined sediment loads from channel bank erosion by estimating channel widening due to increased urban stormwater flows. The linear regression made between the observed areas and 2-year discharge data was used to calculate the annual sediment load due to channel bank erosion. The authors concluded that increased discharges as a result of urbanization cause channels to permanently enlarge to accommodate new flow volumes. Since 20% of the increased sediment load was due to channel bank erosion, the greatest opportunity to limit sediment production is by reducing channel bank eroding stormwater discharges. Although bank hardening may reduce localized erosion, erosion may be increased elsewhere with this management strategy. In order to control the sediment loads to the stream network, efforts should be made to minimize stormwater discharges to the stream system.

Nelson and Booth (2002) evaluated a watershed-scale sediment budget for a rapidly urbanizing watershed in western Washington to determine the relative effects of land use practices on sediment supply and delivery, and to guide management responses toward the most effective source reduction strategies. Human activity caused an increase of 50% in the annual sediment yield. The main sources of sediment in the watershed were landslides (50%), channel-bank erosion (20%), and road surface erosion (15%). Urbanization may ultimately result in decreased local surface erosion rates when large areas are covered with impervious surfaces such as roadways, rooftops, and parking lots (Wolman, 1967). Urbanization can also indirectly increase channel erosion and downstream sedimentation by increasing the frequency and volume of channel-altering flows (Leopold, 1968; Hammer, 1972). For the study, sediment production processes and rates were separated into land use categories to include urban areas, agriculture, forest/timber harvesting, construction areas, landfill, and quarries. Sediment transported from urban areas, construction sites, landfills, quarries, and agricultural areas can only reach the stream system by transport in suspension and are assumed to be fine grained. However, bank erosion and landslides can contribute sediment of both fine and coarse grain sizes. Fine sediment loads were calculated using published TSS yield coefficients for land uses in the Pacific Northwest.

Scholz and Booth (2001) proposed a monitoring strategy with specific existing monitoring protocols based on physical stream characteristics that can be used for the management and rehabilitation of streams in urbanizing watersheds. They suggest the field measurement of six channel features that can be affected by increased streamflows incurred by urbanization. The following features are of particular importance: channel geometry; stream corridor vegetation; channel erosion and bank stability; large woody debris; channel-bed sediment; and instream physical habitat. These characteristics can also be used to group channel types for comparison within the groups. The most useful measure of channel geometry is bankfull channel dimension, which can be determined with cross sectional area. These measurements can be made quickly and are primary variables for relating channel size to watershed parameters. Trend analyses can be made to identify changes in geomorphology over time and show relative deviation of channel morphology from undisturbed conditions, and prioritize streams for rehabilitation. Visual descriptions of bank erosion and bank instability can also be useful in guiding rehabilitation because it is a way to recognize hydrologic disturbance that typically accompanies urban development.

Research by Arnold and Gibbons (1996) into the process by which imperviousness affects water quality indicates that, although the impervious surface does not generate pollution, a clear link has been made between impervious surface and the hydrologic changes that degrade water quality. Impervious surfaces are a characteristic of urbanization and their presence prevents natural pollutant processing in the soil by preventing infiltration. Impervious surfaces allow for the conveyance of pollutants into the waterways usually through the direct piping of stormwater. Therefore, if the direct connection is interrupted through dispersion over a surface area, pollutant transfer to the receiving waterway can be reduced.

Research on Directly Connected Impervious Surfaces and Disconnection as a Management Strategy

Urban imperviousness is a very important parameter in the management of urban watersheds and has profound effects on the downstream water environment. Direct connectivity to the drainage system is an important attribute of urban imperviousness. When impervious surfaces are directly connected to each other, runoff is quickly transported from impervious surfaces to the receiving stream with little or no losses. Any opportunity for mitigation through infiltration is bypassed. Since a large percentage of stormwater runoff is contributed by directly connected impervious area (DCIA), it is a critical parameter for many models. The direct measurement of DCIA, however, is complicated and time consuming, and few accurate analyses have been performed.

Land cover is a primary input in many hydrologic models. Since urbanization has profound impacts on the hydraulic regime of the receiving streams, there is a need for an index variable to characterize the magnitude of urban development. This is difficult in urban areas due to the variation in mitigation already in place and changes in landscape conditions. Watershed imperviousness has been accepted as a key parameter in the modeling of watershed runoff response. Total impervious area (TIA) is a parameter that can be readily determined using remote sensing or other techniques (Hill, Botsford, and Booth 2003).

Hill, Botsford, and Booth (2003) present limitations to the use of the total impervious area (TIA) in watershed modeling. When TIA is used to indicate imperviousness in hydrologic models, it can produce misleading results. TIA can misrepresent the hydrologic effect of impervious surfaces in two ways. First, it ignores nominally pervious areas that contribute runoff. In many urban settings, open green spaces can suffer from compaction or are so low in permeability that they contribute runoff at rates comparable to traditional impervious surfaces. The second misrepresentation of TIA is that it includes disconnected impervious areas that do not contribute to runoff. These disconnected surfaces route flow over pervious areas which slow the runoff velocity. Overall runoff volume is decreased because the stormwater has the opportunity to infiltrate. There are different downstream consequences of rooftops that drain into a piped storm drain system and discharge directly into the receiving stream than rooftops that are disconnected by discharging onto splash-blocks that disperse the water into a garden or lawn area at each corner of the building. A better estimation of imperviousness for watershed models is to use the effective impervious area (EIA). EIA is a summation of the impervious areas that actively contribute runoff throughout the watershed because they have a direct connection to the drainage network. EIA and DCIA are essentially the same. EIA is the preferred parameter for use in models because it captures the hydrologic significance of imperviousness (Hill, Botsford, and Booth 2003).

In their research into the use of remote sensing techniques for the classification of land use, Hill, Botsford, and Booth (2003) emphasize the importance of the accurate classification of land cover for resource and watershed management. This classification is gravely important especially in models where land cover is a primary input parameter. The remote sensing method developed by the authors does not differentiate between TIA and EIA since the designation of EIA is still heavily dependent on field reconnaissance. The authors suggest a need for the evaluation of EIA for accurate hydrologic models.

One of the first examples of the effect of EIA and disconnection on the results of watershed modeling was documented by Beard and Chang (1979). Runoff conditions for a number of areas similar in hydrology to the Tulsa region including areas in Oklahoma City, Dallas Fort Worth, and Austin were simulated using the best available rainfall and runoff data in the US Army Corps HEC-1 model. The study was launched to analyze the effect of total impervious surface cover on streamflow. The study determined that the impervious areas had a much lesser effect on runoff volume than was previously forecasted using the total impervious area as an indicator. The runoff/rainfall ratio was calculated for all storms with a depth less than 2.0 inches and was plotted against the ratio of impervious area to total tributary area. The ratio of impervious surface area to drainage area did not correlate highly with the actual runoff ratios in small storms where the correlation would be the best. About 40% of the impervious area was fully effective in increasing the volume of runoff, therefore, impervious areas had a lesser effect on runoff volume than the ratio of impervious to total area would indicate. Since 40% of the total impervious area was fully effective in contributing to runoff volume, the authors suggest multiplying the ratio of impervious surface to total drainage area by 40% for use as a parameter in HEC-1. It is assumed that there are losses due to infiltration of disconnected stormwater over pervious areas, interception, and detention that account for the runoff generated by the disconnected impervious areas.

Results of urban runoff models vary depending on whether EIA or TIA is used (Alley and Veenhuis, 1983). However as of the time of their research, few studies had separated effective from noneffective impervious areas. A number of studies have outlined methods to determine total impervious area, and engineering manuals contain percentage values for total impervious area based on land use. Alley and Veenhuis (1983) separated impervious surface area into two categories: effective impervious area (EIA) and noneffective impervious area. Effective impervious area describes impervious surfaces that are hydraulically connected to the channel drainage system. Streets with curbs and gutters and paved parking lots that drain directly to the street are examples of EIA. Noneffective impervious surfaces, on the other hand, are impervious surfaces that drain to pervious ground such as a roof that drains to a lawn.

One approach to determining EIA is to relate it to the minimum ratio of runoff/rainfall measured for small storms. This approach has limitations because of the need for rainfall runoff data from the watershed (Alley and Veenhuis, 1983). Soils that have low permeability may also limit the use of this method since pervious areas of low permeability can mimic the runoff response of impervious surfaces. The method may also be limited by the sensitivity to errors in rainfall and flow measurements and estimation of impervious retention (Alley and Veenhuis, 1983).

Research by Alley and Veenhuis (1983) indicates that the ratio of EIA to TIA is independent of lot size. TIA, EIA, and ratio EIA/TIA were estimated for each land use type in the 19 basins that were surveyed. As lot size increased, TIA and EIA decreased, however, the ratio EIA/TIA appeared to be independent of lot size and showed little variation between the basins sampled. Alley and Veenhuis (1983) suggest that the equation $EIA=(0.15)\ TIA^{1.41}$, determined in a previous study for basins within the Denver metropolitan area, is unique to the sample basins and only similar investigations in other urban areas can confirm the utility of the equation.

When TIA is used as the parameter in watershed models, both runoff volume and peak flow are overestimated in ungauged watersheds. Changes in land use intensity are often reflected by a change in impervious surface and calculated as such in prediction models. The sample data suggests that EIA increases faster than TIA as development intensity increases. For every 1% of TIA increase, EIA increases by 1.4%. Therefore, when watershed changes are modeled to simulate increased land use intensity, the corresponding change in runoff may be underestimated if TIA is used instead of EIA. The authors also suggest that the use of TIA instead of EIA in models may also overestimate infiltration rates if the model is calibrated with measured rainfall-runoff data, since a larger volume is predicted, but a smaller volume is observed. Infiltration would be increased to represent the difference when calibrating with TIA.

Different stormwater management strategies, such as the uses of swales instead of curbs and gutters or a different percentage of disconnected rooftops, would have an effect on the EIA/TIA ratio and/or the EIA equation. A large potential exists for developing relationships between EIA and TIA for an urban area, either through a regression equation between the two variables, or through estimates of the ratio EIA/TIA as a function of land use (Alley and Veenhuis, 1983).

Alley and Vennhuis (1983) also discuss how EIA is treated in urban stormwater modeling. Results from many urban runoff models are sensitive to the value used for impervious area, and large differences in results can be obtained depending on whether EIA or TIA is used. While EIA is a better representation of the hydrologic effect of imperviousness on a watershed, most models that require imperviousness as an input do not differentiate between EIA and nonEIA. Common practice is to use TIA or to treat a noneffective impervious area as though it were a pervious area. Models can simulate stormwater disconnection by assuming that rain falling on noneffective impervious areas is instantaneously and uniformly distributed over the pervious area. This volume is expressed as depth over the pervious area and is added to the rain falling on the pervious area. This depth is considered the new rainfall depth, and is used to calculate the runoff from the pervious area. When pervious area runoff is a significant part of the total runoff, the simulated runoff volumes and peak flows are sensitive to estimates of both EIA and infiltration parameters. It is difficult to separate the effects of these parameters in calibration. The authors suggest calibrating the EIA using data from smaller storms and infiltration parameters using larger storms. This is due to the role that EIA plays in the runoff response from small storms in areas where, if left undeveloped, would generate little or no runoff. Although their research does not investigate this topic, EIA may not remain constant for all storms or even for all times during a storm. This may be particularly true for areas with streets without curbs and gutters.

Alley, Dawdy, and Schaake (1980) proposed a parametric-deterministic urban watershed model that considered both effective and noneffective impervious surfaces. The model included a user defined impervious retention, which was a parameter to describe the abstraction from rainfall on effective impervious areas. The retention would be filled before runoff from effective impervious areas would occur. The model assumes that rainfall falling on noneffective impervious areas runs off onto the surrounding pervious area and is added to the rainfall depth over the pervious area. This rainfall is then added to rainfall depth in the rainfall excess computation. Runoff from pervious areas was determined based on Green Ampt infiltration using soil moisture accounting and the hydraulic conductivity at natural saturation.

Lee and Heaney (2003) conducted a study involving both spatial and hydrologic analysis to better understand urban imperviousness and the impacts of directly connected impervious area (DCIA). The first phase of the study was a hydrologic analysis to evaluate long-term impacts from a high-density residential area in Miami. A linear rainfall-runoff estimation model was developed using high-quality hydrologic data, and it was applied to the

52 years of available long-term hourly rainfall data using the Storm Water Management Model (SWMM). Their results show the disproportionate contribution of DCIA to the overall volume of stromwater runoff. The directly connected impervious area, which covered 44% of the catchment, contributed 72% of the total runoff volume over the 52 years of recorded data.

The second phase of the investigation into impervious surface analysis was a spatial analysis of urban imperviousness of a three-block residential neighborhood in Boulder, Colo. The analysis was performed at five levels of effort using both geographic information systems (GIS) and field investigations to show the improvement of accuracy and its impact on the estimated downstream runoff hydrograph for a one-year storm. The field investigations were a key component in determining the extent to which DCIA contributed to overall imperviousness. Remote sensing techniques have been applied to analyze urban imperviousness in many studies, but the spatial resolution and tree canopy of the imagery limit its accuracy. It is difficult or near impossible to distinguish DCIA from the total impervious area (TIA) correctly by using only remote sensing techniques. Most available data about urban imperviousness are based on land use or zoning and use image processing techniques with satellite or airborne imagery. However, this spatial resolution and accuracy may be inappropriate for microscale (lot or subdivision scale) storm water analysis. Therefore, field investigations, although labor intensive, return the best estimation of the DCIA encountered in an urban drainage area.

Hydrologic modeling of the area studied by Lee and Heaney (2003) determined a 265% difference in estimates of peak discharge depending on the level of accuracy of the method used to calculate imperviousness. The determination of how much of the area was

DCIA was an influential factor in the calculation of runoff response. The results suggest the need to focus on DCIA as the key indicator of the effect of urbanization on storm water quantity and quality. In the study area, 97.2% of the DCIA was transportation related. This suggests that in order for disconnection to be a viable strategy for the mitigation of stormwater volumes, not only the rooftops, sidewalks, and driveways need to be disconnected, but the streets and other transportation infrastructure as well (Lee and Heaney, 2003).

Booth and Jackson (1997) define a 10% effective impervious area threshold which relates to degradation of aquatic systems. Mitigation efforts with detention ponds were analyzed, and the authors state that even with the best efforts at mitigation, the magnitude of development activities that fall below the threshold of regulatory concern suggests further watershed degradation will occur with an increase in development. Changes in upland runoff processes from a predominantly subsurface flow regime to a surface flow regime alter the delivery of stormwater discharge, the delivery of sediment to the stream network, and encroach on the stream corridor. The study distinguishes between TIA and EIA to define urban development. Channel stability was analyzed as an indicator of watershed health. It was observed that the previous 10-yr forested discharge was equal to the current developed condition 2-year discharge. This was the threshold of channel stability and occurred at about 10% effective imperviousness. While degradation can occur at very low levels of urbanization, a noteworthy accumulation of biological and physical effects were observed in this study once EIA reached 10%.

After a review of the current literature on impervious thresholds, Brabec, Schulte, and Richards (2002) determined that there are a few flaws in the assumptions and methodologies used to define impervious thresholds for water quality degradation. The literature is characterized into four topics: 1) the determination of a single threshold of watershed imperviousness may not be the only or the most important watershed variable; 2) mitigation efforts such as detention ponds and riparian buffers have limits to their effectiveness; 3) woodland cover and other pervious land uses are critical to the pervious/impervious equation; 4) the location of impervious surfaces in a watershed can have significant impacts on water quality. Topics 3 and 4 are important to stormwater disconnection.

The authors also determined that 1) the methodology for defining the key determinant, which is the percentage of imperviousness per land use, varies between studies, and the percent urbanization is usually equated with percent imperviousness; 2) most studies do not differentiate between TIA and EIA; and 3) the analyses used both biotic and abiotic measures to determine stream impacts, which makes a single threshold of degradation difficult to determine.

The literature argues for the development of a continuum model where the varying percentages and placement of land uses from totally impervious to nearly all pervious can be balanced in a watershed to achieve a desired level of water quality at build-out as noted by Booth and Jackson (1997). Achieving this balance may not be out of the question for existing urban areas if the proper strategies are employed. Disconnection could be a valuable tool in achieving this balance.

Warnemuende et al. (2003) designed a methodology for determining the effects of the extent and geometry of impervious surface on the hydrologic balance under controlled conditions. The feasibility of rainfall simulation methods to evaluate hydrologic and erosional responses to various imperious treatments is examined. Rainfall was simulated over various configuration of impervious surface with an intensity oscillating nozzle rainfall simulator. The hydrologic, erosional, and water quality impacts of specific urban land use configurations and directly connected impervious areas was quantified with a modular segmented soil box design. A 4m x 4m soil box of soil depth 5-8 cm and a two dimensional slope was configured with impervious tiles to represent impervious rooftop ranging from 0-35% total impervious surface. Simulations were run with impervious surface at the periphery of the box and again with impervious surface adjacent to the channel. Both versions of impervious surface configurations contributed more sediment and had a shorter time of concentration than the non-impervious trial.

A transportation surface was also simulated by Warnemuende et al. (2003) with 20 x 20 x 0.6 cm unglazed residential clay tile laid flush with the soil surface. The spaces between tiles were caulked to maintain a flat surface. Two different configurations were set up on a lengthwise 5% slope. One setup placed the impervious surface at the top of the slope to represent development at the headwaters, and the other with impervious surface at the bottom, to simulate development adjacent to the stream channel. Impervious surfaces yielded differences in both the runoff hydrograph and sediment losses through the trials. The trial with the upper area imperviousness contributed less runoff, as water was absorbed by the lower pervious areas. After the initial wetting period, the upper impervious surface trial yielded generally higher runoff and sediment loads. The transition points between pervious and impervious surfaces were prone to scouring and undercutting at higher rainfall intensities, therefore a modular box design was proposed for subsequent research.

The proposed design included soil boxes 1 x 0.6 m with a depth of 20 cm designed to flow on a slope from box to box through a short baffled flume system. Specific land treatments such as vegetation or porous pavement could be installed in the boxes to minimize inconsistencies in preparation of the surface for trials (Warnemuende et al, 2003).

In conjunction with the Warnemuende (2003) project, Bonta et al. (2003), with USDA, proposed a field study conducted at the North Appalachian Experimental Watershed near Coshocton, OH, to examine runoff and sediment contributions from 0-40% impervious surface under disconnected and connected arrangements. Results have not yet been published, however, the investigation of impervious surface effects at the small watershed scale may be very useful in the future. Four small experimental hilltop watersheds were chosen for study. Impervious surfaces were installed to approximate a residential neighborhood without streets and with no house gutters. Percent imperviousness will be increased through the years of the project beginning with 5% impervious surface cover and eventually 40% impervious surface cover. Two configurations will be simulated, impervious surface at the periphery to mimic disconnection and impervious surface adjacent to the stream channel to mimic connected impervious surface. Small impervious roofs will be constructed 1 m above the ground. Land will be disturbed and the foundations and turf installed and managed with fertilizer and pesticides to mimic homeowner activities. Infiltrometer, soil water potential, and water content measurements will be taken before and after construction. Runoff and water quality parameters will also be closely monitored. When the watersheds reach a maximum imperviousness, best management practices (BMPs) will be installed and evaluated to study their effect on the system.

TR-55 and the SCS Curve Number

The U.S. Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS) developed a method for estimating rainfall runoff volume based on measured total rainfall and direct runoff, and physical features as documented in the National Engineering Handbook, Chapter 4 [Soil Conservation Service (SCS) 1972)]. The SCS method is an empirical procedure that was developed to provide a consistent basis for estimating runoff under varying land use and soil types. The two most commonly used SCS models are TR-20 and TR-55 (NRCS 1986). TR-20 is a computerized single event model that uses a surface water runoff hydrograph and includes streamflow and reservoir routing procedures. TR-55 is distillation of the results of a large number of TR-20 runs expressed as a series of tables and graphs. The tables and graphs are used as a reference for runoff computations. TR-55 includes simple procedures that can be used to calculate storm runoff volume, peak rate of discharge, hydrographs, and storage volumes required for flood control reservoirs.

TR-55 was developed with a method for application in small urbanizing watersheds. One of the greatest effects of urbanization on a watershed is the change in its response to precipitation. In the urban watershed, infiltration is reduced and travel time is increased, which greatly increases peak discharges and runoff. TR-55 converts mass rainfall amounts to mass runoff by the application of the curve number through the SCS equations. While there are several versions of the SCS models, and all are described in NEH-4 (Soil Conservation Service 1972), all versions rely on the curve number to estimate runoff volume with the basic expression:

$$Q = \frac{(P - Ia)^2}{(P - Ia + S)} \quad \text{for P>0.2S}$$

Q is the runoff volume, P is the precipitation, I_a is the initial abstraction, and S is the maximum soil storage. All variables are expressed in inches. The storage parameter, S, is related to the curve number, CN, through the following transformation:

$$S = (1000/CN) - 10$$

Storage is limited by either the rate of infiltration at the soil surface or the amount of water storage available in the soil profile. The initial abstraction is a term that lumps the volume lost to interception, depression storage, and infiltration before runoff begins. After runoff begins, additional losses are usually due to infiltration. The SCS curve numbers are based on the assumption that the initial abstraction, I_a, is equal to 0.2S. The empirical relationship was determined from analysis of small experimental watersheds (Rallison and Miller 1982). When this relationship is substituted into the SCS equation, it can be reduced to:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \quad \text{for } P>0.2S$$

The curve number is a parameter that describes the response of the land surface to rainfall and the flood potential for the area. It is a representation of the watershed soil and cover conditions which include hydrologic soil group, cover type, treatment, and hydrologic condition (SCS 1972). The precipitation excess, or runoff, will be zero until the accumulated rainfall exceeds the initial abstraction, Curve numbers are dimensionless and can vary from 0 (no runoff) to 100 (all rainfall becomes runoff). When accumulated runoff is plotted versus rainfall depth the curve is asymptotic to a 45 degree line. Depending on the rainfall depth, there is a curve number threshold below which there is no runoff when the SCS equations are used (Hawkins 1975).

The SCS hydrologic models are among the most widely used models in water resources planning and design. They were originally developed for agricultural areas, however, they have been extended for use in urban areas (NRCS 1986; Rallison and Miller 1982). The curve number is the primary input parameter, and since it can be defined in terms of land use and soil type, SCS methods have great utility because they can be applied to ungauged watersheds.

Curve numbers are determined in TR-55 based on hydrologic soil group (HSG), land cover, hydrologic condition, and antecedent runoff condition (ARC). The curve numbers listed in TR-55 represent the average ARC (Rallison and Miller 1982; NRCS 1986). TR-55 also recommends that consideration is given to whether impervious areas are connected (outlet directly to the drainage system) or disconnected (flow is spread out over a pervious area before entering the drainage system) in curve number selection and includes graphical figures based on the percent directly connected impervious areas to select the appropriate curve number. Figure 2-3 and Figure 2-4 from TR-55 were intended for use in determining the curve number based on the connected impervious area (See Appendix). TR-55 provides the user with tables for curve number selection. Table 2-2a (See Appendix) is provided for urban areas. Curve numbers are listed for each land use based on soil hydrologic group (A, B, C, or D). Open space is designated as lawns, parks, golf courses, cemeteries, and the like, and is also grouped into poor, fair, and good hydrologic condition. All impervious areas including rooftops, paved parking lots, driveways, sidewalks and curbed streets are designated with a curve number of 98. Urban districts are divided into commercial/ business and industrial areas based on percent impervious surface. Residential districts are designated by average lot size and also list an average percent impervious area. The average percent impervious area was used to assign business/commercial, industrial, and residential curve numbers for each hydrologic soil group. Urban curve numbers were derived by taking the average weighted value of the impervious and pervious curve number for each hydrologic soil group based in the given percent impervious surface of the land use category.

HEC-HMS

Many modeling packages include the SCS equations for use in runoff computations. HEC-HMS is the US Army Corps of Engineers' Hydrologic Modeling System (HMS) program and was developed by the Hydrologic Engineering Center (HEC) (USACE 2001). HEC-HMS improves upon the capabilities of the original HEC-1 program. HEC-HMS provides components for precipitation-runoff routing simulation. The model consists of a rainfall-runoff model that converts precipitation excess to overland flow and channel runoff. The model returns hydrographs that can be used in analysis. Precipitation specification options can be used to describe an observed precipitation event, a frequency based hypothetical precipitation event, or an event that represents the upper limit of precipitation possible at a given location. HEC-HMS also includes loss models that can estimate the volume of runoff when precipitation and hydrologic properties of the watershed are given. Direct runoff models that account for overland flow, and the storage and energy losses as water runs off a watershed into the stream channels are also included in the package. Hydrologic routing models are used that account for storage and energy flux as water moves through the channels. HEC-HMS includes an automatic calibration package that can estimate certain model parameters and initial conditions given observed data.

The SCS model is included in the HEC-HMS interface as an event, distributed, empirical, fitted parameter model to simulate runoff volume. Rain gauge data, as well as streamflow, can be entered manually into HEC-HMS for calibration of the model to rainfall-runoff response. One of the drawbacks of using the SCS method to model loss in the watershed system is that it does not consider rainfall intensity in the calculation, only total volume of rainfall. However, it is a preferred method because it is simple and relies on only one parameter, the curve number, which varies as a function of soil group, land use, and antecedent moisture condition (Rallison and Miller 1982). HEC-HMS includes methods for incorporating baseflow into the model. HEC-HMS returns a hydrograph for each storm simulation with detailed information on total direct runoff, loss, and precipitation (USACE 2001).

Research on application of the SCS equation

Accurate curve numbers are important to the estimation of storm runoff because the SCS hydrologic methods are very sensitive to the curve number (Hawkins, 1975; Rawls, Shalaby, and McCuen, 1981; and Bondelid, McCuen, and Jackson 1982). Without the aid of rainfall-runoff data for small watersheds, an accurate estimate of the curve number is the weak input of the SCS method. Hawkins (1975) performed an error analysis using a 10% curve number error estimation over a wide range of precipitation events (0-15 in) and all possible curve numbers. Hawkins (1975) concluded that 1) for a considerable range of rainfall depth, accurate values of curve numbers are more important than accurate estimates of rainfall depth, and 2) errors in estimating runoff volume are especially dangerous near the threshold of runoff. Additional engineering research should give attention to representative watershed studies that focus on the curve number.

Rawls, Shalaby, and McCuen (1981) evaluated several methods of determining urban runoff curve number using data from 175 urban watersheds. Their results indicated that estimates of curve number are more sensitive to the land use classification system than the method of integrating soils and land cover data. The study compared urban runoff curve numbers developed by integrating land use and soil separately using a weighted average, or contingency table, scheme to using USGS land use maps. Both methods were screened against conventional SCS approaches. The conventional method of determining curve numbers involves determining a composite curve number through calculating a weighted average of curve numbers for unique land cover-soil type complexes based on low altitude aerial photographs, land use maps, soil maps, and onsite investigations. Land use and soils data is analyzed separately in the contingency table approach. A weighted mean curve number was determined using the product of percent land use, percent soil group, and associated curve number. USGS land use mapping from aerial photos using the Anderson classification system was also used as a comparative method. The curve number selection was not based on actual volumes that were contributed by the land surface, and issues with the conventional composite curve number method were not presented. The authors suggest that the USGS land use approach would reduce imprecision if the USGS residential land use designations were separated into categories that divided lot size similar to that in the SCS tables or vice versa. Wider curve number variations were noticed for basins less than 26 mi^2. The study did not consider storm depth as a determinant in curve number selection.

Bondelid, McCuen, and Jackson (1982) performed a study to evaluate the sensitivity of SCS methods to curve number variation. As the design rainfall increases, the effects of curve number variation decreases. Therefore, the SCS models are most accurate when applied to large storm events. The changes in runoff volume and peak discharge computations were quantified as a function of changes in curve number estimates. Variations in curve number result in misclassifications of land cover, treatment, hydrologic condition, and/or soil type. The magnitude of the curve number deviation, therefore, depends on both the size of the area misclassified and the type of misclassification. Proportional change in runoff volume decreased with an increase in both rainfall depth and curve number. As precipitation depth and curve number increase, the proportion of rainfall that goes into the initial abstraction and infiltration decreases, so the proportional error decreases. Differences in curve number estimates can occur because of the use of different methods to determine the curve number, or by random variation, both by human judgment and data base errors. Curve numbers are at best approximations of the true runoff potential, and sensitivity analysis can serve as a bridge between the degree of inaccuracy in the curve number tables and the hydrologic effects of the inaccuracy. The authors compared variations in curve number approximations to three watersheds. Conventional curve numbers were used as the standard, but the derivations were not compared to real rainfall-runoff data to test the accuracy of the SCS method. This study mainly discussed the effect of human judgment in estimated curve numbers as reflected in sensitivity curves.

Hjlmfelt (1991) investigated the SCS curve number procedure to establish a logically consistent, experimentally verifiable system. The author attempts to address some of the applications of the method that have evolved since its creation. A difficulty in interpretation is that much of the development process leading to the synthesis of the procedure is unavailable. Mockus (1949 as cited by Hjelmfelt, 1991) began work that would soon lead to the development of the curve number with the goal of developing a procedure for estimating runoff from small ungauged watersheds. Hjelmfelt (1991) also points out questionable interpretations of the curve number. It is noted that infiltration cannot necessarily be derived from the SCS equations, although some researchers attempt to make this correlation. There are also problems when using the SCS method to calculate runoff for small storm events.

When calibrating small events, the curve number increases. The variability of the curve number leads to difficulty in modeling accurate runoff during small events.

Hawkins (1993) presented a method for determining the curve number using an asymptotic method. The runoff curve numbers were determined using direct runoff and event rainfall data sets. Three different patterns of runoff were observed and described: complacent, standard, and violent. The standard and violent cases lead to a constant curve number with increasing rainfall, but the complacent case does not lead to a constant determination of curve number. The complacent pattern of runoff describes a situation in that the observed curve number over the course of the rainfall event declines steadily with increasing rainfall and does not approach a stable value. The most common scenario is standard behavior. The observed curve number declines with increasing storm size and approaches a near constant value. This behavior may describe overland flow and rapid subsurface flow. During violent behavior, curve numbers rise rapidly and asymptotically approach an apparent constant value. At lower rainfalls, there may be an accompanying complacent behavior. The complacent definition suggests that the watershed does not respond in accord with the SCS equations within the range of the observed data.

Bonta (1997) later explored a method for determining watershed curve numbers using derived frequency distributions. The original method of curve number determination used the maximum annual events. Subsequent development of the method incorporated measured rainfall and runoff data as frequency distributions. The proposed method treats precipitation and runoff data as separate frequency distributions and gives fewer variations in curve number estimates for a wide range of watershed sample sizes (Bonta 1997). The model has potential for determining curve numbers when limited rainfall and runoff data are available. Large events are selected in the Bonta (1997) method, therefore, a method for small event modeling is still needed.

Tsihrintzis and Hamid (1997) studied urban stormwater quantity and quality modeling using the SCS method and empirical equations for small watersheds of unique land use (low density residential, high density residential, commercial, and highway) in southern Florida. Quantity and quality data from 95 storm events were used for calibration and derivation of input parameters during short, frequent storm modeling. In this study, the k parameter that is a proportional constant in the initial abstraction equation (I_a = kS) was allowed to vary from 0-0.2 as one of the calibration parameters. All storms in the calibration and verification were of depth less than a 2-year return period. The curve number and k value were the two variables used to calibrate runoff by best matching shape, peak, and volume of predicted hydrographs with measured hydrographs. Best matching was determined by the minimum sum of the square error between the predicted and observed hydrographs from a given storm event.

The curve numbers varied within a specific range depending upon the total rainfall depth. For all land uses, the curve number decreased with increasing rainfall depth and was presented as a linear regression of curve number versus precipitation depth. Impervious portions of the watershed are more responsive to small storms than pervious areas, so the runoff response in calibration is represented by a higher curve number. Parameter k did not show dependence on rainfall depth, so k was averaged for all storms within each land use category. The trend was for k to increase with increasing impervious surface. The parameter is the portion of ultimate storage that is due to initial abstraction. In highly impervious areas there is less loss to infiltration and most loss is due to initial abstraction. The commercial area had the most hydraulically effective impervious area since it was drained by storm sewers, and peak flows in this area reached much higher values than for other land uses. It should also be noted that in the residential areas, the streets had no curbs and were drained by swales, which reduces the effective impervious area.

While the SCS method was originally developed for predicting response to large storms, if properly calibrated with small storms, it can be used as a screening model in small urban watersheds for frequent storm simulations and areas of uniform land cover. Predicted peaks were noticed to fluctuate much more rapidly than actual peaks. The entire area was lumped as one, ignoring the effects of timing, routing, and storage in individual sub-areas and ignoring the response of pervious and impervious portions. The study was limited by a lack of baseflow and groundwater level data.

Grove, Harbor, and Engel (1998) investigated different methods of selecting curve numbers in runoff calculations. The composite curve number was compared to distributed curve numbers and their effect on the estimation of storm runoff depths. The authors point out that the composite curve number method was originally developed as a time saving procedure to reduce the number of necessary calculations. With modern computing technology, it is now possible to use distributed curve numbers. Comparison of the two simulation methods show that when distributed curve numbers are used instead of the composite curve number, the runoff depth can be up to 100% higher. This difference in estimation of runoff is due in part to the curvilinear relationship between curve number and runoff depth. The difference in runoff is the most severe for wide curve number ranges, low curve number values, and small storms. As storm depth increases the difference between the two methods is minimal. Composite curve numbers were determined by overlaying land use and soils data to delineate polygons with unique land use and hydrologic soil group combinations. A curve number value is then assigned to each polygon and the area weighted average is calculated to determine the composite for each watershed area. In the distributed curve number method, a separate curve number was determined for each cell or polygon and runoff from each cell was calculated and summed to estimate an average runoff depth for the basin.

The two methods were compared with simulated idealized watersheds with random curve numbers and compared to the percent change in runoff per unit area. The methods were compared in both a controlled rectangular grid of a 100 x 100 cell matrix and a complex real world watershed scenario to assess prediction variations for realistic conditions. The percent increase in runoff generated increased as the difference between the minimum and maximum curve number increased in simulation. The magnitude of the difference decreased as the storm depth increased. Two sets of runoff simulations were performed, one for small storms (1.5, 2.0, and 2.5 in.) and the 2, 10, and 50 –year storm event (3.0, 4.3, and 5.5 in.). The percent increase in the estimated runoff volume from the composite to distributed methods reached a maximum between 15% and 40% watershed imperviousness.

Fennessy, Miller, and Hamlett (2001) explored the accuracy and precision of the SCS models for small watersheds. The paper reviews some of the data that was originally used to develop TR-55 and TR-20 and tests how well the models estimate annual series peak runoff rates using longer historical data record lengths. The study also provides a comparison of the SCS models with actual watershed runoff data. The curve number was initially developed as a rainfall to runoff transformation term for traditional agricultural lands for a 24-hour maximum runoff rate series. TR-55 is now predominantly used to model urban, pasture, rangeland, meadow, and woodland areas, while the emphasis on the traditional agricultural curve number values appears to have decreased. The best curve number for the observed data is selected such that the standard error is minimized over the analyzed return periods. The authors make the conclusion that the study supports the need for educated hydrologists to alter model parameters, especially the curve number, to better reflect actual watershed conditions.

Kottegoda, Natale, and Raiteri (2000) developed a method of statistical modeling of daily streamflows that used rainfall input and streamflow data for calibration. Losses were obtained from an equivalent curve number that was related to the total rainfall of the event. The curve number was treated as a random variable in the model, but it was adjusted to decrease with increasing cumulative rainfall. The authors pointed out that the initial problem in the investigation of the curve number technique is the identification of independent and isolated rainfall events. The aim was to model storms with at least a five day low flow interval between events, however the relationship between curve number and antecedent rainfall was not well defined. There was a better regression between curve number and total rainfall. There was a negative trend between curve number and increasing rainfall.

Molgen and Beighly (2002) used a method incorporating GIS to develop a time series of land use in an urbanizing watershed. The TR-55 graphical method was then used to predict peak discharge in a spatially explicit scheme through the watershed, as opposed to just at the single observed outlet. Coupling GIS with TR-55 allowed for both the modeling of the temporal and spatial evolution of peak discharge throughout the watershed. The work of Molgen and Beighly (2002) shows that the common practice of transposition of gauge information to locations internal to the watershed neglects internal variability in peak discharge behavior and could potentially lead to the determination of inappropriate design discharges. One of the strengths of using a GIS based approach to hydrologic modeling is that it is able to handle spatially distributed data, perform book-keeping tasks that maintain spatial relationships, and it supports programming language that can be used to customize applications. TR-55 lumped hydrologic modeling was able to be applied to each pixel within the stream network. The study compared the observed flood frequency record for the USGS gauge at the watershed outlet with the modeled 2- year peak discharge at the same location, which were in agreement. Modeling results indicated a doubling in peak discharge after urbanization over the period from 1951 to 1997. However, when internal peak discharges were examined, it was determined that there were some reaches within the watershed where the peak discharge tripled, while other reaches experienced more moderate increases. The impact of local variability was crucial in the study, and this information is important in effective design of control structures and management practices.

MATERIALS AND METHODS

Disconnection was examined on three scales for the Pompeston Creek Watershed: lot scale, subdivision scale, and watershed scale. Each scale of analysis required different levels of effort. Hydrologic soil group and land use data were essential for each stage of analysis. The SCS equations were used in TR-55 to calculate the curve number and the total runoff volume in each stage of analysis. Data were collected to outline the initial boundaries of the drainage areas and hydrologic conditions from which a hydrologic model of the watershed was developed.

The Pompeston Creek Watershed, located in Burlington County, New Jersey is approximately 8.6 square miles in size and drains ultimately discharges to the Delaware River. It lies within Watershed Management Area (WMA) 18 (Figure 1). Areas of the Townships of Moorestown, Cinnaminson, and Delran and the Borough of Riverton are included within the watershed boundary. It is a highly urban area with 69.4% total urban land use. The watershed is predominantly residential at 59.7% of the watershed land area, while the remaining 9.7% that contributes to the urban areas is a mix of commercial, industrial, and other urban land uses. The Pompeston Creek is classified as an FW2-NT surface water body which is a non-trout general surface water classification applied to those waters that are not designated FW1 or pinelands waters by the State of New Jersey. An active biomonitoring station exists along the main stem of the Pompeston Creek and it is listed as severely impaired for aquatic life on Sublist 5 of the "New Jersey 2002 integrated Water Quality Monitoring and Assessment Report 305(b) and 303(d)" (NJDEP 2004a). Water quality monitoring by the Pompeston Creek Watershed Association (PCWA) of enterococci and fecal coliform also indicates significant bacterial contamination in the stream. Nutrients and total suspended solids have also been monitored in the Pompeston Creek. The Pompeston Creek Watershed is characteristic of many urbanizing areas throughout the state of New Jersey and, therefore, is an excellent candidate for analysis.

The water quality storm was examined on the lot and subdivision scale following TR-55 methodology for four sample residential areas in the Pompeston Creek watershed. Four subdivisions were selected as the study areas (See Figure 2). The study areas included subdivisions of unique soil type and land use classification. Two areas were designated by

NJDEP as Residential, Rural, Single Unit; one was designated as Single Unit Low Density; and the last was designated as Single Unit, Medium Density.

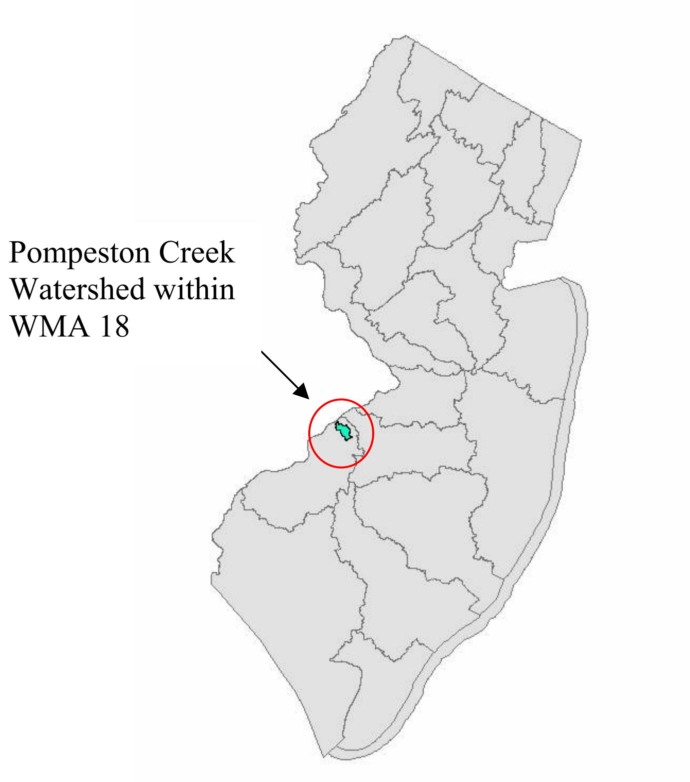

Figure 1: The Pompeston Creek Watershed

These four study areas were selected for the basis of the lot and subdivision scale analysis because they all fell within the boundary of the Pompeston Creek Watershed and represented a cross section of the range of residential lot sizes throughout the watershed. The particular study areas were also selected because they contained a unique soil-cover complex, and thus would be assigned a unique curve number by TR-55. The land use designation and hydrologic soil group for each area was obtained by importing the 1995/1997 Land Use/Land Cover from NJDEP and SSURGO soils data for Burlington County into ArcGIS 9.0. These layers were intersected to return polygons that designate a unique soil-cover complex and were placed over 2002 NJDEP Orthoimagery. A curve number was assigned to each land use-soil complex polygon based on the urban curve numbers given for each lot size and hydrologic soil group in Table 2-2a of TR-55.

TR-55 assumes pervious areas are in good hydrologic condition when determining urban composite curve numbers. Since the Anderson classification system used in the NJDEP Landuse/Land Cover does not designate a land use greater than 1 acre, all residential areas with lots greater than 1 acre are assigned the curve number for residential, rural, single unit land use and the appropriate hydrologic soil group, unless user discretion is employed. Therefore, although Cardinal Drive and Tom Brown Rd. lot sizes differ greatly, integration of the land use and soil data would return the same curve number, which in this case would be 79. However, TR-55 suggests that 2-acre lots and greater with C soils should be assigned a curve number of 77. This composite curve number was used in the subsequent analyses. Since the automated integration of land use and soil data is the norm in GIS curve number processing, it is recommended that the land use and lot size designations in land use classifications should be similar to that in the curve number tables. This situation also emphasizes that there is still a need for consistent classification of land use across applications as discussed by Rawls, Shalaby, ad McCuen (1981) and user discretion should be encouraged when assigning curve numbers to residential areas with lot sizes larger than 1 acre.

The average lot size of each subdivision was determined by importing a layer that included lot outlines for the Township of Moorestown obtained from the Burlington County Office of Land Use into ArcGIS. Lot areas were calculated using a GIS function and averaged for each study area. Impervious surface was determined for each area by summing up the total area of roof, driveway, and street area in AutoCad from files obtained from the Burlington County Office of Land Use. Building footprints, streets, driveways, and lot boundaries were verified against 2002 NJDEP Orthoimagery and edited where necessary. The subdivision locations, land use designations, average lot size, subdivision percent impervious surface, soil type, and curve number are included in Table 1.

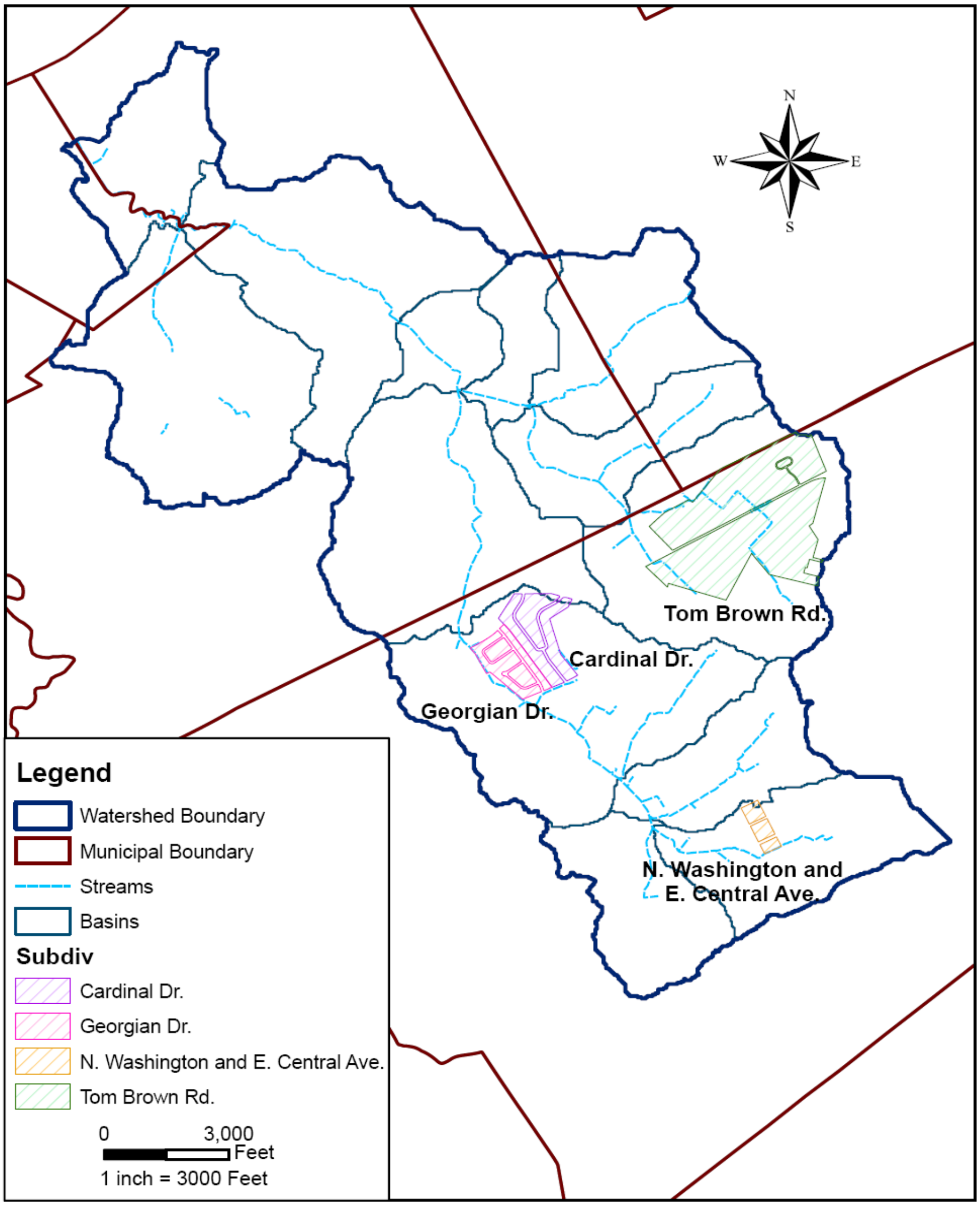

Figure 2: Residential Study Areas in the Pompeston Creek Watershed

Table 1: Residential Study Areas

Location	1995 NJDEP Land Use	Average lot (acres)	Subdivision % impervious surface	Soil Group	TR-55 CN
Georgian Dr	Residential, single unit, low density	0.58	25	B	70
Cardinal Dr	Residential, rural, single unit	1.11	23	C	79
Tom Brown Rd	Residential rural, single unit	5.84	6	C	79 (77)
N Washington Ave & E Central Ave	Residential, single unit, medium density	0.37	28	C	81

<u>Lot Scale Disconnection</u>

The first analysis performed on the lot scale was an analysis of the composite method of calculating residential curve numbers. The composite method was used by SCS to determine the urban curve numbers listed in Table 2-2 of TR-55 (SCS 1986)(See Appendix). The composite curve number was determined for the average lot in each study area by calculating an area weighted average of the impervious and pervious surfaces. The average impervious surface area of the lot was multiplied by the impervious curve number (98). The average pervious area was multiplied by the appropriate SCS curve number based on hydrologic soil group in good condition (B soil = 61, C soil = 74). These values were added and divided by the total lot area to determine the average weighted curve number. Runoff was simulated from each lot using the composite curve number in the SCS equations for the water quality, 2-, 10-, and 100-year storm (1.25", 3.4", 5.2", and 8.8"). Initial abstraction was calculated as 0.2S for consistency of analysis (See Figure 3).

The accuracy of the composite curve number in predicting runoff from urban areas was examined by comparing it to a weighted volume of runoff calculated for each lot in the study area. Runoff was calculated with the weighed volume method by adding the runoff from the impervious surface alone to the volume of runoff from the pervious surface alone (Figure 4). This method simulates the runoff volume from a directly connected impervious surface. By calculating the runoff from each impervious and pervious surface separately the significance of the impervious surface to runoff volume can be quantified. Using a spreadsheet model, a new curve number was determined for each lot by adjusting the curve number until it predicted the runoff volume that matched the weighted runoff volume (See Figure 5).

Next, the disconnected volume was calculated for each lot for various scenarios of disconnection. Two disconnection scenarios were examined: 1) disconnection of all rooftops and 2) disconnection of rooftops and driveways. The simulation was accomplished by first calculating the volume of runoff from the disconnected impervious surfaces. This volume was then added to the rainfall depth falling on the pervious area. The runoff from the pervious area was calculated using the SCS equations and the TR-55 curve number for lawn area in good condition of the designated soil group. The runoff calculated from the pervious area was then added to the runoff from any remaining connected impervious areas. This final volume was assumed to be the overall runoff from the disconnection scenario. The disconnection pervious routing procedure is similar to that described by Alley, Dawdy, and Schaake (1980); Alley and Veenhuis (1983); and NJDEP (2004). Both the overall change in volume and percent change in volume from the weighted volume were calculated for each degree of disconnection.

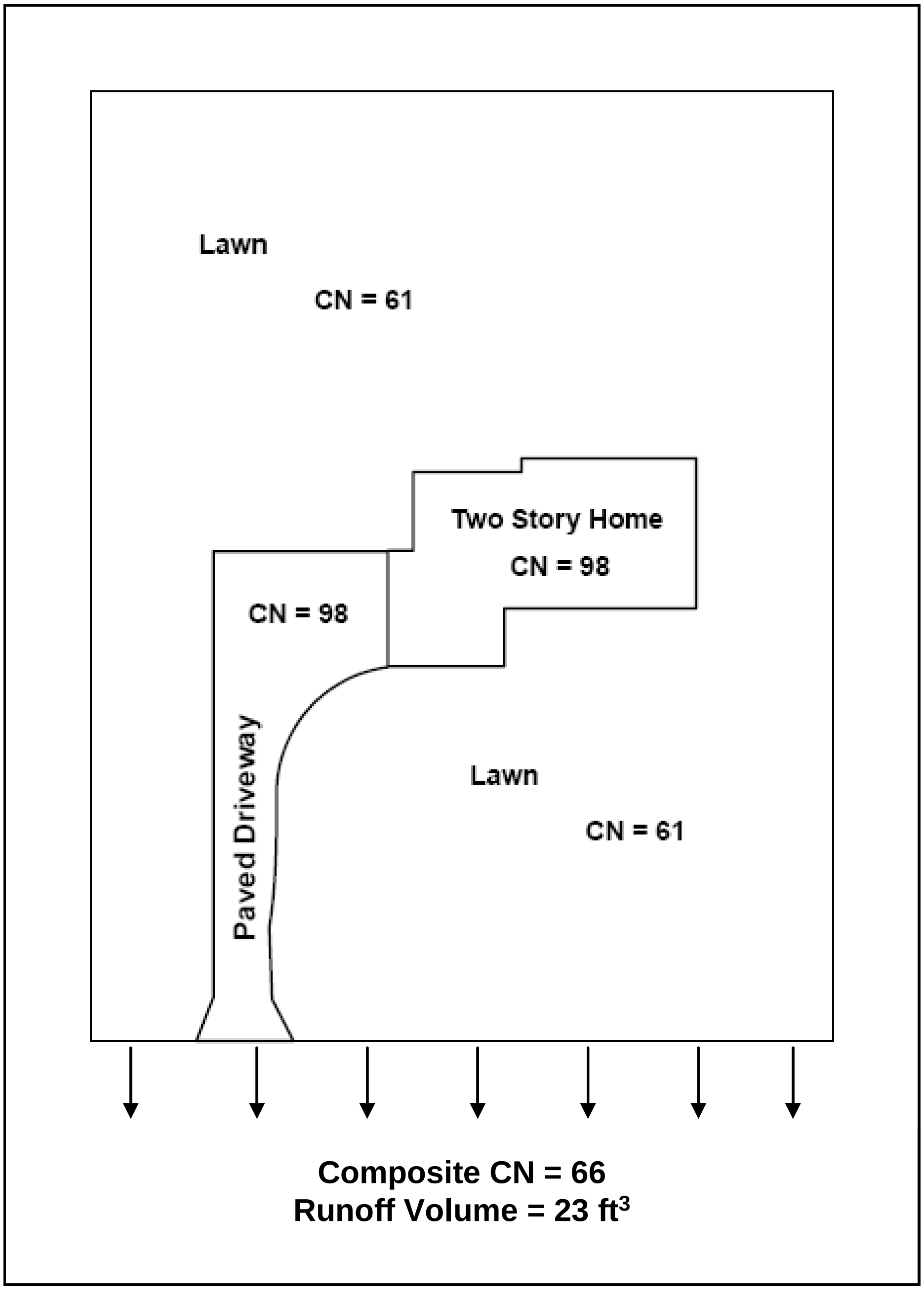

Figure 3: Runoff Diagram for Composite Curve Number Volume (Water Quality Storm) for the Georgian Dr. Average Lot

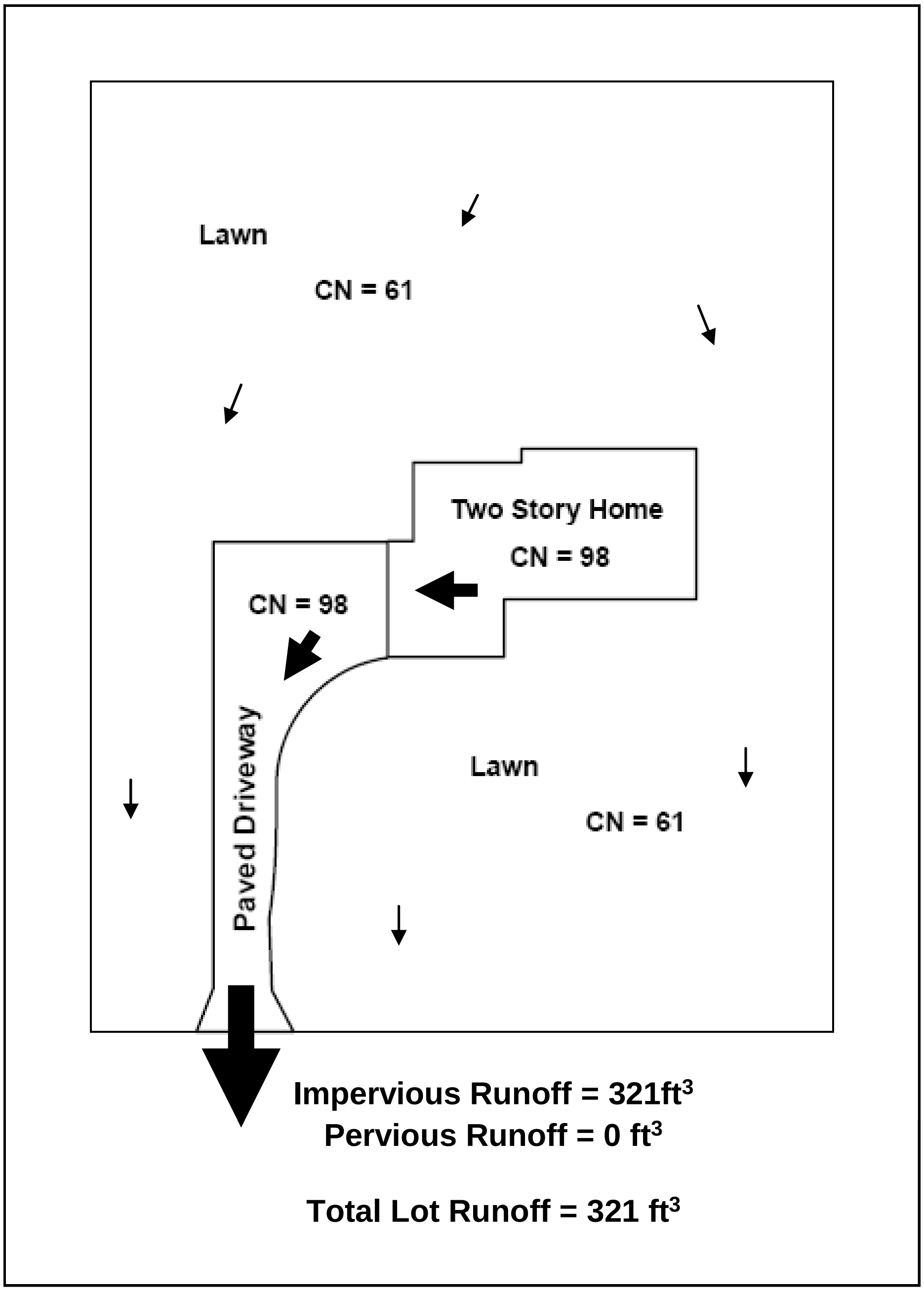

Figure 4: Runoff Diagram for Directly Connected Volume (Water Quality Storm) for the Georgian Dr. Average Lot

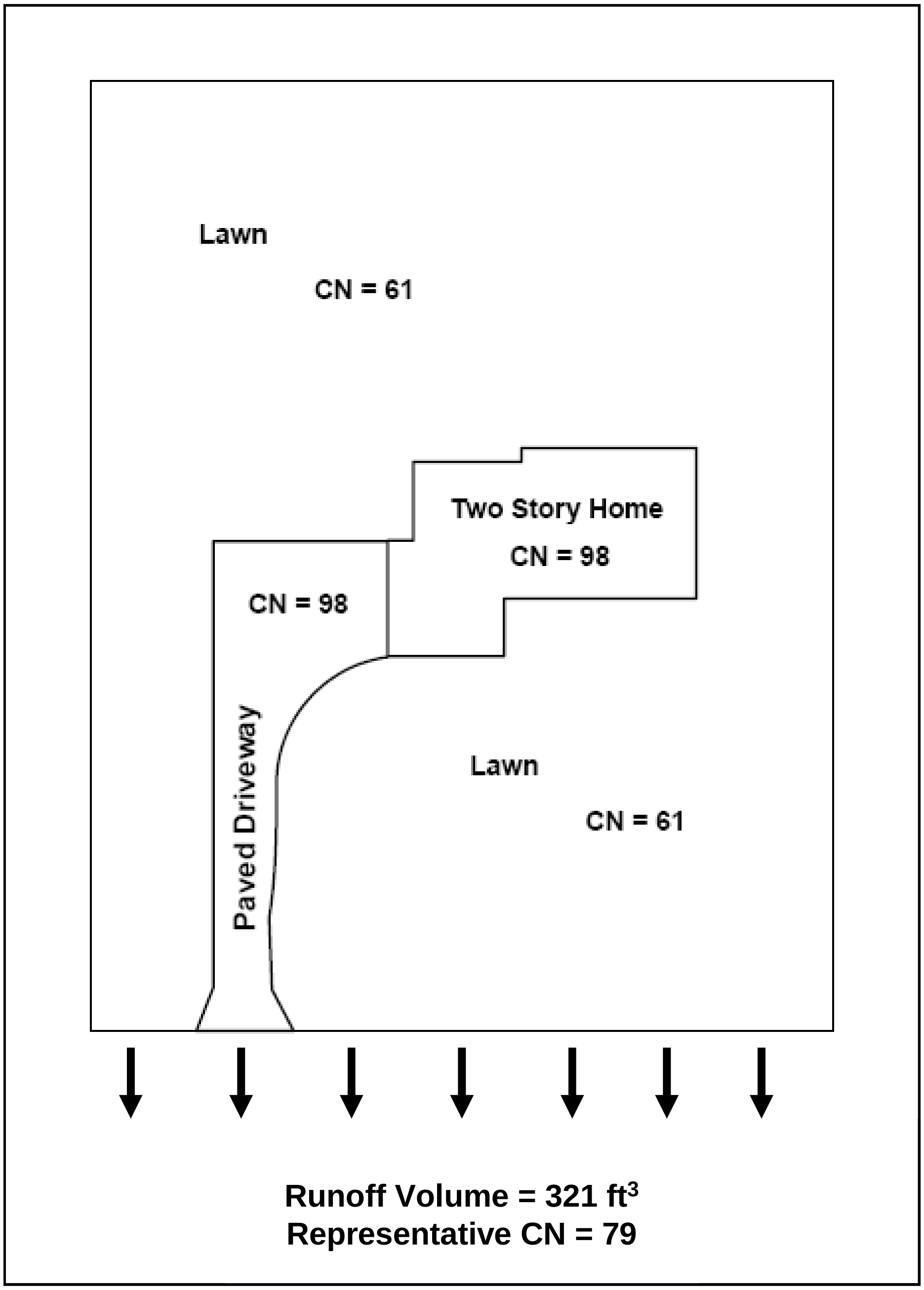

Figure 5: Runoff Diagram for the Curve Number Representing Direct Connection (Water Quality Storm) for the Georgian Dr. Average Lot

<u>Subdivision Scale Disconnection</u>

The subdivision scale analysis was performed to quantify the cumulative effects of individual lot and street disconnection throughout a hydraulically connected neighborhood.

The subdivision lots, stream channels, catch basins and stormwater outfalls are indicated on aerial photos in Figure 6, Figure 7, Figure 8, and Figure 9. Field reconnaissance determined that many of the residential areas in the Pompeston Creek Watershed drain to stormwater outfalls through a storm sewer network. An outfall characteristic of those found to drain the residential areas along the West Branch of the Pompeston Creek is shown in Figure 10. Many outfalls documented along the Pompeston Creek drain directly to the stream with no volume or water quality controls. Bank erosion, scour, and channel widening are seen near and downstream of the outfall locations.

The subdivision scale analysis differs from the lot scale analysis because it considers the effects of the streets throughout the subdivision. The total area for each category of impervious surface (rooftops, driveways, and streets) and pervious surface was calculated in AutoCad. In subdivisions that included a street on the outer boundary, street area was only calculated to its midline. A composite curve number was calculated for each subdivision using the total impervious surface area from rooftops, driveways, and streets and the remaining pervious area. These composite curve numbers were comparable to those given in Table 2-2a for similar land use-coil complexes. Next, direct connection was simulated with the volume weighted method for the water quality, 2-, 10-, and 100-year storms. The total volume weighted runoff was then broken down into the volume contribution from each class of impervious surface.

Incremental stages of disconnection were modeled for each subdivision using the disconnection method of routing impervious runoff over pervious areas as previously described for lot scale disconnection. Both a theoretical and a field based analysis were performed. The theoretical disconnection analysis assumed that 100% of each class of impervious surface could be disconnected (i.e. all rooftops, all rooftops and driveways, and all rooftops, driveways, and streets). The field based scenario established a baseline of disconnection already employed by many homeowners throughout the study areas. Then three reasonable stages of incremental levels of disconnection were established. The stages of disconnection are summarized in Table 2.

The stages of disconnection were selected to reflect different degrees of stormwater management practices. The existing conditions were determined through field reconnaissance. The subdivision neighborhoods were visually inspected, and the locations of rooftop downspouts for each lot and where they drained were indicated on a map. If two roof downspouts discharged to the driveway and two to the backyard, it was assumed that 50% of the rooftop was directly connected and the remaining 50% was disconnected (Alley and Veenhuis 1983; Lee and Heaney 2003). Field reconnaissance determined that approximately 50% of the rooftops in the four study areas examined in the subdivision scale analysis were disconnected from the drainage network. The remaining 50% drained via downspouts to pervious areas such as backyards or gardens. The pitch of the driveway was also indicated. Almost all driveways sloped toward the street and drained to streets with curbs and were assumed to directly discharge to the street. The exception was in the rural residential area of 5-acre lots along Tom Brown Rd. All streets were drained by storm sewers that directly discharged to the receiving stream, except Tom Brown Rd., which was drained by swales that discharge to a tributary of the East Branch of the Pompeston Creek.

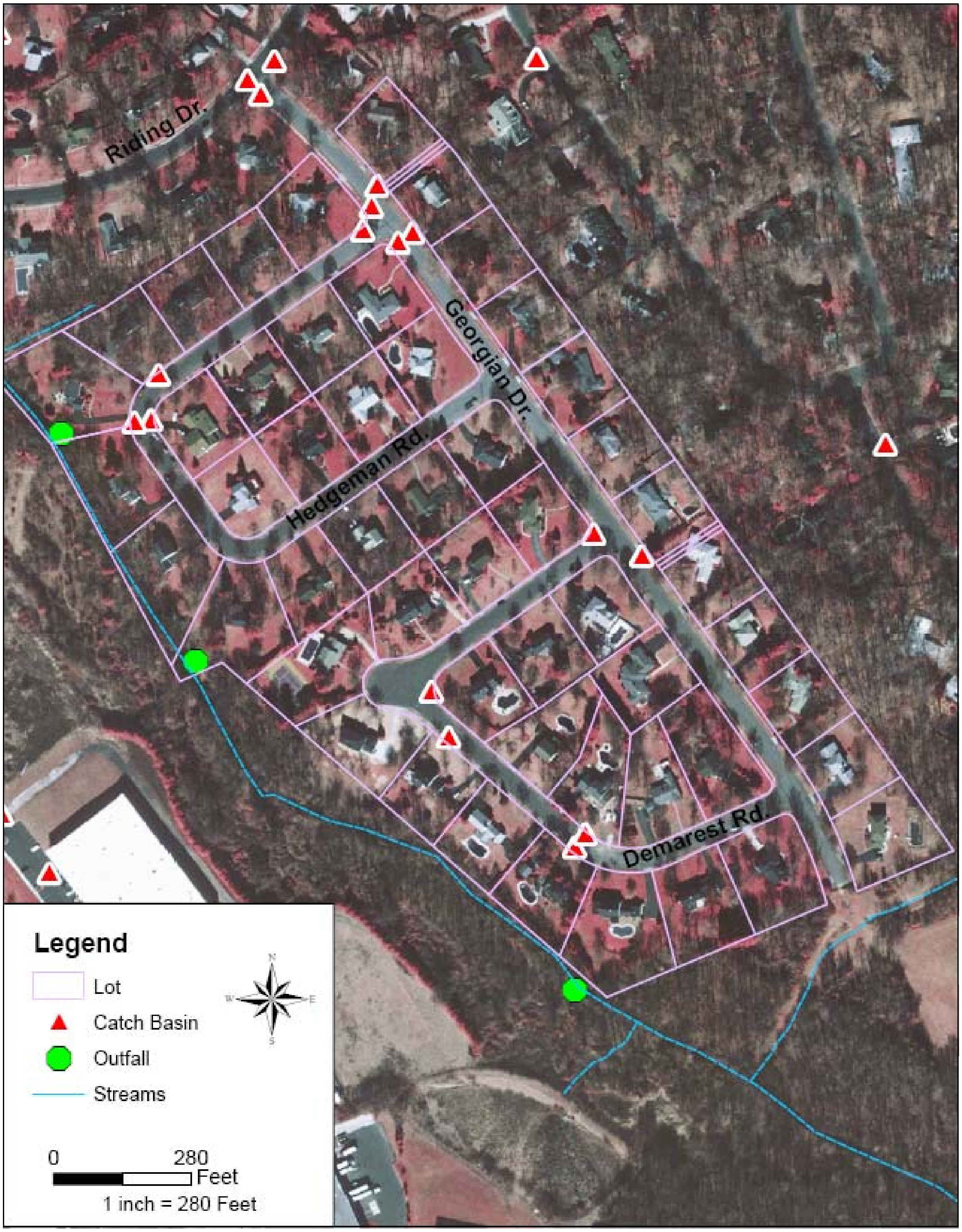

Figure 6: Georgian Drive Subdivision

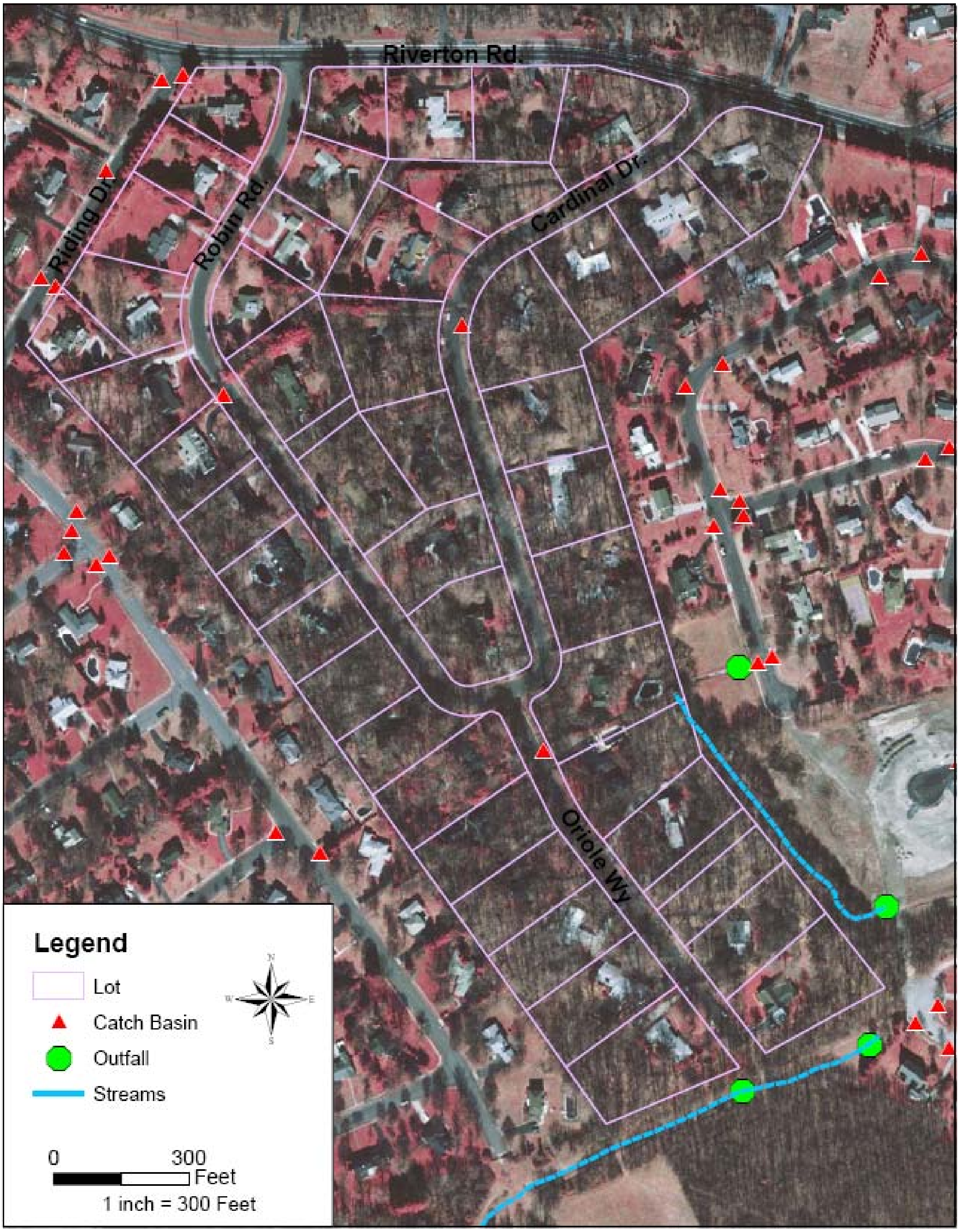

Figure 7: Cardinal Drive Subdivision

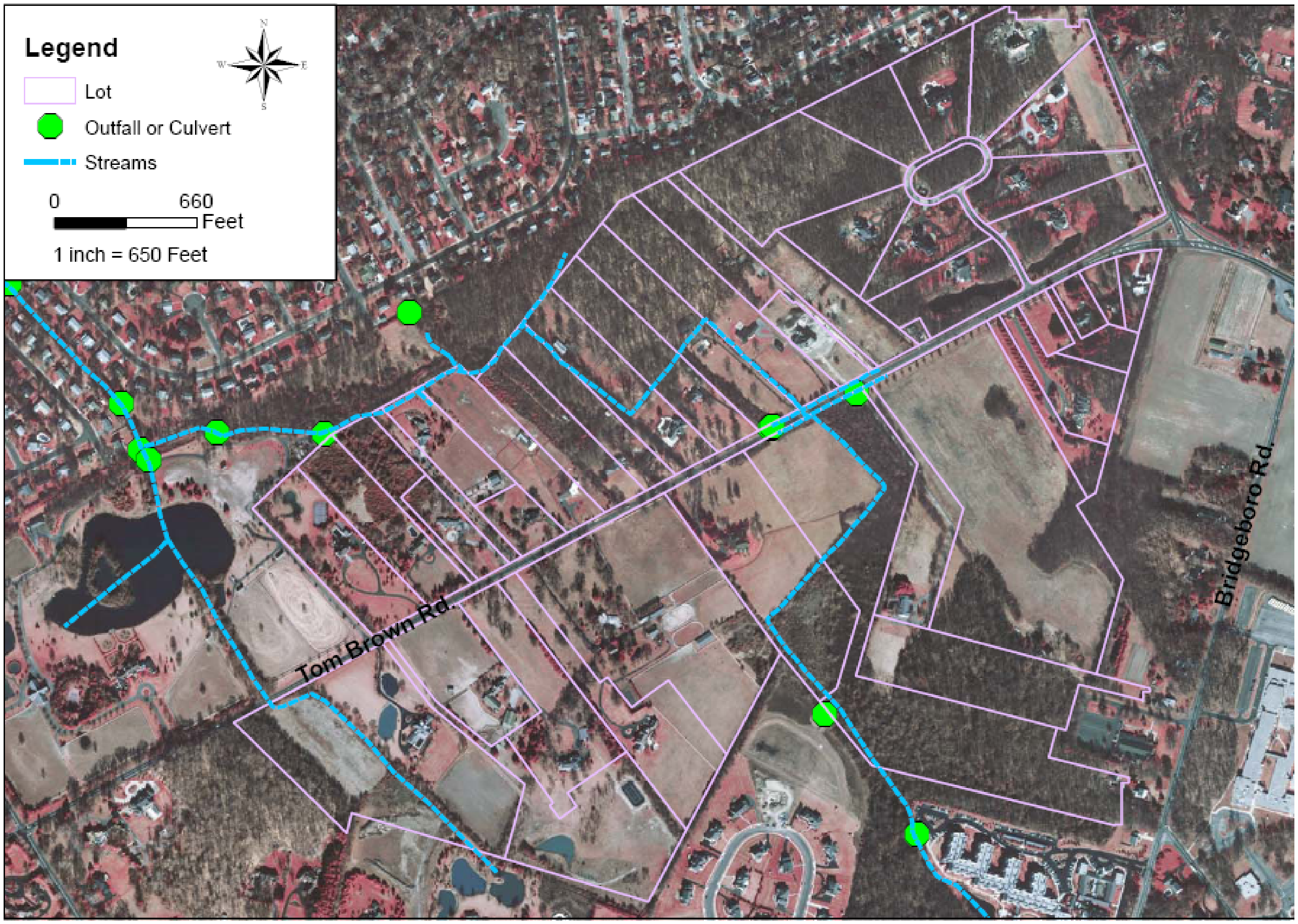

Figure 8: Tom Brown Rd Subdivision

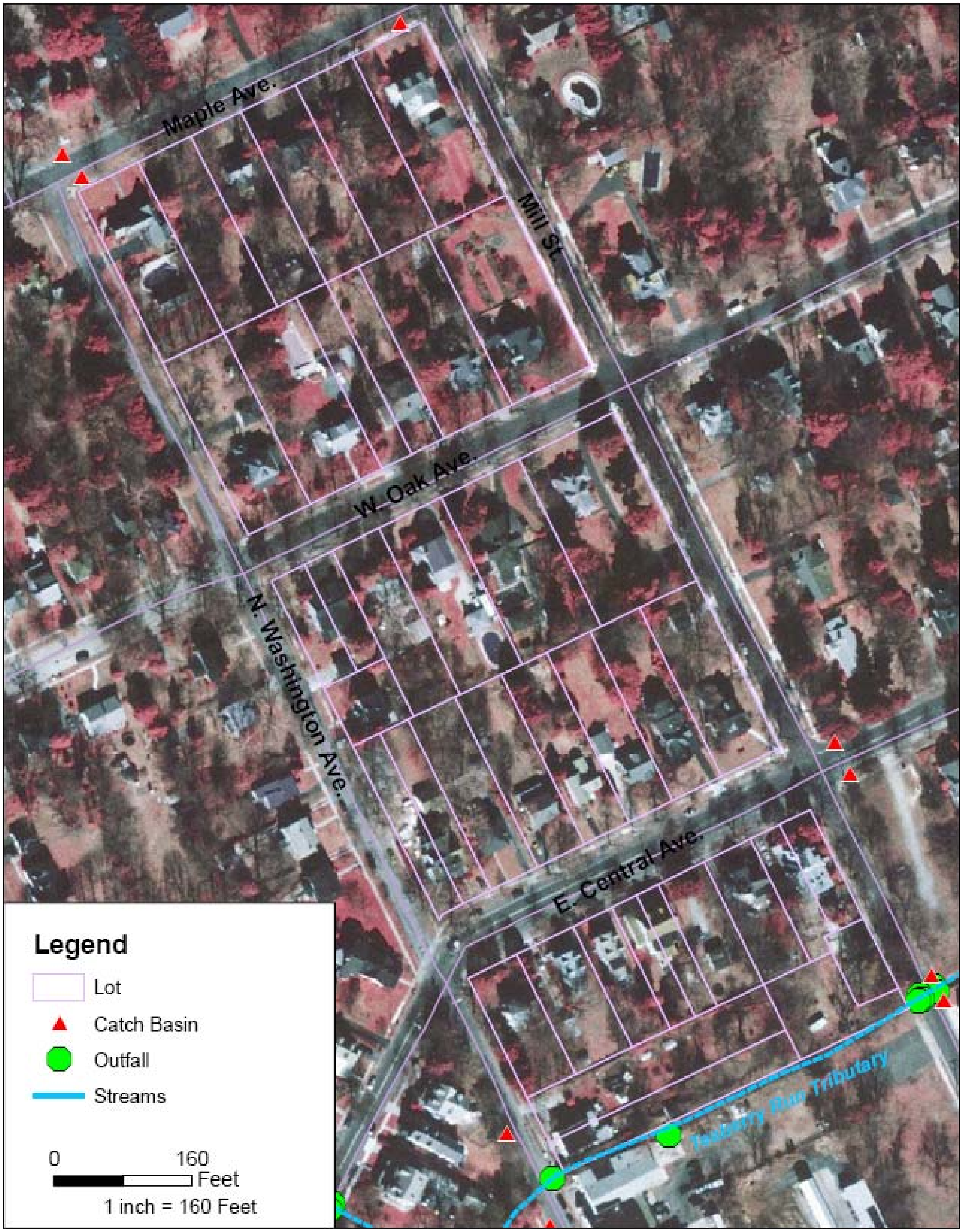

Figure 9: North Washington and East Central Ave. Subdivision

Figure 10: Outfall on the West Branch of the Pompeston Creek Draining Residential Area

Based on the field survey, it was assumed that the existing conditions situation was that 50% of the rooftops in the study areas were already disconnected from the drainage network. Stage 1 disconnection was the disconnection of 100% of the rooftops, selected to simulate the effect of 100% homeowner participation in a rooftop disconnection program. This is the management strategy which is most achievable since it requires the minimal amount of capital and effort. Stage 2 disconnection was determined to reflect the disconnection of all the rooftops, and half the driveways in a residential area. It requires more effort to divert driveway runoff, especially if the driveway slopes toward the street, therefore, it may not be feasible to easily retrofit all driveways with disconnection. For analysis purposes, it was assumed that it was reasonable to disconnect 50% of the driveways in addition to 100% of the rooftops in a residential area. Stage 3 (100% impervious surface disconnection) was the final simulation performed to analyze the maximum benefit of disconnection that could theoretically be obtained. Also, since most rooftops and driveways ultimately drain to the street, the streets could not be totally disconnected without considering runoff from these impervious areas.

Table 2: Stages of Disconnection

Disconnection Stage	**Description**
Existing Conditions	50% rooftops disconnected
Stage 1	100% rooftops disconnected
Stage 2	100% rooftops + 50% driveways disconnected
Stage 3	100% impervious surface disconnected (rooftops + driveways + streets)

Curve numbers were selected for each subdivision to represent the runoff volume generated by each stage of disconnection for the water quality, 2-, 10-, and 100-year storms. This method was used to recreate the composite curve numbers in TR-55 Table 2-2a specific to each study area for the water quality through the 100-year storm. A sample disconnection calculation is given for the Georgian Drive subdivision in Table 3. The runoff volume for the existing conditions scenario is calculated for the water quality storm over a 1-acre lot of the same characteristics of Georgian Drive. Rooftops account for 2.91 acres (or 7.83%) of the 37.17 acres in the Georgian Drive subdivision. In the existing conditions scenario, 50% of the rooftops are disconnected. Therefore, 3.91% of the total area is disconnected. If the total impervious area of Georgian Drive is 9.17 acres, or 25%, a 1-acre lot representative of Georgian Drive, would be 0.75 pervious acres, 0.211 connected impervious acres, and 0.039 disconnected impervious acres. The ratio of EIA/TIA = (0.25-0.039)/0.25= 0.844. The impervious curve number was selected as 98, while the pervious curve number for the B soil in good condition was selected as 61. The runoff volume during the water quality storm over the disconnected impervious area was calculated as 146 ft^3. This volume was added to the rainfall depth, assuming the disconnected volume is spread over the entire pervious area. The new rainfall depth is 1.30". Runoff volume from the pervious surface is calculated as 0.3 ft^3. Finally, the volume from the connected impervious area is calculated as 792 ft^3. The sum of the pervious and the connected impervious runoff volumes is the runoff volume for a 1-acre lot. This volume is 792.3 ft^3. A curve number is then selected to represent the runoff over a 1-acre lot during the water quality storm. The curve number for 50% rooftop disconnection, during the water quality storm, over the Georgian Drive study area is 82. Even with 50% rooftop disconnection, the new curve number is higher than the original composite curve number of 70 designated for Georgian Drive (Table 1). This *increase* in curve number will be discussed later.

Table 3: Existing Conditions Disconnection Calculation for Georgian Dr. during the water quality Storm

	Acres	Curve Number	
Connected Impervious Surface	0.211	98	
Pervious Area	0.75	61	
Disconnected Impervious Surface	0.039	98	
Total Area =	1		
Volume for terms in SCS equation (inches)	Disconnected Impervious Area	Pervious Area	Connected Impervious Area
$P =$	1.25	1.30	1.25
$I_a =$	0.04	1.28	0.04
$S =$	0.20	6.39	0.20
$Q =$	1.03	0.00	1.03
Runoff Volume (ft^3)	146	0.3	792
Final Disconnected Volume(ft^3)=	**792.3**		
Representative Curve Number =	**82**		

The ratio of effective impervious area to total impervious area (EIA/TIA) for each stage of disconnection was calculated for each study area. The EIA was calculated as the impervious surface still connected to the drainage network after disconnection. Alley and Veenhuis (1983) determined that EIA/TIA, although was regionally specific, did not vary by lot size. Therefore, each stage of disconnection was characterized by the average EIA/TIA ratio of the four subdivisions. Average EIA/TIA ratios are given for each stage of disconnection in Table 5. The ratio EIA/TIA was used to build a curve number table for the residential land use breakdown given in the NJDEP 1995/1997 Land Use/Land Cover data set given in Table 4 for use in TR-55. The residential land use categories were multiple dwelling high density, single unit medium density, single unit low density, and single unit rural. The EIA/TIA ratio was applied to the given range of percent imperviousness for the land use categories to calculate runoff for a 1-acre parcel using a method similar to that outlined in Table 3.

Table 4: NJDEP 1995/1997 Residential Land Use/Land Cover used in TR-55

Residential Land Use	% Impervious Surface	Lot size (acres)
High Density, Multiple Dwellings	> 65	1/8-1/5
Medium Density, Single Unit	30-35	1/8-1/2
Low Density, Single Unit	20-25	1/2-1
Rural, Single Unit	5-25	1-2+

For each land use category the percent impervious surface given in the table was broken into percent connected and percent disconnected area using the ratio of EIA/TIA for each stage of disconnection. For example, the residential, single unit, medium density land use is associated with a 30-35% impervious surface cover. To simulate Stage 1 disconnection, the EIA/TIA of 0.671 was applied to the range of percent impervious cover. The calculations were performed for both the low and high value of percent impervious surface and a median value was selected. For example, if 30% impervious surface is assumed, the EIA for a 1-acre design lot was (0.671)x(0.30) = 0.201 acres. The disconnected area was 0.30-0.201 = 0.099 acres and the pervious area was 0.70 acres. The disconnection method shown in Table 4 was applied to calculate a final runoff volume for the 1-acre lot. Runoff volumes were calculated for all design storms (water quality, 2-, 10-, 100-year) and hydrologic soil groups (A, B, C, D) for each land use category and stage of disconnection. Due to the threshold of runoff of the SCS equations at the water quality storm, a pervious curve number of 61 was used for both the A and B soils. TR-55 suggests a curve number of 39 be used for A soils in good hydrologic condition. However, using any curve number less than 61 in the calculation would generate runoff at the water quality storm which is not expected. Curve numbers were back-calculated for the generated volumes to build a curve number table that reflects the stages of disconnection that could be used in the watershed analysis. The curve number that represented the average volume from the range of percent imperviousness was entered into the table.

Table 5: Average EIA/TIA for Stages of Disconnection

Disconnection Stage	Description	Average EIA/TIA ratio
Existing Condition	50% rooftops disconnected	0.84
Stage 1	100% rooftops disconnected	0.67
Stage 2	100% rooftops + 50% driveways disconnected	0.52
Stage 3	100% impervious surface disconnected (rooftops + driveways + streets)	0.0

Watershed Scale Disconnection

A HEC-HMS model calibrated with small storm rainfall-runoff data was used to evaluate if disconnection would have an impact on decreasing the volume of stormwater runoff at the watershed scale. The new curve number table that was developed after the subdivision scale analysis was applied to the entire calibrated Pompeston Creek HEC-HMS watershed model. The basin curve numbers derived through calibration with rainfall-runoff data were compared to the composite curve numbers calculated for each basin using the average weighted technique. The model results for four different disconnection scenarios were compared to the results using calibrated curve numbers and the base model curve numbers. The calibrated curve numbers were used to represent observed conditions while the composite curve number was used to represent the base model. Discharge volumes from the observed storms were compared to the calculated volumes from the composite curve number and the modeled disconnection volumes.

HEC-HMS Model Calibration

The watershed boundary of the Pompeston Creek watershed was initially delineated using HECGeoHMS 1.1 extension in ArcView 3.3. The NJDEP stream layer was edited using 2002 Digital NJDEP Orthoimagery and integrated with the 10-meter WMA18 Digital Elevation Model (DEM) (NJDEP) in AVSWAT2000. The modified DEM was imported into HECGeoHMS 1.1 for processing and delineation of the watershed boundary. The watershed was then subdivided into 13 basins in HECGeoHMS 1.1. Basin dimensions such as flow length, centroid, and width were also generated by HECGeoHMS 1.1. The basin model and map was then imported into HEC-HMS.

HEC-HMS is an interface that utilizes a number of user-specified loss and routing models that with careful selection of input parameters and hypothetical rainfall events can be used to model ungauged watersheds. The watershed in this study was gauged so it could be calibrated to analyze the selection of parameters used in the SCS method. Since the purpose of this study is to evaluate the utility of the SCS curve number in disconnection scenarios, the SCS method was selected to model watershed loss. The standard procedure when using the SCS equations to determine runoff response in HEC-HMS requires the input of a basin-wide curve number and corresponding initial abstraction value. The curve number is calculated for the base model by first assigning a specific curve number to each land use polygon in GIS. The input polygon was the New Jersey 1995/1997 Land Use /Land Cover data from NJDEP. The data in the file was updated to reflect land use changes observed in 2002 aerial photos. The composite curve number was initially calculated for each basin by intersecting the NJDEP 1995/1997 Land Use/Land Cover set with the SSURGO soils layer to determine a unique soil-cover complex for each polygon. TR-55 Table 2-2a was used to designate a curve number for each polygon based on land use and hydrologic soil group. Wetlands were given the curve number of 98, while water bodies were assigned a curve number of 100. A composite curve number was obtained for each basin, and the initial abstraction was determined using the relationship $I_a = 0.2S$ in the SCS equations (SCS 1986). Base model curve numbers and initial abstractions are shown in Table 6 for each basin. The watershed basins are shown in Figure 11.

Snyder's lag was used as the routing method since this was the method previously used to model the Pompeston Creek in the regional stormwater management plan (Rutgers Cooperative Extension 2005). Lag time was calculated for the base model using the equation

$t_p = CC_t(LL_c)^{0.3}$ where C_t is a basin coefficient typically in the range of 1.8 to 2.2; L is the length of the main stream from the outlet to the basin divide; L_c is the length along the main stream from the outlet to a point nearest the watershed centroid; and C is a conversion constant (0.75 for SI and 1.00 for the foot-pound system) (USACE 2001). L and L_c were calculated using basin processing in HECGeoHMS 1.1. The peaking coefficient (C_p) was set at 0.6, as recommended in the HEC-HMS user's manual. Snyder's parameters for the base model are listed in Table 7. The parameters C_t and C_p are best found via calibration since they are not physically derived parameters (USACE 2001).

Figure 11: Pompeston Creek numbered basins

Table 6: Basin Composite CN and I_a

Basin	Composite CN	Initial Abstraction (in)
8	79.4	0.52
9	81.1	0.47
13	81.1	0.47
12	84.7	0.36

Table 7: Base Model Snyder's Unit Hydrograph parameters

Basin	8	9	13	12
Snyder's Lag (hr)	1.69	2.22	1.99	2.61
Peaking Coefficient (C_p)	0.6	0.6	0.6	0.6

Hydrologic routing was performed using the Muskingum-Cunge Standard method. The Muskingum-Cunge standard method is based on the continuity equation and the diffusion form of the momentum equation. Routing coefficients are automatically computed by the program from specified parameters. Standard cross-sections can be circular or prismatic. Required input includes channel shape, length, energy slope, bottom width or diameter, channel side slope, and Manning's n roughness coefficient (USACE 2001). Manning's coefficient was set as 0.04, an approximate value for natural stream channels (Sturm 2001), and later adjusted during calibration. Other parameters were determined through HECGeoHMS 1.1 basin processing or field inspection. All parameters are shown in Table 8.

The model was calibrated using streamflow and rainfall data collected over a four month period in the fall of 2005. Three sites were gauged with WL15 Water Level Logger pressure transducers (Global Water Instrumentation, Inc) over a four month period (September through December 2005). Transducers were installed at the pour points of three sub-basins (Basin 13, Basin 12, and Basin 10) to measure water depth at 30 minute intervals. Gauge locations are shown on the HEC-HMS model in Figure 12. Basins 12 and 13 are the headwaters of the West Branch of the Pompeston Creek, while Basin 10 is a headwater to the East Branch of the Pompeston Creek. The transducer was fastened to a concrete block and deployed at the midline of the stream channel. Rebar was used to secure the concrete block to the stream bed. The data logger of the transducer was fastened to either a nearby tree or stump (See Figure 13 and Figure 14). The logger could be easily removed to periodically download recorded data. Flow at each site was periodically measured using the 2-dimensional open channel profiling cross sectional method and Flo-Mate 2000 flowmeter equipped with a standard wading rod and velocity sensor (Marsh-McBirney, Inc.). Streamflow was related to the water level observed by each transducer for a specific recording time. A rating curve was developed for each site and transducer based on this flow data. Data were collected for base flow conditions through a 4.38" storm. Once the rating curve was developed, the water level data recorded by the pressure transducer was converted to flow data in cubic feet per second (ft^3/s) and presented in hydrographs.

Table 8: Muskingum-Cunge Standard Parameters

Reach	14	13	8
Shape	Prism	Prism	Prism
Length (ft)	5906	1316	257
Energy Slope (ft/ft)	0.0062	0.0059	0.0001
Bottom Width (ft)	15	10	5
Side Slope (ft/ft)	1	1	1
Manning's n	0.04	0.04	0.04

An RG200 6" Tipping Bucket Rain Gauge (Global Water Instrumentation, Inc.) was installed in a central location in Basin 12 near New Albany Rd. to record storm depth at two minute intervals. The bucket would tip once for each 0.01" of rainfall. A pulse was sent to a

GL400 Data Logger (Global Water Instrumentation, Inc.) to record each tip of the bucket. Rainfall data were downloaded from the logger and compiled for use in the model. Both total rainfall depth and a time series distribution could be determined from the rainfall data.

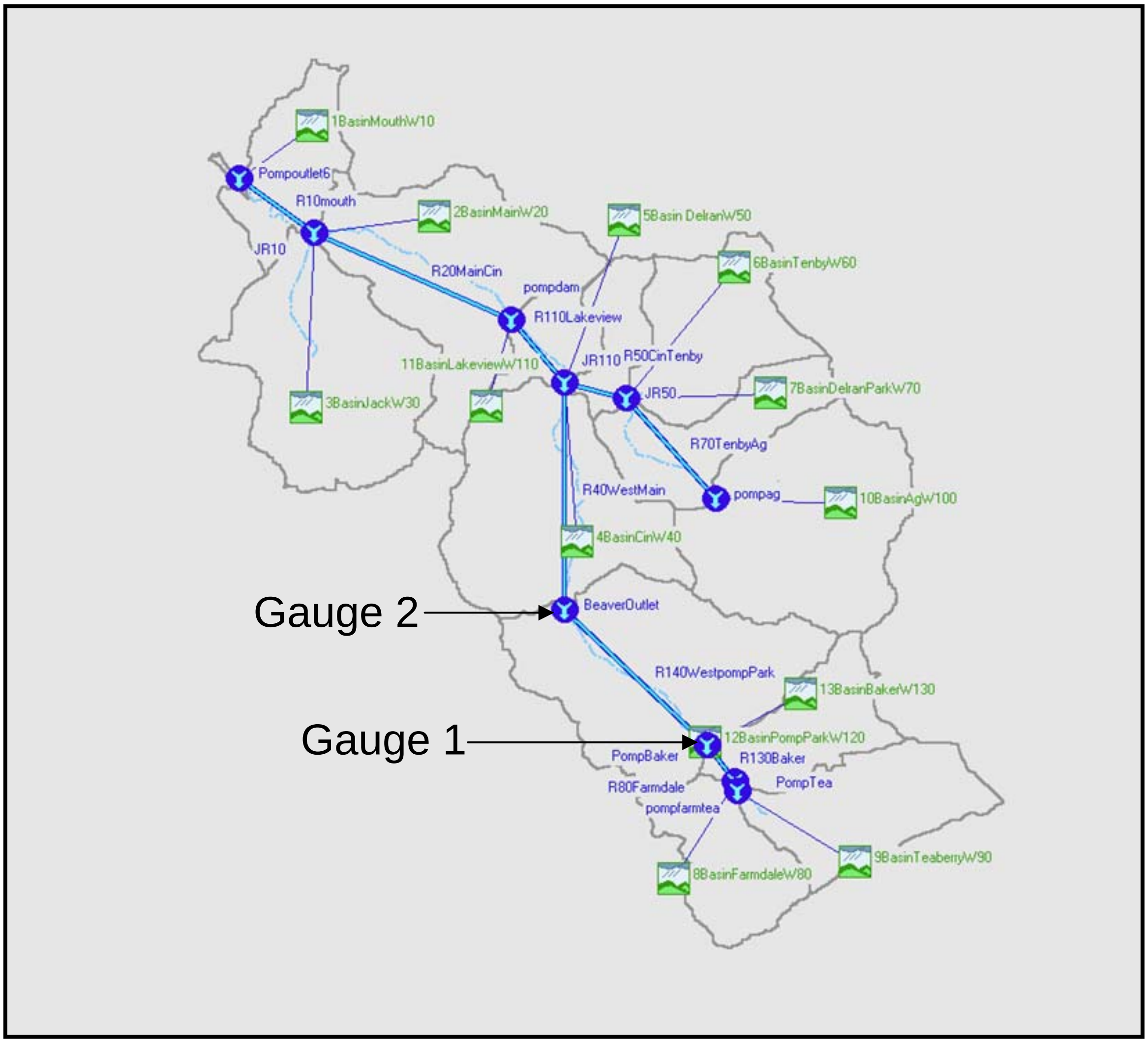

Figure 12: HEC-HMS model with stream gauges

Figure 13: Data Logger Secured to Tree at Site 2, North Riding Dr.

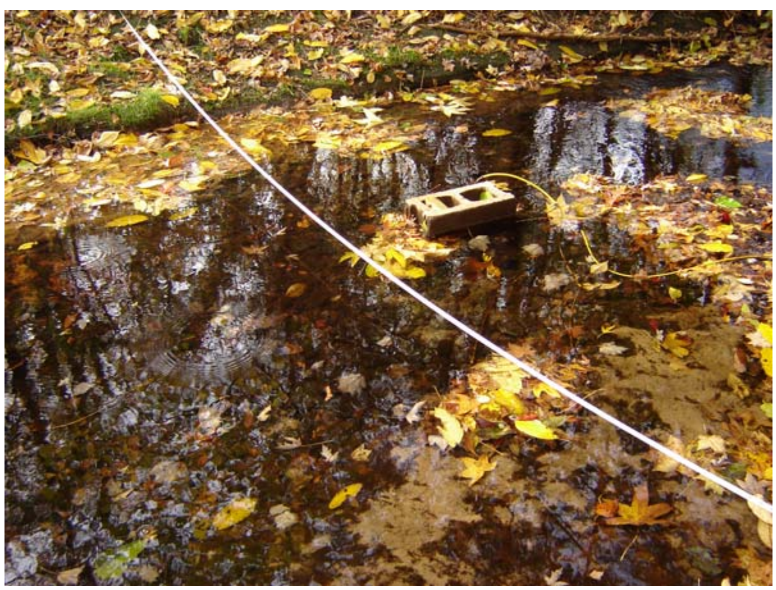

Figure 14: Pressure Transducer Deployed in the West Branch of the Pomepston Creek at Site 1

Three storms were recorded with complete sets of flow and rainfall data that were used in the calibration of the model at Site 1 (outlet of basin 13, drains basin 8, 9, and 13) and Site 2 (outlet of Basin 12) (Figure 12). Site 1 drained the headwater area of the West Branch of the Pompeston Creek. Site 2 is downstream of Site 1 and includes drainage from a small tributary and the wetlands throughout Pompeston Park. The stream was shallower and easier to access at Site 1 than at Site 2, therefore, it was much easier to measure flow at Site 1. Some difficulties were encountered in measuring streamflow at Site 2 after heavy rainfall events. There were technical issues with the pressure transducer at Site 3 (outlet of Basin 10 on the East Branch of the Pompeston Creek) and it did not record usable data, so this basin was omitted from analysis. Rainfall data were paired with streamflow data and entered into the model. HEC-HMS includes an option to enter data for observed discharge and precipitation gauges. Stream discharge was entered in cubic feet per second and rainfall was entered as inches with corresponding dates and time steps of the storm events. Data were entered into the gauge data view of HEC-HMS from an Excel spreadsheet. The control specifications were set in real time as to ensure the rainfall data were accurately synchronized with the response of runoff. The observed rainfall depths used in calibration were between 0.65” and 1.2”.

The model was optimized using the Peak-weighted RMS error method (USACE 2001), however, user-defined calibration obtained a better hydrograph fit. Observed hydrograph volumes at the gauge sites were calculated by HEC-HMS. The constant monthly baseflow method was used to separate baseflow from storm discharge. The average monthly baseflow was determined for each gauge site as the constant flow before storm discharge began and entered into the model. The baseflow during each storm duration was then calculated using the trapezoidal rule, and subtracted from the overall storm hydrograph to determine the direct runoff from the storm event. In the case of Site 2, which was downstream of Site 1, the observed volume from Site 1 was subtracted from Site 2 before baseflow separation. The remaining volume was then used to calibrate the curve number. The composite curve number in the base model was adjusted until the observed direct runoff volume (calculated by HEC-HMS) matched the calculated volume in the model. Each time the curve number was changed, the initial abstraction was also adjusted based on the SCS relationship $I_a = 0.2S$ for consistency since the maximum storage, S, is a function of the curve number through the relationship $S=(1000/CN)-10$. The timing of each peak in the hydrograph was adjusted by adjusting the Snyder's lag time. The optimization function in HEC-HMS was used to specify the peaking coefficient.

Once optimal parameters were determined for each storm, overall parameters were selected to best represent the response of all storms within the selected range (0.65" to 1.2"). A median curve number and its corresponding initial abstraction were selected. The curve number calibration for areas upstream of Site 1 was applied to all contributing basins (8, 9, and 13). The calibration at Site 2 was used to determine the curve number for Basin 12. An average value from the three storm events was used for the lag time and peaking coefficient. The model was then verified with simulations of each storm using the calibrated parameters. The model results of each storm were compared with observed flow.

Regression analyses were performed to determine the statistical significance between the measured and predicted data sets. The correlation and covariance were used to describe the model data because of their ability to directly compare the relationship between two data points. The observed and predicted output flows were compared for both Site 1 and Site 2 at one-minute intervals. The correlation between the two sets should provide the best description of how well the calibration performed to match the measured hydrograph.

Simulation of Disconnection on the Watershed Scale

Once the model was calibrated, it could then be used to compare the effects of disconnection against existing watershed conditions. It should be noted that the model is only calibrated against actual rainfall-runoff data for Basins 8, 9, 13, and 12. These basins are the focus of the analysis. The model was used to predict how the runoff response would change for each basin if the various stages of disconnection were used as a stormwater management strategy. New basin curve numbers were calculated for each basin using the new curve number tables derived for the existing conditions and the stages of disconnection. In this way the watershed effects of 100% rooftop disconnection, 100% rooftop and 50% driveway, and 100% impervious surface disconnection could possibly be determined. Hydrographs and total discharge volumes were determined for each basin, and compared to the results of the model of existing conditions. The basin curve number for each stage of disconnection was also compared to the calibrated curve number.

An analysis of the volume generated from each surface throughout the watershed was also performed. The runoff volume produced from each polygon was calculated for the water quality storm using the SCS equations and the "calculate" function in GIS. Each soil-cover complex polygon has a unique curve number and area. The curve number and area were used in the SCS equations to develop a data set of runoff volumes for each independent polygon for 1.25" of rainfall. Due to the threshold boundary of the curve number for the water quality storm, the lower bound of the curve number was set at 61. All curve numbers below 61 were set equal to 61. This represents the assumption that these surfaces do not generate runoff at the water quality storm because no runoff will be generated when a curve number of 61 is used. The runoff volumes were summed for each basin and compared to the volumes predicted for the water quality storm in the calibrated model. While this average weighted volume method does not account for any losses due to routing to adjacent pervious surfaces, existing stormwater management, or other forms of retention or loss throughout the drainage area, it can be used to approximate the runoff contribution from each land use through the watershed.

The area and contribution of expected runoff from each land use during the water quality storm was calculated. Land use was separated into type: urban, agriculture, forest, wetlands, water, and barren land. The urban area was further broken down into commercial/services, industry, recreational, residential, and transportation related categories. The final breakdown was of the residential category into high density, multiple dwelling; single unit medium density; single unit low density; and single unit rural. The volume and land use analysis can be used to isolate areas of concern where high local runoff volumes would be expected (Molgen and Beighly 2002).

RESULTS

Lot Scale Disconnection

The total area of each surface category (pervious, rooftops, and driveways) was calculated for each lot and an average value was calculated for the entire study area (See Table 9). Although curve numbers are provided in TR-55 for residential districts by average lot size, these curve number calculations include streets and are not representative of individual lots. Therefore, composite curve numbers were calculated for the average individual lot for each study area based on the lot impervious surface area (See Table 10). The newly calculated composite curve numbers in Table 10 are lower than the curve numbers given in TR-55 because streets were not included. Using these curve numbers, the SCS equations were used to determine runoff volumes for each lot for the water quality, 2-, 10-, and 100-year storms (See Table 10). It is clear from the literature review that the use of composite curve numbers for urbanized areas tends to result in an under-estimation of runoff volumes for small storms. To further explore this hypothesis, the total runoff contribution from each impervious surface was calculated for various storm events. In this way the areas responsible for the greatest runoff contribution for each storm could be isolated. The breakdown of the total runoff contribution from each impervious surface, is shown in Table 11 for the water quality, 2-, 10-, and 100-year storms. As the storm depth increases, the overall percent contribution from impervious surfaces decreases. In the Georgian Drive study area, impervious surface contributes 99.9% of the total runoff volume for the water quality storm. Impervious surface accounts for 49.7%, 33.2%, and 53.1% of the total runoff volume in the Cardinal Drive, Tom Brown Rd., and E. Central Ave. and N. Washington Ave. study areas, respectively.

Table 9: Average impervious and pervious area for lots in each study area

	Average Area (acres)				
Study area	**Rooftop**	**Driveway**	**Total Impervious**	**Pervious**	**Total Lot**
Georgian Dr.	0.05	0.03	0.08	0.50	0.58
Cardinal Dr.	0.09	0.06	0.15	0.96	1.11
Tom Brown Rd.	0.11	0.21	0.32	5.52	5.84
N. Washington & E. Central Ave.	0.04	0.02	0.06	0.31	0.37

Table 10: Composite Curve Number and Runoff Volume for the Average Lot in Each Study Area for the Water Quality, 2-, 10-, and 100-year Storms

Study Area	Composite Curve Number for Average Lot	Composite CN Runoff Volume (ft^3)
1.25" Storm		
Georgian Dr.	66	23
Cardinal Dr.	77	493
Tom Brown Rd	75	1,937
E. Central Ave & N. Washington Ave	78	186
3.4" Storm		
Georgian Dr.	66	1,626
Cardinal Dr.	77	5,560
Tom Brown Rd	75	26,512
E. Central Ave & N. Washington Ave	78	1,943
5.2" Storm		
Georgian Dr.	66	4,029
Cardinal Dr.	77	11,387
Tom Brown Rd	75	55,956
E. Central Ave & N. Washington Ave	78	3,927
8.8" Storm		
Georgian Dr.	66	10,030
Cardinal Dr.	77	24,440
Tom Brown Rd	75	123,062
E. Central Ave & N. Washington Ave	78	8,341

Table 11: Breakdown of Runoff Contributions from Impervious Lot Surfaces for the Water Quality, 2-, 10-, and 100-year Storms

Study Area	Roof Runoff Volume (ft^3)	% Roof Volume	Driveway Runoff Volume (ft^3)	% Driveway Volume	Impervious Surface Runoff Volume (ft^3)	% Total Impervious Surface Volume
1.25" Storm						
Georgian Dr.	195	60.8%	125	39.1%	321	99.9%
Cardinal Dr.	344	41.3%	231	27.8%	575	69.1%
Tom Brown Rd	417	15.5%	793	29.5%	1,211	45.1%
E. Central Ave & N. Washington Ave	163	50.0%	81	24.8%	245	74.8%
3.4" Storm						
Georgian Dr.	597	30.8%	384	19.8%	981	50.6%
Cardinal Dr.	1,053	18.0%	708	12.1%	1,760	30.1%
Tom Brown Rd	1,278	4.7%	2,428	8.9%	3,706	13.6%
E. Central Ave & N. Washington Ave	500	24.3%	248	12.1%	748	36.4%
5.2" Storm						
Georgian Dr.	936	22.1%	602	14.2%	1,538	36.2%
Cardinal Dr.	1,650	14.3%	1,109	9.6%	2,759	23.9%
Tom Brown Rd	2,002	3.6%	3,806	6.8%	5,808	10.3%
E. Central Ave & N. Washington Ave	784	19.6%	389	9.7%	1,173	29.4%
8.8" Storm						
Georgian Dr.	1,615	16.1%	1038	10.3%	2,653	26.4%
Cardinal Dr.	2,846	11.6%	1912	7.8%	4,759	19.5%
Tom Brown Rd	3,453	2.8%	6564	5.3%	10,017	8.1%
E. Central Ave & N. Washington Ave	1,352	16.2%	671	8.1%	2,023	24.3%

Table 12: Composite Curve Number Runoff Volume and Volume Runoff for Separate Impervious and Pervious Areas for the Water Quality, 2-, 10-, and 100-year Storms

Study Area	Composite CN Runoff Volume (ft^3)	Impervious Surface Runoff Volume (ft^3)		Pervious Surface Runoff Volume (ft^3)		Total Runoff Volume (ft^3)	% Volume Difference
1.25" Storm							
Georgian Dr.	23	321	+	0	=	321	1,300%
Cardinal Dr.	493	575	+	257	=	832	69%
Tom Brown Rd	1,937	1,211	+	1476	=	2,687	39%
E. Central Ave & N. Washington Ave	186	245	+	82	=	327	76%
3.4" Storm							
Georgian Dr.	1,626	981	+	960	=	1,941	19%
Cardinal Dr.	5,560	1,760	+	4,082	=	5,842	5%
Tom Brown Rd	26,512	3,706	+	23,446	=	27,152	2%
E. Central Ave & N. Washington Ave	1,943	748	+	1,310	=	2,058	6%
5.2" Storm							
Georgian Dr.	4,029	1,538	+	2,706	=	4,244	5%
Cardinal Dr.	11,387	2,759	+	8,796	=	11,555	1%
Tom Brown Rd	55,956	5,808	+	50,535	=	5,6343	1%
E. Central Ave & N. Washington Ave	3,927	1,173	+	2,823	=	3,996	2%
8.8” Storm							
Georgian Dr.	10,030	2,653	+	7,378	=	10,031	0%
Cardinal Dr.	24,440	4,759	+	19,673	=	24,432	0%
Tom Brown Rd	123,062	10,017	+	113,028	=	123,045	0%
E. Central Ave & N. Washington Ave	8,341	2,023	+	6,314	=	8,337	0%

A comparison was then made between the runoff volume generated by using the composite curve number method and the volume weighted method for each average lot. As shown in Table 12, even though there is very little difference between the results from the two methods for the larger storms, the water quality storm has percent differences in volumes ranging from 39% to 1,300%. Although the directly connected volume from the volume weighted method is assumed to be a more realistic representation of the runoff volume because it includes the contribution of impervious surface during small rainfall events, it is important to note that most of the residential development in the Pompeston Creek Watershed already has some level of rooftop disconnection.

The effect of disconnection on the lot scale was theoretically examined with the simulation of two disconnection scenarios. The runoff from a lot with rooftop disconnection and the same lot with rooftop and driveway disconnection was simulated for each lot. The results of these simulations suggest the general volume reduction goals that can be achieved with disconnection for an individual lot. The total runoff volumes that were generated from disconnection of the rooftops and disconnection of all impervious surfaces (rooftops and driveways) for the average lot in each study area are presented in Table 13 and Table 14. As storm depth increases, the percent volume reduction from each lot due to disconnection decreases. The calculations show that disconnection may remove the greatest percentage of stormwater for the lots in the Georgian Drive study area during the water quality storm. Rooftop disconnection reduces the overall runoff volume by 60.4%, while disconnecting all impervious surfaces reduces the overall volume by 98.1%.

A summary of the overall lot runoff volume calculations for the various disconnection scenarios for each average lot are presented in Table 15. As discussed earlier, the composite curve number runoff calculation is the standard TR-55 method for determining runoff volumes. The next column in Table 15 provides a summary of the runoff volumes assuming that all the impervious surfaces are directly connected. For the water quality storm, these two calculations yield very different runoff volumes while there is little difference for the larger storms.

Table 13: Runoff Volume from Disconnection of all Rooftops for the Average Lot in Each Study Area for the Water Quality, 2-, 10-, and 100-year Storms

Study Area	Original Runoff (ft^3)	New Runoff (ft^3)	Disconnected volume (ft^3)	% Volume Disconnected
1.25" Storm				
Georgian Dr.	321	127	194	60.4%
Cardinal Dr.	832	581	251	30.2%
Tom Brown Rd	2,687	2,376	311	11.6%
E. Central Ave & N. Washington Ave	327	209	118	36.1%
3.4" Storm				
Georgian Dr.	1,941	1,616	324	16.7%
Cardinal Dr.	5,842	5,520	321	5.5%
Tom Brown Rd	27,152	26,748	405	1.5%
E. Central Ave & N. Washington Ave	2,058	1,909	149	7.3%
5.2" Storm				
Georgian Dr.	4,244	3,901	343	8.1%
Cardinal Dr.	11,555	11,255	300	2.6%
Tom Brown Rd	56,343	55,963	380	0.7%
E. Central Ave & N. Washington Ave	3,996	3,857	139	3.5%
8.8" Storm				
Georgian Dr.	10,031	9,711	320	3.2%
Cardinal Dr.	24,432	24,189	243	1.0%
Tom Brown Rd	123,045	122,733	312	0.3%
E. Central Ave & N. Washington Ave	8,337	8,225	112	1.3%

1.25" Storm				
Study Area	**Original Runoff (ft^3)**	**New Runoff (ft^3)**	**Disconnected volume (ft^3)**	**% Volume Disconnected**
Georgian Dr.	321	6	315	98.1%
Cardinal Dr.	832	418	414	49.7%
Tom Brown Rd	2,687	1,794	893	33.2%
E. Central Ave & N. Washington Ave	327	153	174	53.1%
3.4" Storm				
Study Area	**Original Runoff (ft^3)**	**New Runoff (ft^3)**	**Disconnected volume (ft^3)**	**% Volume Disconnected**
Georgian Dr.	1,941	1,420	520	26.8%
Cardinal Dr.	5,842	5,321	521	8.9%
Tom Brown Rd	27,152	26,001	1,152	4.2%
E. Central Ave & N. Washington Ave	2,058	1,842	216	10.5%
5.2" Storm				
Study Area	**Original Runoff (ft^3)**	**New Runoff (ft^3)**	**Disconnected volume (ft^3)**	**% Volume Disconnected**
Georgian Dr.	4,244	3,698	546	12.9%
Cardinal Dr.	11,555	11,072	483	4.2%
Tom Brown Rd	5,6343	55,265	1,078	1.9%
E. Central Ave & N. Washington Ave	3,996	3,797	200	5.0%
8.8" Storm				
Study Area	**Original Runoff (ft^3)**	**New Runoff (ft^3)**	**Disconnected volume (ft^3)**	**% Volume Disconnected**
Georgian Dr.	10,031	9,525	507	5.1%
Cardinal Dr.	24,432	24,042	390	1.6%
Tom Brown Rd	123,045	122,165	879	0.7%
E. Central Ave & N. Washington Ave	8,337	8,177	160	1.9%

Table 14: Runoff Volume from 100% Impervious Surface Disconnection for the Average Lot in Each Study Area for the Water Quality, 2-, 10-, and 100-year Storms

Table 15: Calculated Runoff Volume for Various Disconnection Scenarios for the Average Lot in Each Study Area for the Water Quality, 2-, 10-, and 100-year Storms

Study Area and Design Storm	Composite CN Runoff Volume (ft^3)	Runoff Volume (ft^3): All Impervious Surfaces are Directly Connected	Runoff Volume (ft^3): Rooftops are Disconnected	Runoff Volume (ft^3): Rooftops & Driveways are Disconnected
Georgian Dr.				
Water Quality	23	321	127	6
2-yr	1,626	1,941	1,616	1,420
10-yr	4,029	4,244	3,901	3,698
100-yr	10,030	10,031	9,711	9,525
Cardinal Dr.				
Water Quality	493	832	581	418
2-yr	5,560	5,842	5,520	5,321
10-yr	11,387	11,555	11,255	11,072
100-yr	24,440	24,432	24,189	24,042
Tom Brown Rd.				
Water Quality	1,937	2,687	2,376	1,794
2-yr	26,512	27,152	26,748	26,001
10-yr	55,956	56,343	55,963	55,265
100-yr	123,062	123,045	122,733	122,165
E. Central & N. Washington				
Water Quality	186	327	209	153
2-yr	1,943	2,058	1,909	1,842
10-yr	3,927	3,996	3,857	3,797
100-yr	8,341	8,337	8,225	8,177

Subdivision Scale Disconnection

The next larger scale of analysis was the subdivision scale and included groups of adjacent lots that shared common streets and drained to a common area. The main difference between the subdivision and lot scale analysis was the inclusion of streets. The subdivisions were divided into categories of impervious (rooftop, driveway, streets) and pervious surfaces by total area and percentage (See Table 16 and Table 17). Streets are the greatest impervious elements in the Georgian Drive, Cardinal Drive,, and N. Washington and E. Central Ave subdivisions and account for 12%, 10%, and 13% of the total land area. Driveways are the largest percent of impervious cover and are 4% of the total area at Tom Brown Rd. Composite curve numbers were calculated for each subdivision with the information presented in Table 16 and the soil type. Runoff volumes for each subdivision were calculated using the composite curve number method for the water quality storm. These volumes are presented in Table 18.

Runoff volume for the subdivisions was also calculated using the weighted volume method to simulate the effect of directly connected impervious surface. For the water quality storm, total runoff contribution from each surface and the sum of the volumes is given in Table 19. The percent of the total volume from each surface is given in Table 20. For the water quality storm, streets contribute the greatest percentage of overall runoff from Georgian Drive, Cardinal Drive, and N. Washington and E. Central Ave. The pervious area contributes the largest percent of runoff (51%) from Tom Brown Rd. The volume calculated using the composite curve number method is compared to the volume weighted method in Table 21. At the water quality storm, the composite curve number volume was less than the weighted volume for all four subdivisions.

Table 16: Sum of Impervious and Pervious Surface for Each Study Area

	Subdivision Impervious and Pervious Surface Area (acres)					
Subdivision	**Streets**	**Driveways**	**Rooftops**	**Total Impervious**	**Pervious**	**Total Area**
Georgian Dr	4.39	1.87	2.91	9.17	28.00	37.17
Cardinal Dr.	2.13	1.05	1.56	4.74	16.32	21.05
Tom Brown Rd.	1.25	4.44	2.33	8.02	115.79	123.8
N. Washington & E. Central	1.68	0.67	1.35	3.70	9.55	13.25

Table 17: Percent of Impervious and Pervious Surface for Each Study Area

	Subdivision Impervious and Pervious Surface Area by %					
Subdivision	**Streets**	**Driveways**	**Rooftops**	**Total Impervious**	**Pervious**	**Total Area**
Georgian Dr	12%	5%	8%	25%	75%	100%
Cardinal Dr.	10%	5%	8%	23%	77%	100%
Tom Brown Rd.	1%	4%	2%	7%	93%	100%
N. Washington & E. Central	13%	5%	10%	28%	72%	100%

Table 18: Subdivision composite curve number and runoff volume from water quality storm

Subdivision	**Total Area (acres)**	**Composite CN**	**Composite CN Runoff Volume (ft^3)**
Georgian Dr	37.17	70	4,443
Cardinal Dr.	21.05	79	11,674
Tom Brown Rd.	123.80	77	45,520
N. Washington & E. Central Ave.	13.25	81	9,365

Table 19: Volume contribution from impervious surface type at the water quality storm

	Runoff Volume (ft^3)				
Subdivision	**Streets**	**Driveways**	**Rooftops**	**Pervious**	**Volume Sum**
Georgian Dr	16,422	7,013	10,890	0	34,325
Cardinal Dr.	7,971	3,920	5,837	4,356	22,085
Tom Brown Rd.	4,661	16,596	8,712	31,015	60,984
N. Washington & E. Central	6,273	2,526	5,053	2,570	16,422

Table 20: Percent volume contribution from impervious surface type at the water quality storm

	% Total Runoff Volume				
Subdivision	**Streets**	**Driveways**	**Rooftops**	**Pervious**	**Volume Sum**
Georgian Dr	48%	20%	32%	0%	100%
Cardinal Dr.	36%	18%	26%	20%	100%
Tom Brown Rd.	8%	27%	14%	51%	100%
N. Washington & E. Central	38%	15%	31%	16%	100%

Table 21: Comparison of Directly Connected Volume and Composite CN Volume for the water quality Storm

Subdivision	Volume Sum (ft^3)	Composite CN Runoff Volume (ft^3)	% Volume Difference
Georgian Dr	34,325	4,443	77%
Cardinal Dr.	22,085	11,674	47%
Tom Brown Rd.	60,984	45,520	25%
N. Washington & E. Central Ave	16,422	9,365	43%

A theoretical simulation was performed to analyze disconnection at the water quality storm by routing the runoff volume from each impervious surface over the adjacent pervious areas. Disconnection of only the rooftop area was simulated first. Driveway and then street disconnection were cumulatively added to the rooftop disconnection. These calculations approximate the runoff volume that could theoretically be reduced during the water quality storm if 100% disconnection of each surface was achieved. The volumes after disconnection are presented in Table 22. The percent volume reduction column in Table 22 is the percent the directly connected volume is reduced through disconnection of the specified impervious elements. Table 23 shows the new curve number that was needed to produce the runoff volume from each disconnection scenario during the water quality storm.

Since in a realistic scenario not all of the impervious surfaces are directly connected, the next analysis was to establish the baseline amount of disconnection in each subdivision. Since field inspections of the subdivisions determined that approximately 50% of the rooftops were already disconnected, this condition was used to define the existing conditions. Three stages of disconnection were then selected to mimic the different achievable levels of stormwater management that could be implemented throughout the study areas. Runoff volumes were calculated for existing conditions and Stages 1 through 3 of disconnection for the water quality through 100-year storm for the four subdivisions in the study. These volumes are presented in Table 24, Table 25, Table 26, and Table 27. When compared to the volumes calculated for the stages of disconnection, the composite curve number volume fell between the volume generated for Stage 2 and Stage 3 disconnection in all subdivisions for the water quality storm (Table 28 and Figure 15). As storm depth increased, the composite curve number volume approached the existing conditions volume. The volumes in the following tables are presented in acre-feet for easier comparison at larger storm depths. One acre-foot is equal to a depth of one foot over an area of one acre. The conversion is 1 acre-foot = 43,560 ft^3.

Table 22: Runoff volume based on degrees on disconnection during the water quality storm

Subdivision	Disconnected Surface	New Volume (ft^3)	Volume Reduction (ft^3)	% Volume Reduction
Georgian Dr.	None	34,325	0	0%
	Rooftops	23,522	10,803	31%
	Rooftops & Driveways	16,770	17,555	51%
	Rooftops, Driveways, & Streets	1,481	32,844	96%
Cardinal Dr.	None	22,085	0	0%
	Rooftops	17,816	4,269	19%
	Rooftops & Driveways	15,159	6,926	31%
	Rooftops, Driveways, & Streets	9,714	12,371	56%
Tom Brown Rd	None	60,984	0	0%
	Rooftops	54,406	6,578	11%
	Rooftops & Driveways	42,297	18,687	31%
	Rooftops, Driveways, & Streets	38,768	22,216	36%
N. Washington & E. Central	None	16,422	0	0%
	Rooftops	12,807	3,615	22%
	Rooftops & Driveways	11,021	5,401	33%
	Rooftops, Driveways, & Streets	6,970	9,452	58%

Table 23: Subdivision Disconnection Runoff Volumes and New Curve Numbers for the water quality Storm

Subdivision	Disconnected Surface	Volume after Disconnection (ft^3)	New Curve Number
Georgian Dr.	None	34,325	83
	Rooftops	23,522	80
	Rooftops & Driveways	16,771	78
	Rooftops, Driveways, & Streets	1,481	67
Cardinal Dr.	None	22,085	85
	Rooftops	17,816	83
	Rooftops & Driveways	15,159	81
	Rooftops, Driveways, & Streets	9,714	78
Tom Brown Rd	None	60,984	78
	Rooftops	54,406	77
	Rooftops & Driveways	42,297	76
	Rooftops, Driveways, & Streets	38,768	75
N. Washington & E. Central	None	16,422	86
	Rooftops	12,807	84
	Rooftops & Driveways	11,021	83
	Rooftops, Driveways, & Streets	6,970	79

Table 24: Volume reductions for stages of disconnection for the water quality Storm

Subdivision	Disconnected Surface	Runoff Volume (acre-ft)	Volume Reduction (acre-ft)	% Volume Reduction
Georgian Dr.	Existing Conditions	0.68	0.00	0%
	100% Rooftops	0.55	0.12	18%
	100% Rooftops & 50% Driveways	0.48	0.20	30%
	All Impervious Surface	0.03	0.64	95%
Cardinal Dr.	Existing Conditions	0.47	0.00	0%
	100% Rooftops	0.42	0.05	10%
	100% Rooftops & 50% Driveways	0.39	0.08	17%
	All Impervious Surface	0.23	0.24	51%
Tom Brown Rd.	Existing Conditions	1.28	0.00	0%
	100% Rooftops	1.21	0.08	6%
	100% Rooftops & 50% Driveways	1.06	0.22	17%
	All Impervious Surface	0.88	0.40	31%
E. Central & N. Washington Ave.	Existing Conditions	0.34	0.00	0%
	100% Rooftops	0.29	0.04	12%
	100% Rooftops & 50% Driveways	0.27	0.06	18%
	All Impervious Surface	0.16	0.18	52%

Table 25: Volume reductions for stages of disconnection for the 3.4" Storm

Subdivision	Disconnected Surface	Runoff Volume (acre-ft)	Volume Reduction (acre-ft)	% Volume Reduction
Georgian Dr.	Existing Conditions	3.47	0.00	0%
	100% Rooftops	3.26	0.20	6%
	100% Rooftops & 50% Driveways	3.14	0.33	10%
	All Impervious Surface	2.45	1.02	29%
Cardinal Dr.	Existing Conditions	2.80	0.00	0%
	100% Rooftops	2.73	0.06	2%
	100% Rooftops & 50% Driveways	2.69	0.10	4%
	All Impervious Surface	2.51	0.29	10%
Tom Brown Rd.	Existing Conditions	13.23	0.00	0%
	100% Rooftops	13.12	0.10	1%
	100% Rooftops & 50% Driveways	12.94	0.29	2%
	All Impervious Surface	12.71	0.51	4%
E. Central & N. Washington Ave.	Existing Conditions	1.86	0.00	0%
	100% Rooftops	1.80	0.05	3%
	100% Rooftops & 50% Driveways	1.78	0.08	4%
	All Impervious Surface	1.65	0.21	11%

Table 26: Volume reductions for stages of disconnection for the 5.2" Storm

Subdivision	Disconnected Surface	New Volume (acre-ft)	Volume Reduction (acre-ft)	% Volume Reduction
Georgian Dr.	Existing Conditions	7.08	0.00	0%
	100% Rooftops	6.87	0.21	3%
	100% Rooftops & 50% Driveways	6.74	0.35	5%
	All Impervious Surface	6.03	1.05	15%
Cardinal Dr.	Existing Conditions	5.35	0.00	0%
	100% Rooftops	5.30	0.06	1%
	100% Rooftops & 50% Driveways	5.26	0.09	2%
	All Impervious Surface	5.09	0.26	5%
Tom Brown Rd.	Existing Conditions	27.47	0.00	0%
	100% Rooftops	27.37	0.10	0%
	100% Rooftops & 50% Driveways	27.20	0.27	1%
	All Impervious Surface	26.99	0.48	2%
E. Central & N. Washington Ave.	Existing Conditions	3.49	0.00	0%
	100% Rooftops	3.44	0.05	1%
	100% Rooftops & 50% Driveways	3.42	0.07	2%
	All Impervious Surface	3.30	0.19	5%

Table 27: Volume reductions for stages of disconnection for the 8.8" Storm

Subdivision	Disconnected Surface	Runoff Volume (acre-ft)	Volume Reduction (acre-ft)	% Volume Reduction
Georgian Dr.	Existing Conditions	15.86	0.00	0%
	100% Rooftops	15.66	0.20	1%
	100% Rooftops & 50% Driveways	15.54	0.32	2%
	All Impervious Surface	14.91	0.95	6%
Cardinal Dr.	Existing Conditions	11.03	0.00	0%
	100% Rooftops	10.99	0.05	0%
	100% Rooftops & 50% Driveways	10.96	0.08	1%
	All Impervious Surface	10.82	0.21	2%
Tom Brown Rd.	Existing Conditions	59.99	0.00	0%
	100% Rooftops	59.91	0.08	0%
	100% Rooftops & 50% Driveways	59.77	0.22	0%
	All Impervious Surface	59.60	0.39	1%
E. Central & N. Washington Ave.	Existing Conditions	7.09	0.00	0%
	100% Rooftops	7.06	0.04	1%
	100% Rooftops & 50% Driveways	7.04	0.06	1%
	All Impervious Surface	6.95	0.15	2%

Table 28: Runoff volume from the water quality storm for all subdivisions through each stage of disconnection

	Runoff Volume (ac-ft)				
	Composite CN	Existing Conditions	Stage 1	Stage 2	Stage 3
Georgian Dr.	0.10	0.68	0.55	0.48	0.03
Cardinal Dr.	0.27	0.47	0.42	0.39	0.23
Tom Brown Rd.	1.04	1.28	1.21	1.06	0.88
E. Central & N. Washington Ave.	0.21	0.34	0.29	0.27	0.16

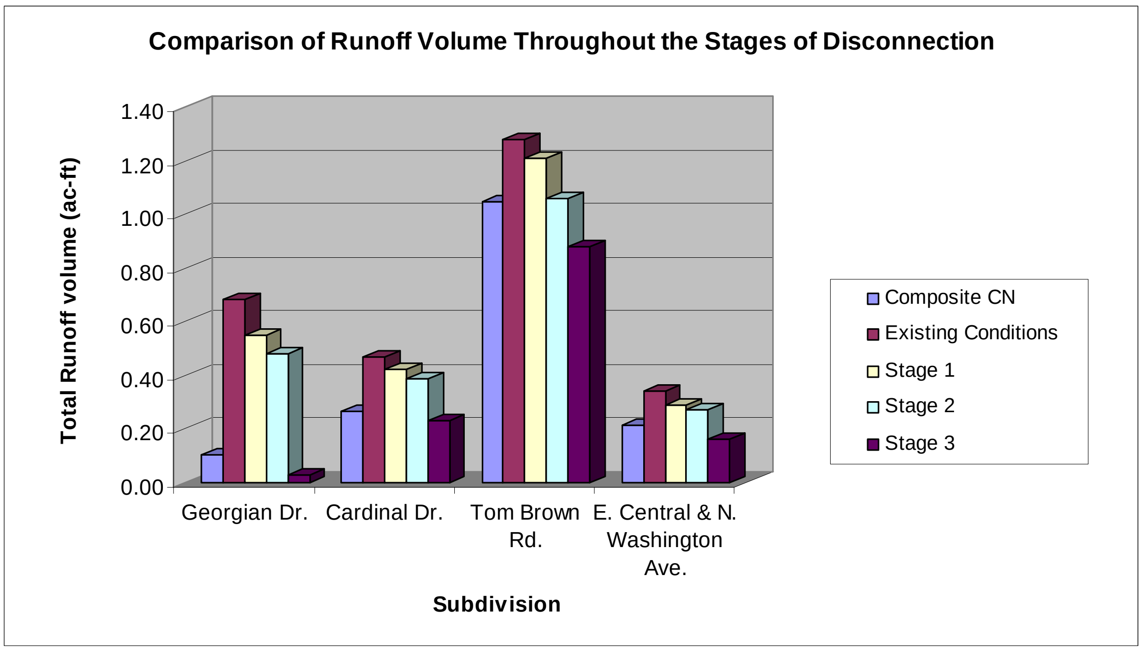

Figure 15: Comparison of Runoff Volume through the Stages of Disconnection at the water quality Storm

The calculated disconnection volumes were used to generate curve number tables specific to the subdivisions in the study. Table 29 represents the composite urban curve numbers given in TR-55. They are similar to the curve numbers calculated with percent impervious surface and the soil data. The curve numbers derived from the weighted volume method are listed with the disconnection curve numbers determined from the calculated disconnection volumes and are listed for each design storm and existing conditions, Stage 1, Stage 2, and Stage 3 disconnection in Table 30.

Table 29: Original Composite Curve Numbers for Each Study Area

Lot size (acres)	Subdivision	% IS	Soil	Composite CN
1/3	E. Central & N. Washington	28	C	81
½	Georgian Dr.	25	B	70
1	Cardinal Dr.	23	C	80
5	Tom Brown Rd.	6	C	77

The EIA and EIA/TIA ratio was calculated for each subdivision at each stage of disconnection in Table 31 so the observed data could be applied to the lot sizes used in the NJDEP Land Use/Land Cover classification. The average ratio was also calculated for each stage of disconnection and is summarized in Table 32. The ratio of EIA/TIA decreases as the amount of effective impervious area decreases through disconnection. The EIA/TIA ratio does not vary very much across the subdivisions, except at Stage 2 disconnection at Tom Brown Rd. The condition of Stage 2 disconnection is 100% rooftop disconnection and 50% driveway disconnection. Tom Brown Rd is proportionately more driveway than any other impervious surface, while the other study areas have proportionately more street area. When the driveways are factored into disconnection at Tom Brown Rd, proportionately more surface is disconnected and the EIA and thus the EIA/TIA ratio decreases more than in the other scenarios.

Table 30: Subdivision Scale Volume Weighted and Disconnection Curve Numbers for WQ through 100-yr storm

Lot size (acres)	Subdivision	Volume Weighted	Existing Conditions	Stage 1	Stage 2	Stage 3
1.25” Storm						
1/3	E. Central & N. Washington	86	85	84	83	79
½	Georgian Dr.	84	82	80	79	66
1	Cardinal Dr.	85	84	83	82	78
5	Tom Brown Rd.	78	77	77	76	75
3.4” Storm						
1/3	E. Central & N. Washington	82	82	81	81	79
½	Georgian Dr.	74	73	72	71	67
1	Cardinal Dr.	81	80	80	80	78
5	Tom Brown Rd.	76	76	76	75	75
5.4” Storm						
1/3	E. Central & N. Washington	81	81	81	80	79
½	Georgian Dr.	72	71	70	70	67
1	Cardinal Dr.	80	80	79	79	78
5	Tom Brown Rd.	76	76	75	75	75
8.8” Storm						
1/3	E. Central & N. Washington	80	80	80	79	79
½	Georgian Dr.	70	69	69	67	67
1	Cardinal Dr.	79	79	79	78	78
5	Tom Brown Rd.	75	75	75	75	75

Table 31: EIA, TIA, and EIA/TIA ratio for each subdivision at the stages of disconnection

	Existing conditions			
Lot size (acres)	**Subdivision**	**% EIA**	**% TIA**	**EIA/TIA**
1/3	E. Central & N. Washington Ave.	22.9	28	0.818
1/2	Georgian Dr.	21.1	25	0.844
1	Cardinal Dr.	19.3	23	0.839
5	Tom Brown Rd.	5.1	6	0.850
			avg	0.838
	Stage 1 Disconnection			
Lot size (acres)	**Subdivision**	**% EIA**	**% TIA**	**EIA/TIA**
1/3	E. Central & N. Washington Ave.	17.8	28	0.636
1/2	Georgian Dr.	17.2	25	0.688
1	Cardinal Dr.	15.6	23	0.678
5	Tom Brown Rd.	4.1	6	0.683
			avg	0.671
	Stage 2 Disconnection			
Lot size (acres)	**Subdivision**	**% EIA**	**% TIA**	**EIA/TIA**
1/3	E. Central & N. Washington Ave.	15.3	28	0.546
1/2	Georgian Dr.	14.7	25	0.588
1	Cardinal Dr.	13.1	23	0.570
5	Tom Brown Rd.	2.3	6	0.383
			avg	0.522
	Stage 3 Disconnection			
Lot size (acres)	**Subdivision**	**% EIA**	**% TIA**	**EIA/TIA**
1/3	E. Central & N. Washington Ave.	0	28	0.000
1/2	Georgian Dr.	0	25	0.000
1	Cardinal Dr.	0	23	0.000
5	Tom Brown Rd.	0	6	0.000
			avg	0.000

Table 32: Average EIA/TIA ratio for each stage of disconnection

Stage of Disconnection	Average EIA/TIA
Existing Conditions	0.838
Stage 1	0.671
Stage 2	0.522
Stage 3	0.000

A new curve number table was constructed that includes curve numbers designated for the stages of disconnection at the water quality and 2-year storms for use in place of TR-55 Table 2-2a. The 10- and 100-year storms were not included, because as seen in the field scale subdivision analysis, the large storm curve numbers were approximately equal to the original composite curve numbers. The table was constructed using the values of EIA/TIA given in Table 32. Runoff volumes were calculated for a 1-acre area of land based on percent imperviousness, percent connected, and percent disconnected impervious area. Curve numbers were calculated for a complex of hydrologic soil groups A, B, C, and D; and high density, medium density, low density, and rural residential land uses defined by percent impervious surface cover. The curve number for A soils is 34 and is not valid at the water quality storm because it is below the runoff threshold for 1.25" of rainfall, which is at a curve number of 61. If the limitations of the SCS equations are honored, simulations using a curve number of 34 would not be accurate unless there was over three inches of rainfall. Since no runoff is calculated when a pervious curve number of 61 is used in the SCS equations for the water quality storm, 61 was used to generate curve numbers for urban areas with A type soils at this storm. Therefore, it should be noted that the water quality storm curve numbers for A soils are a conservative estimate. In actuality they may be lower because more rainfall may be able to infiltrate, however, the volume could not be quantified.

The new curve numbers were determined from the average runoff volume calculated for the range of impervious surface given for each classification of residential land use in the table. The new curve numbers are presented in Table 34 through Table 37. The original composite curve numbers from TR-55 are in Table 33. The highest curve numbers are calculated for the water quality storm and high density land use. The percent change in disconnected curve number from the composite curve number (Table 33) for each scenario is documented in Table 38 through Table 41. Hydrologic D soils see a -2% to 2% change in curve number from the published TR-55 values. A group soils, on the other hand, see a 3% to a 65% change in curve number throughout the stages of disconnection and rainfall depths. The greatest change (increase or decrease) in curve number is for A and B soils at the water quality storm.

Table 33: Residential Land Use Curve Numbers from TR-55

Residential Land Use	Lot Size (acres)	% Impervious Surface Range	Hydrologic Soil Group			
			A	B	C	D
High Density	1/8 to 1/5	>65	77	85	90	92
Medium Density	1/8 to 1/2	30-35	57	72	81	86
Low Density	1/2 to 1	20-25	51	68	79	84
Rural	1 to 2	5-20	46	65	77	82

Table 34: Curve Number Table for existing conditions (50% rooftop disconnection) at the water quality (1.25") and 2-yr (3.4") storm

Residential Land Use	Lot Size (acres)	% Impervious Surface Range	Hydrologic Soil Group			
			A	B	C	D
1.25" Storm						
High Density	1/8 to 1/5	>65	91	91	92	93
Medium Density	1/8 to 1/2	30-35	84	84	86	88
Low Density	1/2 to 1	20-25	80	80	84	86
Rural	1 to 2	5-20	71	71	80	84
3.4" Storm						
High Density	1/8 to 1/5	>65	87	87	90	92
Medium Density	1/8 to 1/2	30-35	76	76	83	86
Low Density	1/2 to 1	20-25	72	72	80	85
Rural	1 to 2	5-20	68	68	78	83

Table 35: Curve Number Table for 100% rooftop disconnection at the water quality (1.25") and 2 –yr (3.4") storm

Residential Land Use	Lot Size (acres)	% Impervious Surface Range	Hydrologic Soil Group			
			A	B	C	D
1.25" Storm						
High Density	1/8 to 1/5	>65	89	89	91	92
Medium Density	1/8 to 1/2	30-35	82	82	85	87
Low Density	1/2 to 1	20-25	79	79	83	86
Rural	1 to 2	5-20	75	75	79	83

3.4" Storm						
High Density	1/8 to 1/5	>65	85	85	89	92
Medium Density	1/8 to 1/2	30-35	75	75	82	86
Low Density	1/2 to 1	20-25	71	71	80	84
Rural	1 to 2	5-20	67	67	77	82

Table 36: Curve Number Table for 100% rooftop & 50% driveway disconnection at the water quality (1.25") and 2-yr (3.4") storm

Residential Land Use	Lot Size (acres)	% Impervious Surface Range	Hydrologic Soil Group			
			A	B	C	D
1.25" Storm						
High Density	1/8 to 1/5	>65	87	87	90	92
Medium Density	1/8 to 1/2	30-35	80	80	85	87
Low Density	1/2 to 1	20-25	77	77	82	85
Rural	1 to 2	5-20	73	73	81	83
3.4" Storm						
High Density	1/8 to 1/5	>65	84	84	89	91
Medium Density	1/8 to 1/2	30-35	73	73	82	86
Low Density	1/2 to 1	20-25	70	70	79	84
Rural	1 to 2	5-20	66	66	77	82

Table 37: Curve Number Table for 100% impervious surface disconnection at the water quality (1.25”) and 2-yr (3.4”) storm

Residential Land Use	Lot Size (acres)	% Impervious Surface Range	Hydrologic Soil Group			
			A	B	C	D
1.25” Storm						
High Density	1/8 to 1/5	>65	79	79	87	90
Medium Density	1/8 to 1/2	30-35	68	68	79	84
Low Density	1/2 to 1	20-25	66	66	78	83
Rural	1 to 2	5-20	64	64	76	82
3.4” Storm						
High Density	1/8 to 1/5	>65	81	81	87	90
Medium Density	1/8 to 1/2	30-35	69	69	80	84
Low Density	1/2 to 1	20-25	66	66	78	83
Rural	1 to 2	5-20	64	64	76	82

Table 38: Percent Change in curve number from composite curve number for existing conditions (50% rooftop disconnection) at the water quality (1.25") and 2-yr (3.4”) storm

Residential Land Use	Lot Size (acres)	% Impervious Surface Range	% Change in CN			
			A	B	C	D
1.25” Storm						
High Density	1/8 to 1/5	>65	18%	7%	2%	1%
Medium Density	1/8 to 1/2	30-35	47%	17%	6%	2%
Low Density	1/2 to 1	20-25	59%	19%	6%	2%
Rural	1 to 2	5-20	65%	17%	4%	2%

3.4” Storm						
High Density	1/8 to 1/5	>65	13%	2%	0%	0%
Medium Density	1/8 to 1/2	30-35	33%	6%	2%	1%
Low Density	1/2 to 1	20-25	41%	6%	2%	1%
Rural	1 to 2	5-20	47%	4%	1%	1%

Table 39: Percent Change in curve number from TR-55 for Stage 1 disconnection at the water quality (1.25") and 2-yr (3.4”) storm

Residential Land Use	Lot Size (acres)	% Impervious Surface Range	% Change in CN			
			A	B	C	D
1.25” Storm						
High Density	1/8 to 1/5	>65	16%	5%	1%	0%
Medium Density	1/8 to 1/2	30-35	44%	14%	5%	1%
Low Density	1/2 to 1	20-25	55%	16%	5%	2%
Rural	1 to 2	5-20	63%	15%	3%	1%
3.4” Storm						
High Density	1/8 to 1/5	>65	10%	0%	-1%	-1%
Medium Density	1/8 to 1/2	30-35	31%	4%	2%	0%
Low Density	1/2 to 1	20-25	39%	4%	1%	0%
Rural	1 to 2	5-20	45%	3%	0%	1%

Table 40: Percent Change in curve number from TR-55 for stage 2 disconnection at the water quality (1.25") and 2-yr (3.4”) storm

Residential Land Use	Lot Size (acres)	% Impervious Surface Range	% Change in CN			
			A	B	C	D
1.25” Storm						
High Density	1/8 to 1/5	>65	13%	2%	0%	0%
Medium Density	1/8 to 1/2	30-35	40%	11%	5%	1%
Low Density	1/2 to 1	20-25	51%	13%	4%	1%
Rural	1 to 2	5-20	59%	12%	5%	1%
3.4” Storm						
High Density	1/8 to 1/5	>65	9%	-2%	-1%	-1%
Medium Density	1/8 to 1/2	30-35	29%	2%	1%	0%
Low Density	1/2 to 1	20-25	37%	3%	1%	0%
Rural	1 to 2	5-20	44%	2%	0%	0%

Table 41: Percent Change in curve number from TR-55 for stage 3 disconnection at the water quality (1.25") and 2-yr (3.4”) storm

Residential Land Use	Lot Size (acres)	% Impervious Surface Range	% Change in CN			
			A	B	C	D
1.25” Storm						
High Density	1/8 to 1/5	>65	3%	-7%	-3%	-2%
Medium Density	1/8 to 1/2	30-35	19%	-6%	-2%	-2%
Low Density	1/2 to 1	20-25	29%	-3%	-1%	-1%
Rural	1 to 2	5-20	39%	-2%	-1%	0%

3.4" Storm						
High Density	1/8 to 1/5	>65	5%	-5%	-3%	-2%
Medium Density	1/8 to 1/2	30-35	21%	-4%	-1%	-2%
Low Density	1/2 to 1	20-25	30%	-3%	-1%	-1%
Rural	1 to 2	5-20	39%	-2%	-1%	0%

HEC-HMS Calibration

The new curve number tables were used to analyze disconnection at the watershed scale with a calibrated HEC-HMS watershed model. Streamflow data from two gauge sites on the West Branch of the Pompeston Creek were coupled with rainfall data to calibrate the curve number, initial abstraction, Snyder's Lag, and peaking coefficient in the model. Water depths at the gauge sites were monitored for three months with a pressure transducer installed at each site. Cross sectional flow was measured and used to develop a rating curve for each gauge location (Figure 16 and Figure 17). The r^2 value for the rating curve at Site 1 is 0.80. Ten cross sectional flows were measured and were in the range of 0.19 to 18.6 ft^3/s. The r^2 value for the rating curve at Site 2 is 0.91. Eight flow measurements were taken in the range of 1.55 to 59.63 ft^3/s.

The equations derived from the rating curve were used to convert the water depths recorded by the transducers to streamflow in cfs. The streamflow time series was in 30 minute intervals. Hydrographs were then plotted for select storm events. Rainfall was recorded by a tipping bucket gauge installed near New Albany Rd. Recorded storms and rainfall depth are presented in Table 42, and the recorded storm distributions are in Figure 18, Figure 19, and Figure 20. Hydrographs and corresponding rainfall depths are shown for the 10/11/05, 10/24/05, and 11/16/05 storm events at each gauge site in Figure 21 through

Figure 26. The three storms chosen for calibration were all less than the water quality storm. These storms were also ideal for the calibration because they were complete sets of data that could be paired with complete sets of runoff data. Both the 10-11-05 and 10-24-05 storms had rainfall events approximately two days earlier. There was no rain for seven days prior to the 11-16-05 storm. A total of ten storms were recorded over the sample period. Eight of the ten rainfall events were less than 1.25".

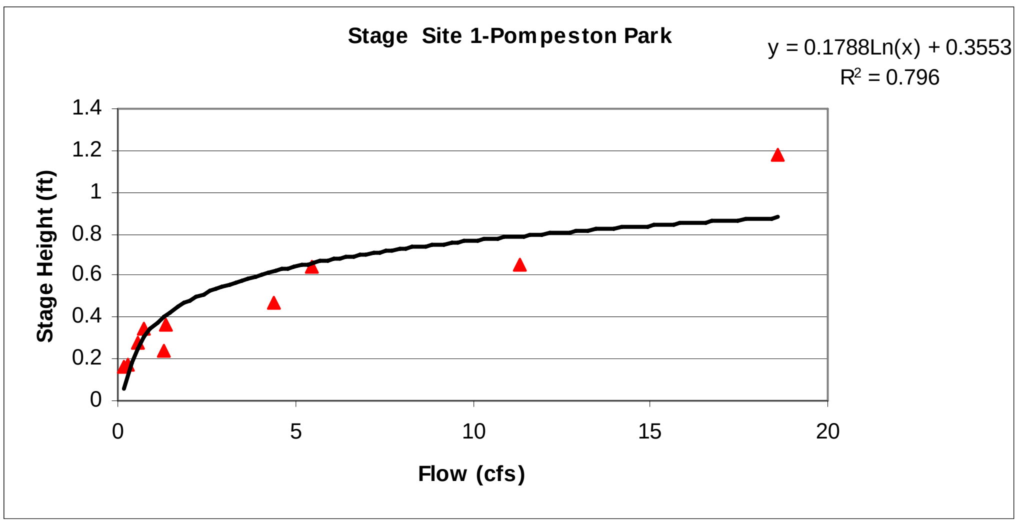

Figure 16: Rating Curve at Gauge 1, Pompeston Park

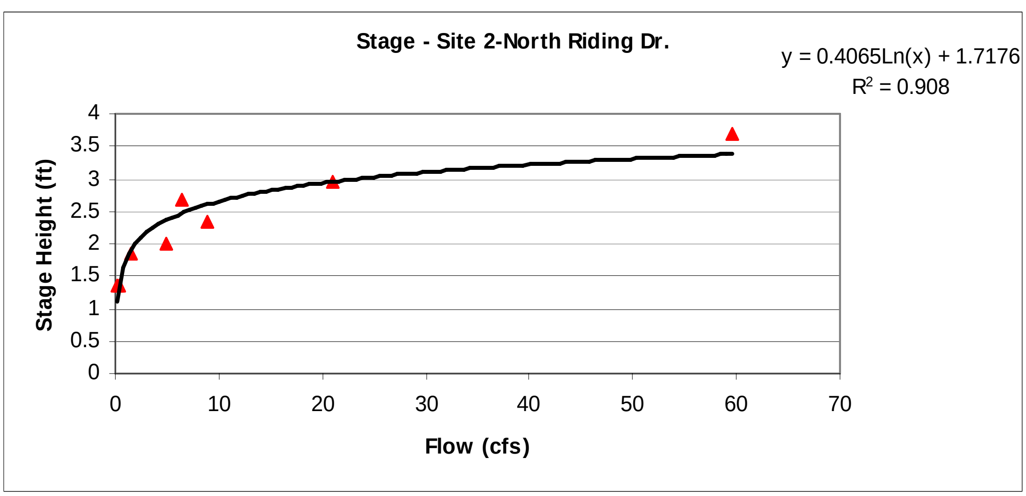

Figure 17: Rating Curve for Site 2, North Riding Dr.

Table 42: Recorded storms

Date	Total Rainfall depth (in.)
9/26/2005	0.14
9/29/2005	0.07
10/8/2005	2
10/9/2005	1.45
10/11/2005	0.65
10/21/2005	0.16
10/22/2005	0.72
10/24/2005	1.2
11/9/2005	0.15
11/16/2005	0.76
11/21/2005	1.1
11/29/2005	0.76

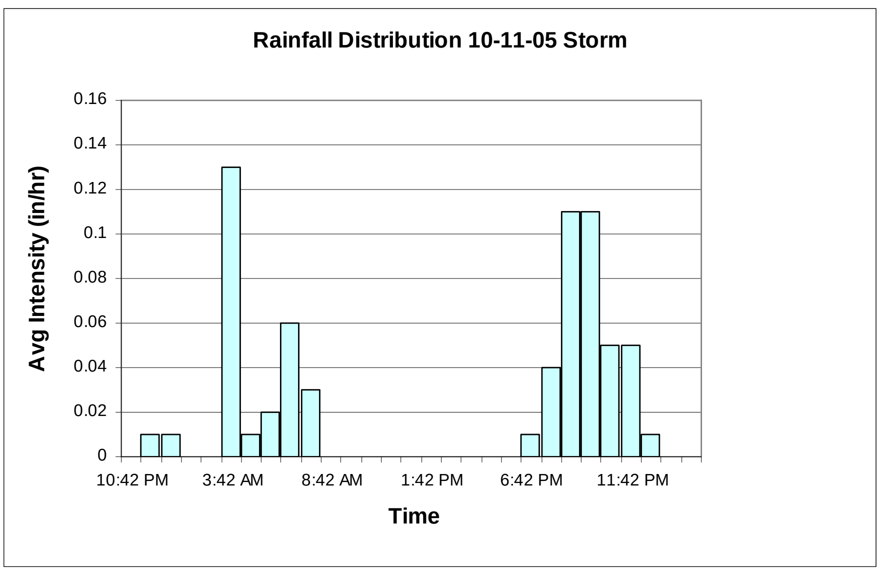

Figure 18: Rainfall distribution of 10-11-05 storm

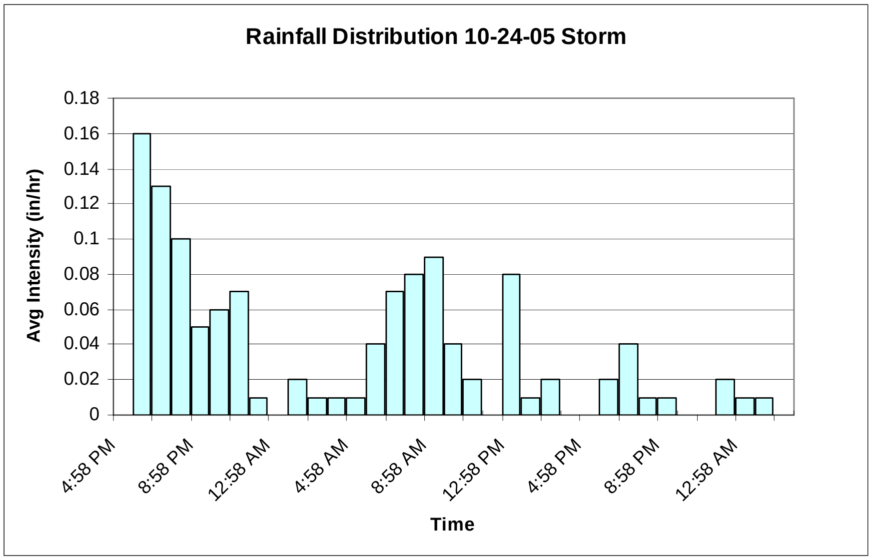

Figure 19: Rainfall distribution of 10-24-05 storm

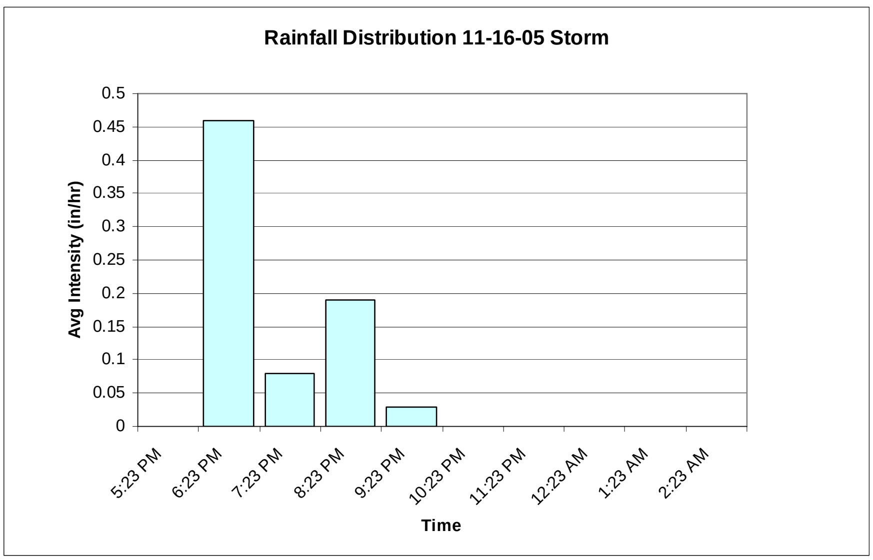

Figure 20: Rainfall distribution of 11-16-05 storm

An optimization function in the model software package was used to optimize parameters based on observed flow data. Observed streamflow data was entered into HEC-HMS as a discharge gauge at 30 minute intervals. Gauge 1 monitored flow from Basins 8, 9, and 13. Gauge 2 was downstream of Gauge 1 at the outlet of Basin 12. Baseflow was separated from the hydrograph using the flow observed during dry periods. After baseflow was separated, the total observed discharge volume was calculated for each storm. In order to find the discharge volume for area 2, the total volume from Gauge 1 was subtracted from the total volume from Gauge 2, then baseflow was separated. Basin curve numbers were adjusted until the calculated discharge volume matched the observed volume. The initial abstraction was held to the relationship I_a=0.2S where S = (1000/CN)-10. The calibrated parameters and model results for each storm are shown in Table 43, Table 44, and Table 45. The discharge volumes in the tables include baseflow.

Table 43: 10-11-05 Storm Calibration, P=0.65"

Drainage Area	CN	Ia (in)	Calculated volume (ac-ft)	Observed Volume (ac-ft)
Area 1	85	0.353	4.922	4.957
Area 2	89	0.247	7.790	7.833
Total			12.700	12.790

Table 44: 10-24-05 Storm Calibration, P=1.20"

Drainage Area	CN	Ia (in)	Calculated volume (ac-ft)	Observed Volume (ac-ft)
Area 1	81	0.47	17.575	17.998
Area 2	90	0.22	26.957	26.358
Total			44.526	44.356

Table 45: 11-16-05 Storm Calibration, P=0.76"

Drainage Area	CN	Ia (in)	Calculated volume (ac-ft)	Observed Volume (ac-ft)
Area 1	86	0.326	8.516	8.454
Area 2	93	0.15	17.477	18.296
Total			25.985	26.75

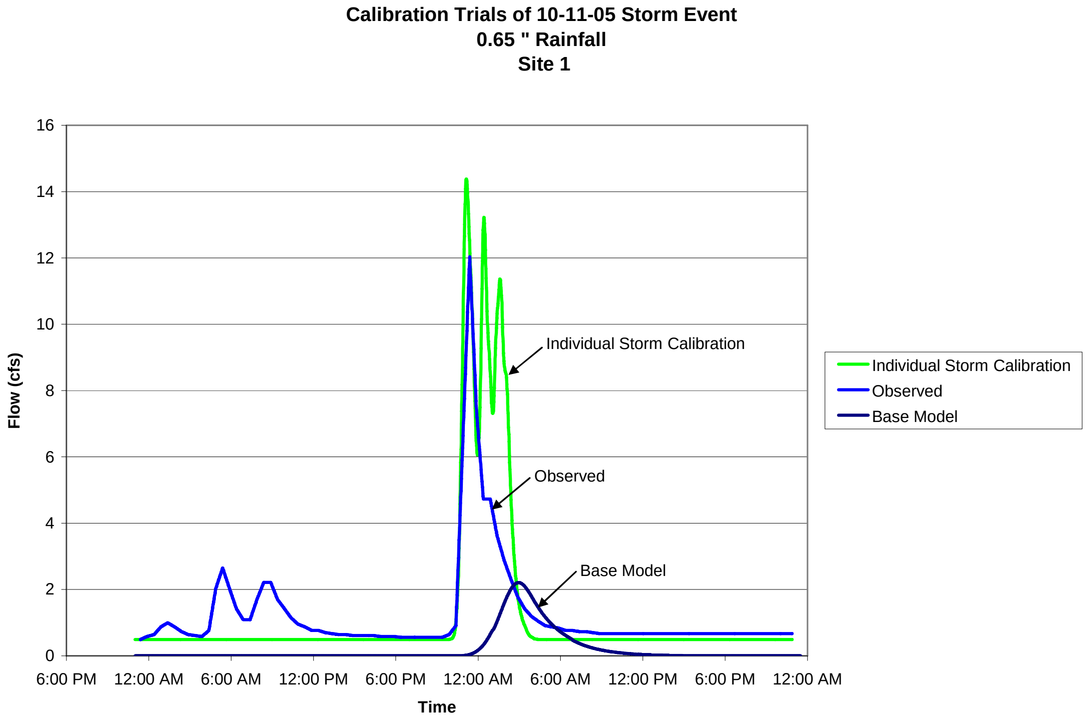

Figure 21: Site 1 Calibration Oct 11, 2005 Storm

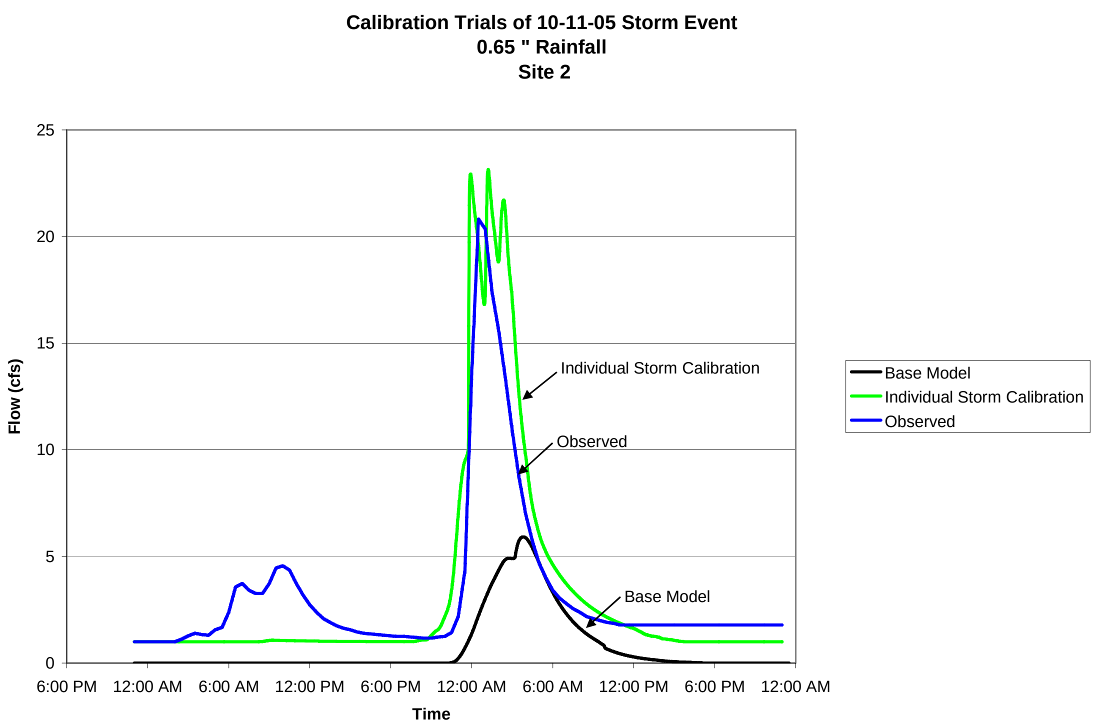

Figure 22: Site 2 Calibration Oct 11, 2005 Storm

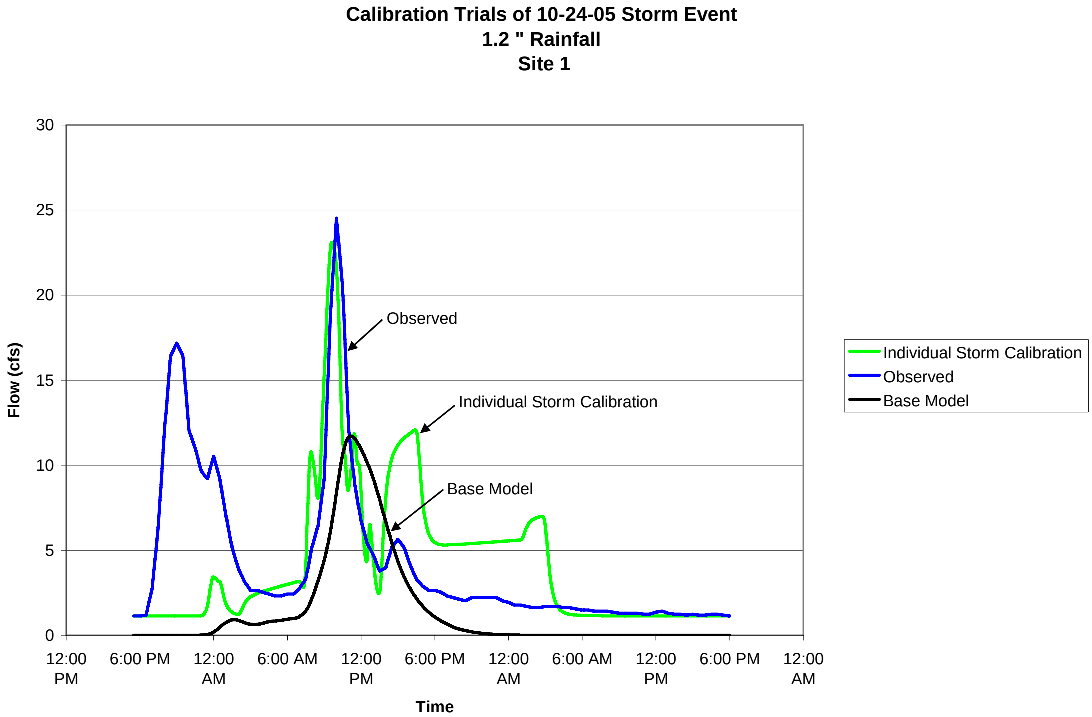

Figure 23: Site 1 Calibration Oct 24, 2005 Storm

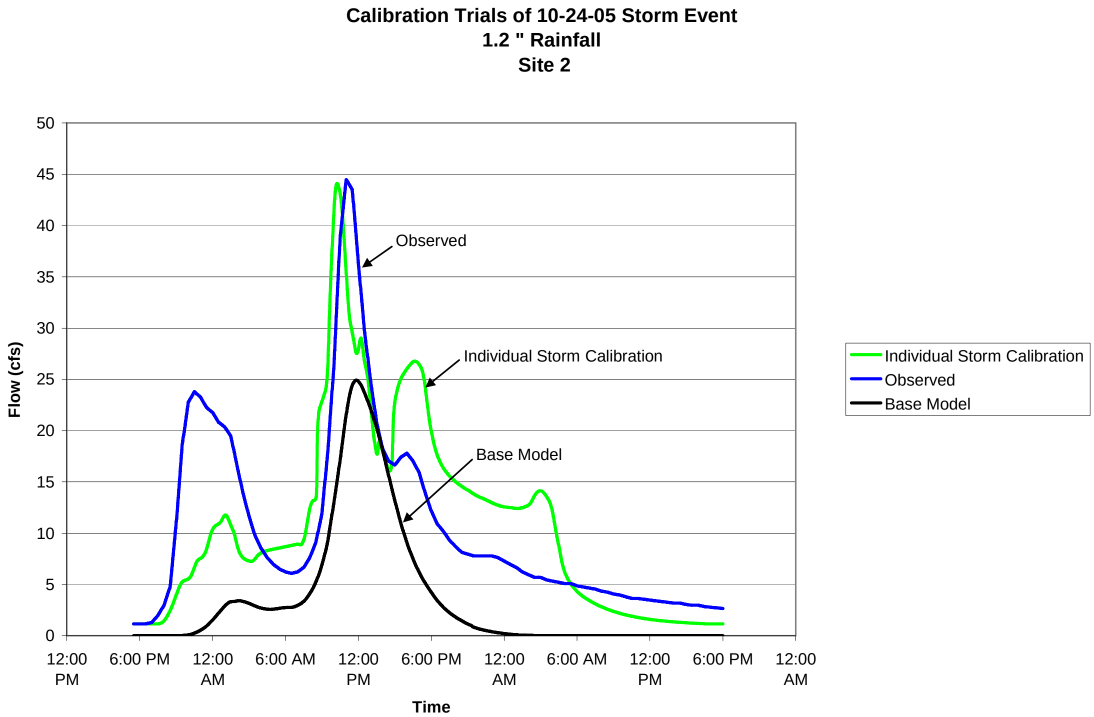

Figure 24: Site 2 Calibration Oct 24, 2005 Storm

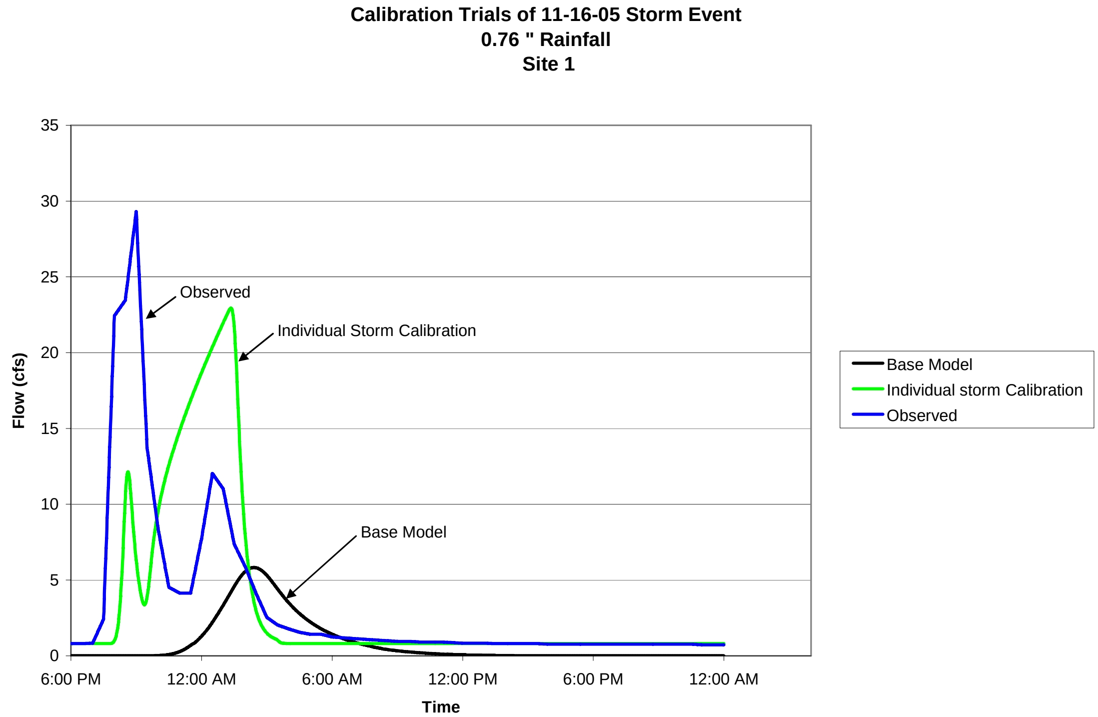

Figure 25: Site 1 Calibration Nov 16, 2005 Storm

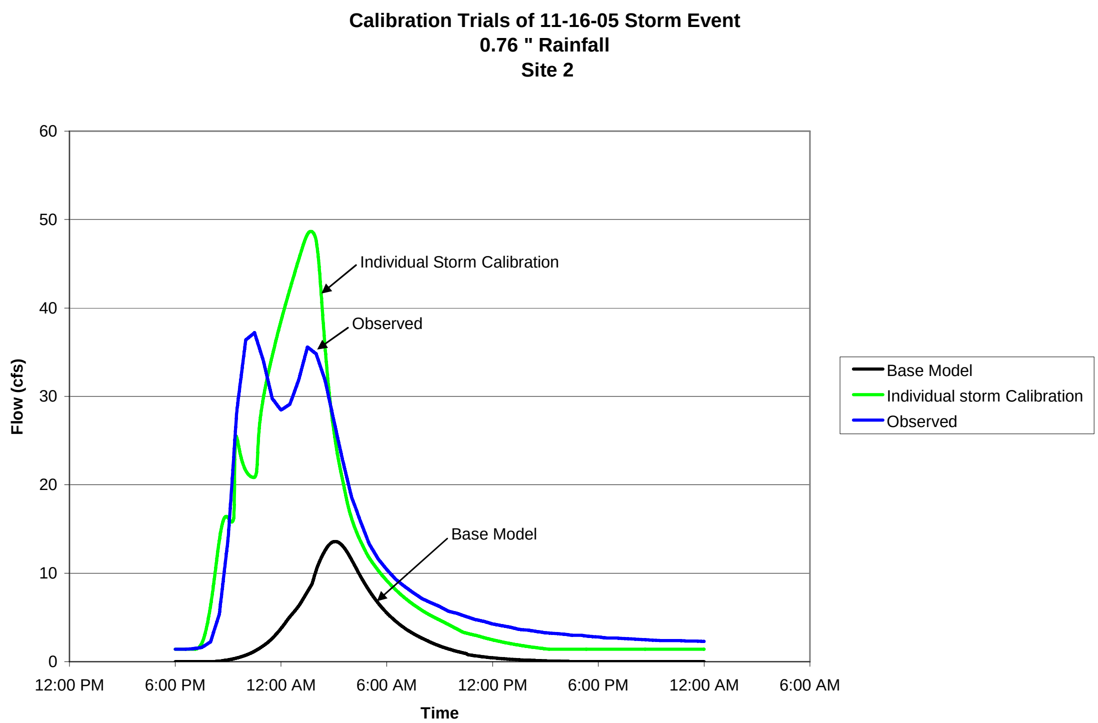

Figure 26: Site 2 Calibration Nov 16, 2005 Storm

The average curve number from the optimization of the three observed storms was used as the calibrated curve number. The calibrated curve number for Basins 8, 9, and 13 is 84, and for Basin 12 is 91. The model was validated using these storms in Table 48. Figure 27 through Figure 32 illustrate the comparison of the predicted hydrographs using the calibrated curve number with the observed hydrographs. The correlation between the observed and predicted hydrographs at Sites 1 and 2 is presented in Table 46 and Table 47.

The correlation coefficient of the data set shows the strength of the linear relationship between the measured and predicted values. The governing equation for the calculation of the correlation between two data sets is:

$$\text{Correl (X,Y)} = \frac{\sum (x-\chi)(y-\gamma)}{\sqrt{\sum (x-\chi)^2 \sum (y-\gamma)^2}}$$

x, y = sample value
χ, γ = sample mean

For the type of data sets in this study, this test provides sufficient results to support the validation of the small storm HEC-HMS model created for the Pompeston Creek Watershed. The results of these tests may be viewed in Table 46 and Table 47 below. The correlation results suggest that the model is better at predicting streamflow at Site 2 because the values are closer to 1, which represents a perfect fit, or 100%, of the predicted data to observed data. However, the correlation at Site 1 does suggest that there is a 38% to an 82% match of the data to the model predictions throughout the storm events, which can be considered a reasonable simulation.

Table 46: Site 1 Observed vs. Predicted Flow Correlation

Storm	Correlation
10-11-05	0.822
10-24-05	0.548
11-16-05	0.384

Table 47: Site 2 Observed vs. Predicted Flow Correlation

Storm	Correlation
10-11-05	0.927
10-24-05	0.791
11-16-05	0.903

Table 48: CN Validation for 3 Storms

Discharge Volume (ac-ft)	10-11-05 storm 0.65”		10-24-05 storm 1.20”		11-16-05 storm 0.76”	
	Obs	Calc	Obs	Calc	Obs	Calc
Site 1	4.96	4.26	18.00	22.85	8.45	6.50
Site 2	7.83	10.20	23.36	29.48	18.3	13.35
Total	12.79	14.46	44.36	52.33	26.75	19.85

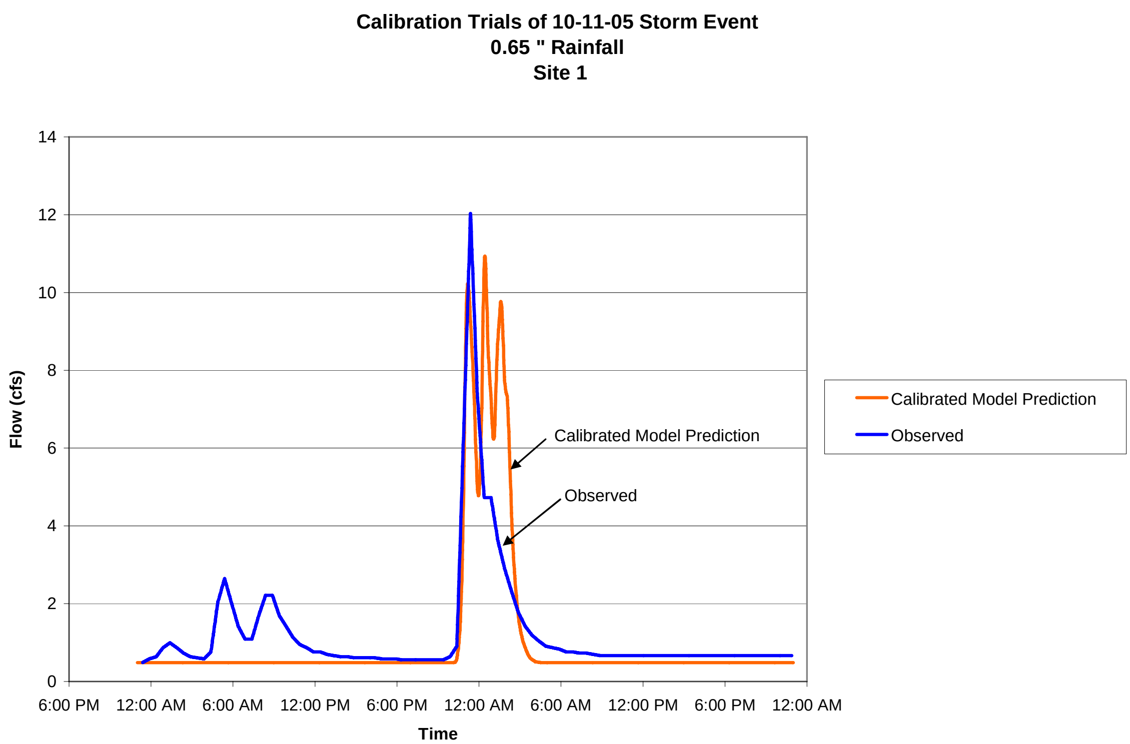

Figure 27: Site 1 Calibration Oct 11, 2005 Storm

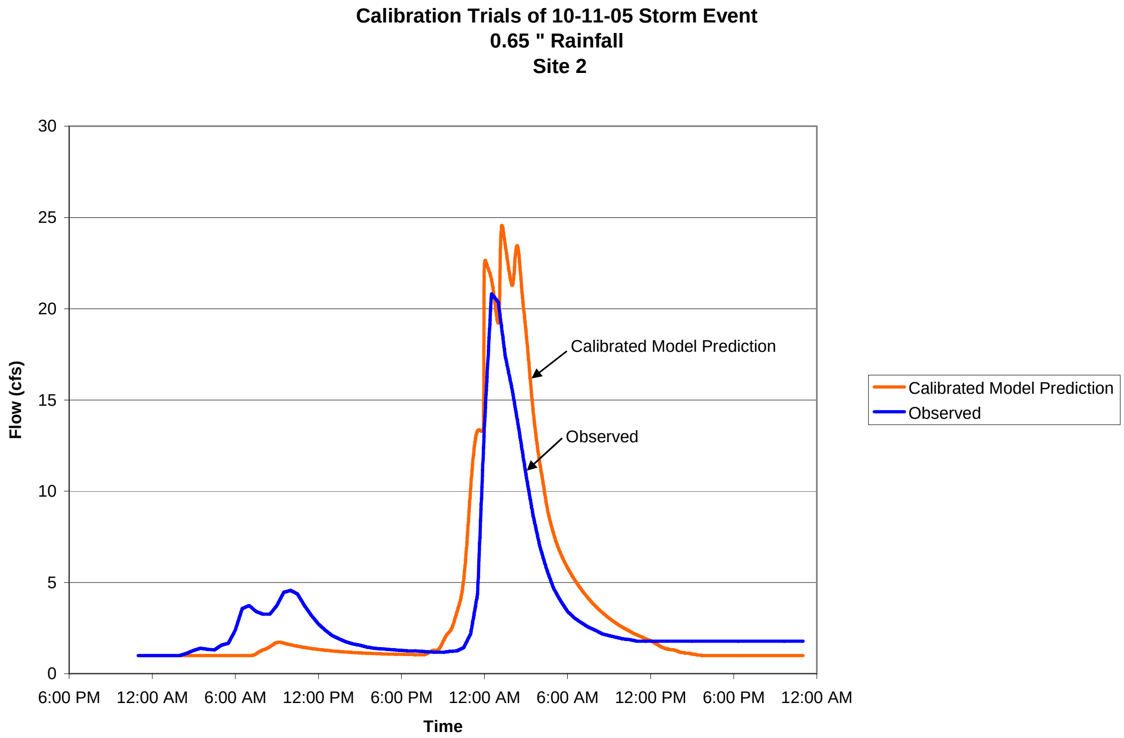

Figure 28: Site 2 Calibration Oct 11, 2005 Storm

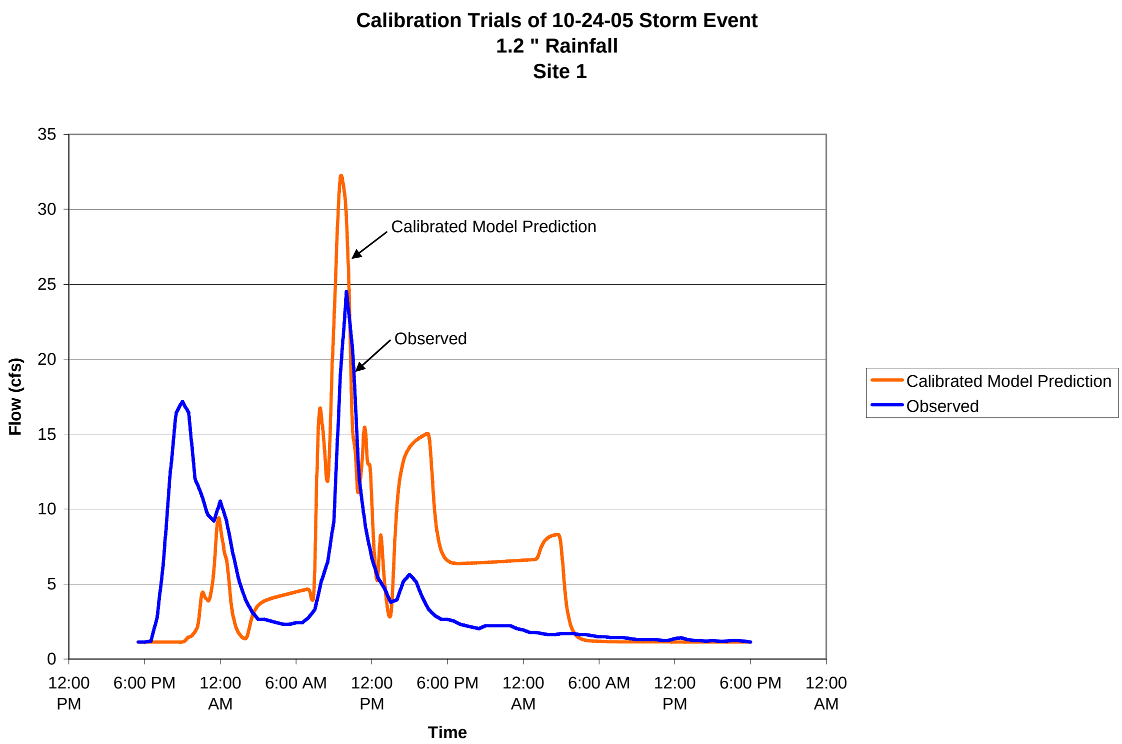

Figure 29: Site 1 Calibration Oct 24, 2005 Storm

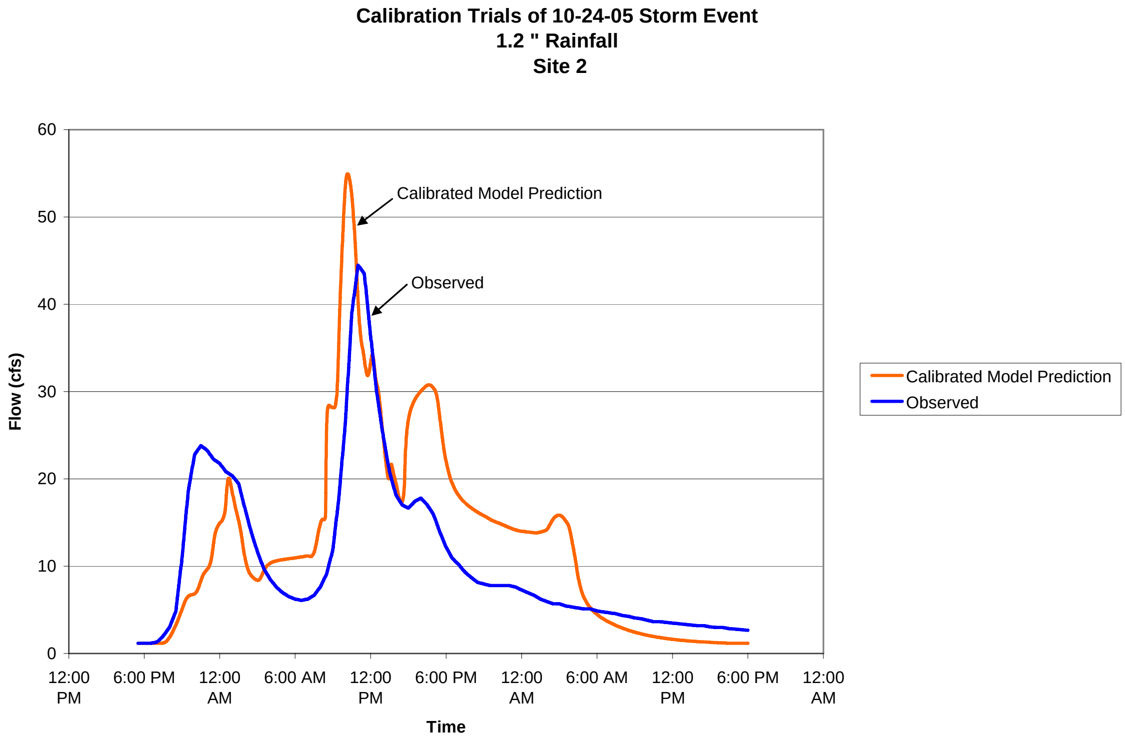

Figure 30: Site 2 Calibration Oct 24, 2005 Storm

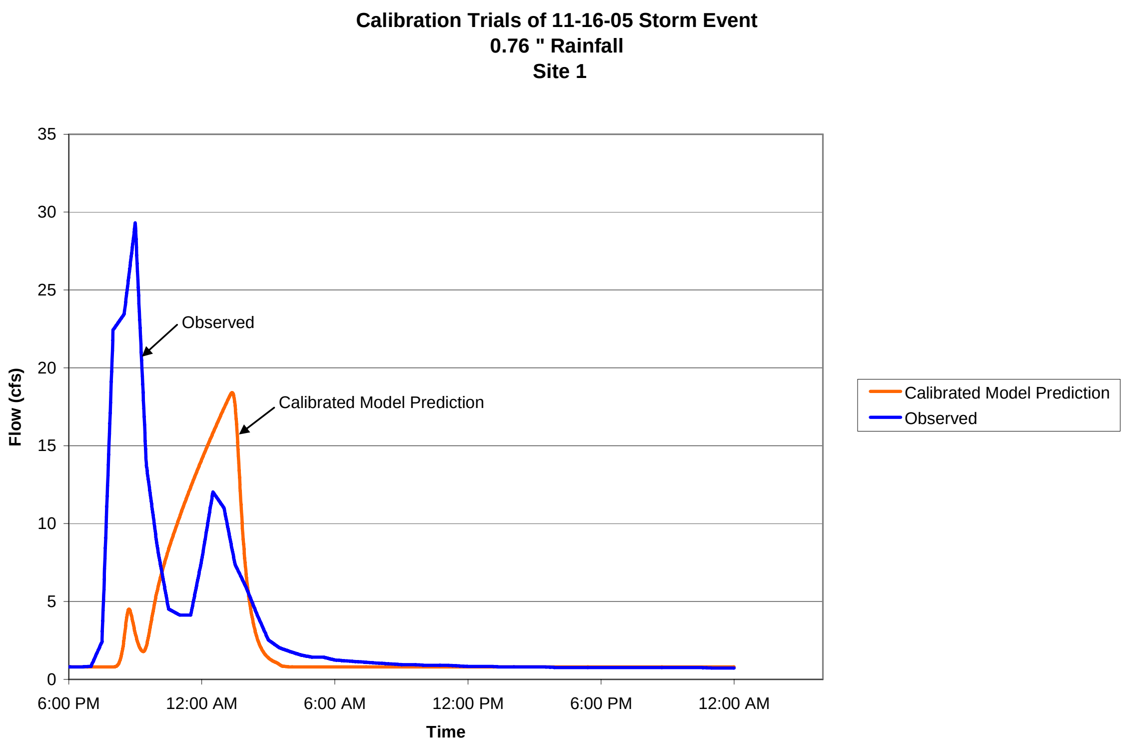

Figure 31: Site 1 Calibration Nov 16, 2005 Storm

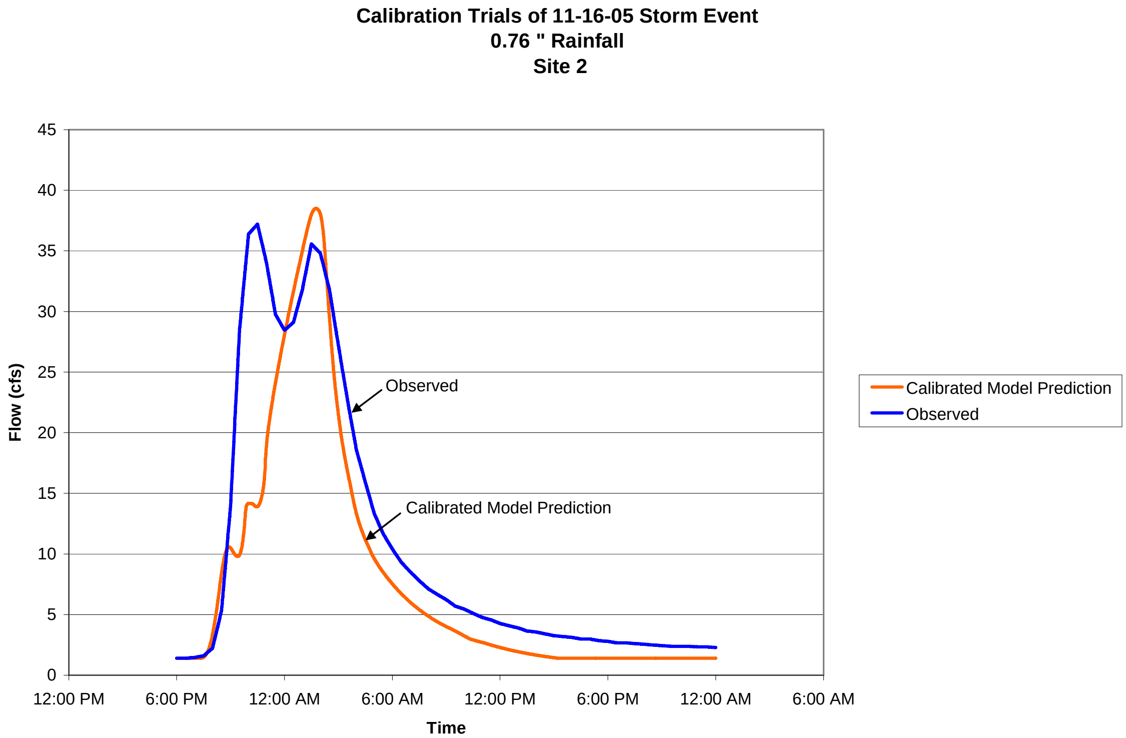

Figure 32: Site 2 Calibration Nov 16, 2005 Storm

<u>Watershed Scale Disconnection</u>

In order to examine the effects of disconnection at the watershed scale, the new residential curve numbers for the stages of disconnection presented in Table 34 through Table 37 were used to simulate small storm runoff in the Pompeston Creek Watershed. Model simulations were run to represent watershed-wide application of each disconnection scenario. Curve numbers were assigned to residential land use polygons based on soil, density, and stage of disconnection. Since the model requires the input of a single curve number for each basin, a composite curve number was calculated for each basin by taking the weighted average of the curve numbers assigned to each polygon from the new curve number tables for the basin areas (Table 49).

Table 49: Basin drainage area

Basin	Area (acres)
1	252
2	625
3	610
4	625
5	132
6	355
7	274
8	261
9	346
10	651
11	124
12	699
13	254

The series of new composite curve numbers for each basin and stage of disconnection is given in Table 50. Table 51 calculates the change from the composite curve number used in the base model to the new curve number. The new initial abstraction for each basin was calculated based on the new basin curve number. Simulations were performed for existing conditions, stage 1, stage 2, and stage 3 disconnection. Runoff volumes are summarized in Table 52. The simulated volumes were compared to the predicted runoff using the base model composite curve number (Table 53). The runoff hydrograph and total runoff volume were calculated at each gauge site. The change in curve number and change in simulated volume from the existing conditions simulation are presented in Table 54 and Table 55.

A volume analysis was performed to locate areas throughout the watershed that have the potential to generate the most runoff. The runoff volume generated from each land use-soil complex polygon during the water quality storm was calculated. The curve numbers derived for existing conditions were applied to each residential polygon. For comparison, the land use breakdown by percent area is given for all basins in Table 58, and the residential land use breakdown is given in Table 59. The total runoff generated from the total urban area, wetlands, and the total residential area for each basin is presented in Table 56. The percent runoff from the residential areas was further broken down into contribution from high density, medium density, low density, and rural residential land use in Table 57. The volume analysis tables were compared with the predicted water quality storm runoff for the calibrated basins in Table 60.

Table 50: Curve numbers used in HEC-HMS to calculate basin runoff volume for each stage of disconnection

Basin	Base Model CN	Existing Conditions	Stage 1	Stage 2	Stage 3	Calibrated CN
1	72.9	80.9	76.2	75.7	73.1	
2	76.9	81.5	78.9	78.4	75.6	
3	73.7	80.6	79.0	77.9	71.5	
4	78.3	84.6	82.8	81.7	75.1	
5	76.5	84.9	82.2	81.2	74.6	
6	82.7	86.3	84.8	84.4	79.1	
7	82.7	86.7	85.2	84.9	80.2	
8	79.4	84.7	83.1	82.2	75.8	84
9	81.1	86.5	84.8	83.9	77.3	84
10	83.3	85.2	84.5	84.2	82.3	
11	66.8	75.2	70.7	70.3	67.3	
12	84.7	86.6	86.0	85.5	83.6	91
13	81.1	84.4	83.5	82.8	79.4	84

Table 51: Percent change in curve number from base model composite curve number

Basin	Existing Conditions	Stage 1	Stage 2	Stage 3	Calibrated CN
1	11.0%	4.5%	3.9%	0.4%	
2	5.9%	2.6%	2.0%	-1.7%	
3	9.4%	7.2%	5.6%	-3.0%	
4	8.0%	5.8%	4.4%	-4.0%	
5	11.0%	7.4%	6.2%	-2.4%	
6	4.3%	2.5%	2.0%	-4.4%	
7	4.8%	3.0%	2.7%	-3.0%	
8	6.7%	4.7%	3.6%	-4.5%	5.9%
9	6.6%	4.5%	3.4%	-4.8%	3.5%
10	2.2%	1.4%	1.0%	-1.2%	
11	12.5%	5.8%	5.1%	0.8%	
12	2.3%	1.6%	1.0%	-1.2%	7.5%
13	4.1%	2.9%	2.0%	-2.1%	3.5%

Basin	Base Model CN	Existing Conditions	Stage 1	Stage 2	Stage 3	Calibrated Model
1	1.43	3.87	2.12	1.95	1.29	
2	6.62	10.63	7.96	7.39	4.90	
3	4.63	9.49	7.80	6.68	2.32	
4	8.96	14.93	12.59	10.96	4.64	
5	1.60	3.32	2.47	2.21	0.90	
6	7.78	10.17	8.74	8.40	4.56	
7	6.13	8.15	7.01	6.78	4.08	
8	3.49	6.39	5.34	4.90	2.13	5.936
9	5.77	10.20	8.52	7.83	3.50	7.871
10	13.74	16.68	15.49	14.96	12.29	
11	0.13	0.93	0.40	0.36	0.16	
12	17.06	20.63	19.64	18.46	15.22	31.49
13	4.22	6.00	5.51	5.01	3.39	5.761

Table 52: Basin runoff volume(ac-ft) from the water quality storm for the stages of disconnection

Table 53: Percent change in disconnected volume from base model simulated at the water quality storm

Basin	Existing Conditions	Stage 1	Stage 2	Stage 3	Calibrated Model
1	170.5%	47.7%	36.0%	-10.1%	
2	60.5%	20.2%	11.6%	-26.0%	
3	104.7%	68.3%	44.2%	-49.9%	
4	66.7%	40.6%	22.4%	-48.2%	
5	107.1%	54.2%	37.7%	-44.0%	
6	30.8%	12.4%	8.1%	-41.3%	
7	33.1%	14.4%	10.6%	-33.3%	
8	83.3%	53.2%	40.5%	-39.0%	70.2%
9	76.7%	47.6%	35.7%	-39.4%	36.4%
10	21.4%	12.7%	8.9%	-10.6%	
11	602.4%	203.3%	171.4%	19.7%	
12	20.9%	15.1%	8.2%	-10.8%	84.6%
13	41.9%	30.4%	18.5%	-19.9%	36.4%

Table 54: Percent Change in basin curve number from existing conditions

Basin	Stage 1	Stage 2	Stage 3	Calibrated CN
1	-5.8%	-6.4%	-9.6%	
2	-3.1%	-3.7%	-7.2%	
3	-2.0%	-3.4%	-11.3%	
4	-2.1%	-3.4%	-11.1%	
5	-3.2%	-4.4%	-12.1%	
6	-1.8%	-2.2%	-8.4%	
7	-1.7%	-2.0%	-7.4%	
8	-1.9%	-2.9%	-10.5%	-0.8%
9	-2.0%	-3.0%	-10.7%	-2.9%
10	-0.8%	-1.2%	-3.4%	
11	-6.0%	-6.6%	-10.5%	
12	-0.7%	-1.3%	-3.4%	5.1%
13	-1.1%	-1.9%	-6.0%	-0.5%

Basin	Stage 1	Stage 2	Stage 3	Calibrated CN
1	-45.4%	-49.7%	-66.8%	
2	-25.1%	-30.5%	-53.9%	
3	-17.8%	-29.5%	-75.5%	
4	-15.7%	-26.6%	-68.9%	
5	-25.5%	-33.5%	-73.0%	
6	-14.1%	-17.4%	-55.1%	
7	-14.0%	-16.9%	-49.9%	
8	-16.4%	-23.3%	-66.7%	-7.1%
9	-16.5%	-23.2%	-65.7%	-22.8%
10	-7.1%	-10.3%	-26.3%	
11	-56.8%	-61.4%	-83.0%	
12	-4.8%	-10.5%	-26.2%	52.6%
13	-8.1%	-16.5%	-43.5%	-3.9%

Table 55: Percent change in volume from simulation of existing conditions at the water quality storm

Table 56: Percent of total runoff volume contributed by each land use during the water quality storm

Basin	Total Area (acres)	Total Runoff Volume (ac-ft)	Urban	Wetlands	Residential	Other Urban
1	252	7.81	36%	58%	17%	19%
2	625	17.49	63%	31%	22%	42%
3	610	13.12	95%	3%	57%	38%
4	625	17.17	83%	16%	79%	4%
5	132	3.98	95%	4%	68%	28%
6	355	10.62	98%	2%	89%	9%
7	274	8.81	88%	12%	76%	11%
8	261	6.94	92%	2%	72%	20%
9	346	10.91	99%	1%	59%	40%
10	651	21.48	43%	47%	35%	7%
11	124	2.14	87%	0%	52%	34%
12	699	24.75	59%	39%	37%	22%
13	254	7.09	72%	28%	61%	11%

Table 57: Percent runoff contribution from residential land uses during the water quality storm

Basin	High Density	Medium Density	Low Density	Rural	Total Residential
1	0.3%	15.2%	1.5%	0.1%	17%
2	0.9%	20.0%	0.4%	0.3%	22%
3	3.6%	42.0%	9.5%	1.8%	57%
4	0.6%	58.2%	16.0%	4.7%	79%
5	1.3%	66.5%	0.0%	0.0%	68%
6	3.4%	85.4%	0.0%	0.1%	89%
7	0.0%	72.6%	3.5%	0.0%	76%
8	0.3%	56.7%	14.9%	0.3%	72%
9	0.5%	55.2%	3.1%	0.2%	59%
10	0.2%	11.8%	11.4%	11.9%	35%
11	0.0%	52.1%	0.0%	0.0%	52%
12	0.0%	2.2%	31.6%	3.1%	37%
13	0.0%	12.4%	42.0%	6.6%	61%

Table 58: Percent Basin Land Use by Area

Basin	Total Area (acres)	Urban	Wetlands	Residential	Other urban
1	252	54%	21%	23%	31%
2	625	66%	10%	25%	42%
3	610	95%	1%	56%	39%
4	625	92%	5%	85%	7%
5	132	93%	1%	73%	20%
6	355	99%	1%	86%	13%
7	274	93%	4%	80%	13%
8	261	96%	1%	75%	21%
9	346	98%	0%	70%	28%
10	651	62%	18%	54%	8%
11	124	89%	0%	32%	57%
12	699	76%	16%	54%	22%
13	254	88%	9%	77%	11%

Table 59: Percent Residential Land Use by Area

Basin	% High Density	% Medium Density	% Low Density	% Rural	% Total Residential
1	1.1%	18.5%	2.9%	0.4%	23%
2	1.0%	22.1%	0.6%	0.8%	25%
3	3.1%	35.9%	12.5%	4.7%	56%
4	0.4%	57.2%	18.0%	9.7%	85%
5	1.3%	71.5%	0.0%	0.0%	73%
6	2.5%	83.1%	0.0%	0.4%	86%
7	0.0%	75.4%	4.1%	0.1%	80%
8	0.3%	53.4%	20.3%	0.8%	75%
9	0.6%	63.0%	5.8%	0.4%	70%
10	0.2%	12.5%	16.6%	24.8%	54%
11	0.0%	31.9%	0.0%	0.0%	32%
12	0.0%	2.6%	44.7%	7.1%	54%
13	0.0%	11.2%	53.0%	12.6%	77%

Table 60: Volume sum of urban area runoff compared with HEC-HMS modeled volume

Basin	Urban Area Weighted Volume using Existing Conditions CN (ac-ft)	Modeled Volume in HEC-HMS (ac-ft)	Difference (ac-ft)
8	6.37	5.94	0.43
9	10.81	7.87	2.94
12	14.53	31.49	-16.96
13	5.09	5.76	0.67

DISCUSSION

Runoff volume can be calculated in two ways using the SCS equations. The first is the "composite curve number method" and requires the calculation of a composite curve number which can then be used to determine the volume of runoff using the SCS equations for a design storm event. The second is the "volume weighted method" which requires the calculation of runoff volumes for the impervious and pervious portions of the site separately and then summing these runoff volumes to generate the total runoff from the site. Both methods were applied to four residential areas in the Pompeston Creek Watershed. For each of these areas, runoff calculations were performed using both methods on the lot scale and on the subdivision scale. Both methods tend to produce similar results for the larger design storms but can yield dramatically different runoff volumes for the small water quality storm and the 2-year design storm.

Comparison of the Two Methods at the Lot Scale and Subdivision Scale

For the 10 and 100-year design storms, the runoff volumes that were predicted using the composite curve number method closely match the runoff volumes calculated by the volume weighted method. At the lot scale, percent differences were virtually zero for the 100-year storm and ranged from 1% to 5% for the 10-year storm. Runoff volumes from the small storms (the water quality storm and the 2-year design storm) were much lower using the composite curve number method as opposed to the volume weighted method. At the lot scale percent difference ranged from 2% to 19% for the 2-year design storm and from 39% to 1,300% for the water quality storm.

Since the composite curve number method combines the impervious surface with the pervious surface to generate a single curve number, the runoff that is calculated using this single composite curve number is often much less than the runoff from the impervious surface alone. The water quality storm (1.25 inches of rain over two hours) on an impervious surface generates approximately 1.25 inches of runoff from that surface. In most cases, the impervious surfaces are directly connected to the stream so there are no opportunities for this runoff to flow over a pervious surface and infiltrate into the ground. Therefore, the volume calculated by the volume weighted method is assumed to be a more realistic representation of runoff since it captures the hydrologic significance of directly connected impervious surfaces. If this volume is defined as the “true” volume, the composite curve number under-estimates small storm runoff because it does not account for the contribution from directly connected impervious surfaces. By calculating the runoff using the volume weighted method, the impervious surfaces are all assumed to be directly connected to the storm sewer system and ultimately to the stream. For many of the residential areas in the Pompeston Creek Watershed, the roadways, driveways and much of the rooftops were directly connected to the stream, thereby making direct connection in the runoff calculation a valid assumption.

The most important difference between the lot and subdivision scale analysis was the influence of streets. The four sample subdivisions were modeled to determine the runoff contribution from the impervious and pervious surfaces. Since they are directly connected to the natural stream system via the storm drainage network, all streets were considered effective impervious areas (EIA) in the subdivisions. The total impervious surface of the subdivisions was broken up into streets, driveways, and rooftops. Total surface areas for the impervious and pervious surfaces throughout each subdivision were calculated (See Table 16 and Table 17). The runoff contribution from each surface type for the water quality storm was calculated in Table 19. As seen in Table 17, streets account for the largest portion of the impervious surface area on the subdivision scale in the Georgian Drive, Cardinal Drive, and N. Washington and E. Central Ave. subdivisions. When streets are included as directly connected impervious surface, the EIA and likewise the runoff contribution from the impervious surface nearly doubles in these subdivisions for the water quality storm (Table 19) when compared to the lot scale analysis. The drastic effect of directly connected streets on runoff volume was also documented by Lee and Heaney (2003). The disproportionate runoff contribution from directly connected impervious areas was also seen in drainage areas studied by Beard and Chang (1979), Booth and Jackson (1997), and Alley and Veenhuis (1983).

In all four subdivisions in the Pompeston Creek study, the runoff contribution from the sum of the streets and driveways alone is either close to or dramatically exceeds what would be expected if using the composite curve number method (Table 19 and Table 20). This is particularly alarming because in the three most urban subdivisions, the street runoff is directly discharged to the area streams through the storm sewer network. This direct connection of the residential street area may be the primary reason for the TR-55 under-estimation of runoff at the subdivision scale during the small storm. There are no management practices in place to reduce the volume of runoff from the streets or promote infiltration. Therefore, even if the rooftops are not actively contributing runoff during the water quality storm, the contribution from streets and driveways exceeds what is predicted using the composite curve number. These results suggest that the street EIA is a crucial parameter for accurate watershed modeling in urbanizing areas.

The addition of streets does not have such a drastic effect in the Tom Brown Rd subdivision. Due to the large lot sizes in the more rural area, streets only account for 1% of the total land area, or 16% of the total impervious area, and half of the runoff volume is contributed by the pervious surface. Also, the streets are drained by swales that can help to promote infiltration and slow runoff velocity. Long driveways are characteristics of the area, and account for the greatest proportion of the impervious area, which is 4% of the total land area (Table 17). This suggests that driveway and rooftop disconnection would be the most effective at reducing runoff from the overall Tom Brown Rd. subdivision. This is in contrast with the other more densely urban subdivisions. While rooftop and driveway disconnection could be achieved with homeowner participation at Georgian Drive, Cardinal Drive, and N. Washington and E. Central Ave., streets have a greater influence on total storm runoff because approximately half of the runoff from the impervious surfaces in the subdivisions is conveyed by streets. Street disconnection would need to be facilitated with a different management strategy.

<u>Sensitivity of runoff volume to curve number in the SCS equations</u>

The under-estimation of the composite curve number in predicting runoff volume at the small storm may be partially attributed to the sensitivity of the SCS runoff volume calculations to curve number. Slight variation in the curve number can produce large variations in the prediction of runoff volume (Hawkins 1975). Therefore, SCS models are also very sensitive to the land use classification used to define the curve number of a land parcel (Rawls, Shalaby, and McCuen 1981). Hawkins (1975) also found that the runoff volume in the SCS equations is more sensitive to curve number than precipitation depth. However, rainfall depth is still very important to the application of the SCS equations. Curve numbers from drainage area studies in the literature were found to increase as storm depth decreases (Bonelid, McCuen, and Jackson 1982; Hjlmfelt 1991; Hawkins 1993; Tshrintzis and Hamid 1997; Kottegoda, Natale, and Hamlet 2003). In the cited studies, the curve number used to represent runoff during the small storm was larger than the curve number used to represent the runoff response from a larger storm over the same drainage area.

Results similar to those in the literature were observed when composite curve numbers were adjusted to represent the runoff predicted with the volume weighted method on both the lot and subdivision scale for the water quality through the 100-year storm. These calculations confirm that as the rainfall depth decreases, the curve number increases. Even though, for the 100-year design storm, the composite curve number method was shown to predict a similar runoff volume to the volume weighted method, it was necessary to dramatically increase the composite curve number to predict the same volumes for the water quality storm. Table 61 was compiled to illustrate the drastic change in the subdivision curve number when the volume weighted method is used at the water quality storm.

Table 61: Adjusted Curve Number for Predicting Runoff Volumes at the Subdivision Scale for the Water Quality Storm

Subdivision	Original Composite CN	Composite CN Runoff Volume (ft^3)	New Composite CN	Weighted Volume Runoff (ft^3)
Georgian Dr.	70	4,443	83	34,325
Cardinal Dr.	79	11,674	85	22,085

Tom Brown Rd.	77	45,520	78	60,984
N. Washington & E. Central	81	9,365	86	16,422

In lieu of the previous results, the SCS equations were examined for their application to the small, frequent storm event in New Jersey. When using the SCS equations there is a threshold at which the runoff response is zero (Hawkins 1975). For this reason the SCS runoff equation is only valid when $P > 0.2S$. Graphing the runoff for a 1-acre drainage area against increasing curve numbers for the water quality storm displayed a curvilinear relationship between curve number and runoff volume (Figure 33). When rainfall depth was equal to 0.2S, the runoff was zero. The curve number that is associated with this situation is the threshold where runoff response is zero. The curve number threshold for the water quality storm is 61. If a curve number less than 61 were to be used in the SCS equations, an invalid runoff volume would be predicted from the left side of the curve. However, as the storm depth increases, the threshold curve number decreases, and the runoff response curve begins to flatten out asymptotically to a 45 degree line (See Figure 34). The curve number that corresponds to zero runoff for the design storm depths is given in Table 62. Low curve numbers should be used with caution in the modeling of small storm events because there is a possibility that runoff will be predicted for an event that should not produce runoff. It should also be noted that the curve numbers are empirical parameters and were developed from data from small agricultural watersheds (Fennessy, Miller and Hamlet 2001). For this reason, the application of the SCS equations in urban areas is not well understood and is the basis of many studies (Rallison and Miller 1982; Rawls, Shalaby, and McCuen 1981; Bonelid, McCuen, and Jackson 1982; Hjlmfelt 1991; Grove, Harbor, and Engel 1998; Fennessy, Miller, and Hamlet 2001).

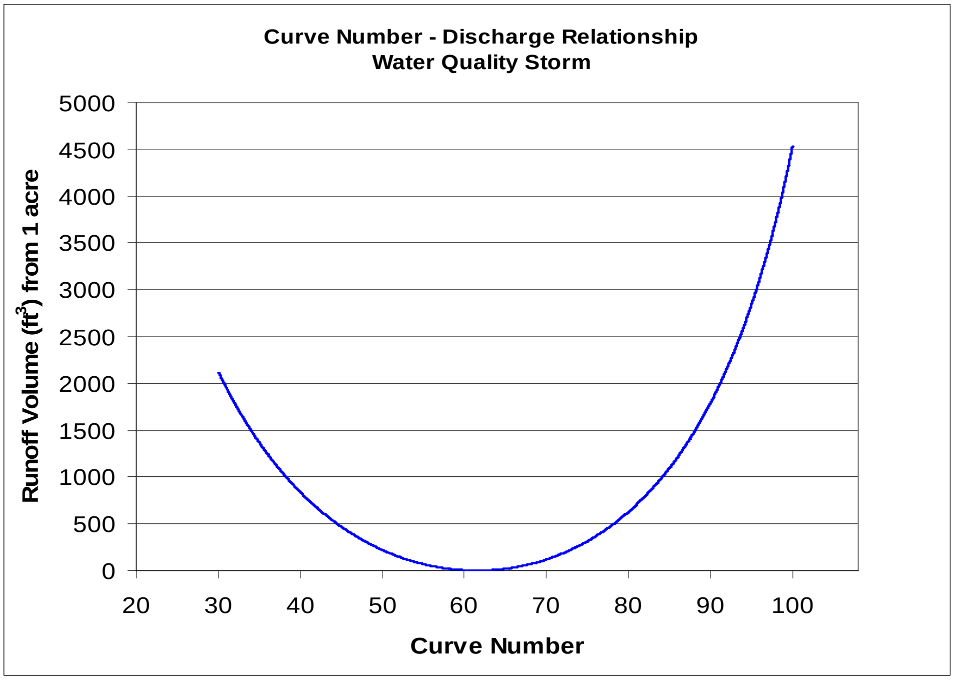

Figure 33: Curve number – discharge relationship for water quality storm

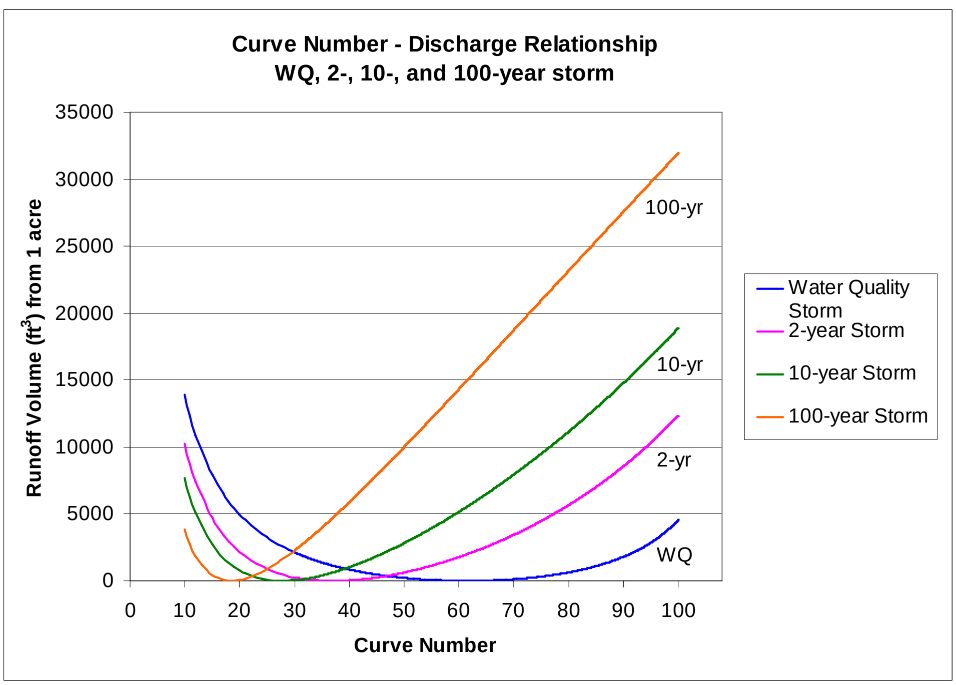

Figure 34: Curve Number-Discharge Relationship for water quality through 100-yr storm

Table 62: CN Threshold for Design Storms

Storm Depth (in)	CN Threshold
1.25	61
3.4	37
5.2	28
8.8	18

The SCS equations were originally developed to predict the runoff response of a watershed whose soil and land cover can be represented by a uniform curve number (SCS 1986). The curve numbers were determined from runoff and infiltration studies of uniform, agricultural watersheds. Since then, TR-55 has been developed with applications for use in urban watersheds. For ease of calculation for the stormwater engineer or manager, TR-55 recommends a composite curve number be used for urban land uses. The composite urban curve numbers in TR-55 Table 2-2a are based on average lot size and hydrologic soil group of the pervious areas within residential areas. A composite curve number was calculated as the average weighted curve number of the percent impervious surface area designated for the land use and the curve number assumed for the pervious surface, which is pasture in good hydrologic condition (SCS 1986). The composite curve numbers calculated for the subdivisions were consistent with the curve number that would be assigned to each subdivision if TR-55 Table 2-2a was applied to the imperviousness and soil type of each land use, except at Tom Brown Rd. Since they were calculated using the composite method, the urban curve numbers listed in Table 2-2a of TR-55 may not accurately represent runoff when used for the small storm in watershed models.

TR-55 makes a key assumption in using the composite curve number: the average weighted curve number can be used to represent the runoff response of an area with varied curve numbers. In this scenario, all rainfall is converted to runoff in a spatially uniform fashion, no one piece of land area in the basin will contribute more than the next, and time is not a factor in the runoff response. This is not entirely the situation in a real world example.

<u>Influence of directly connected impervious surfaces</u>

The composite curve number under-estimates runoff from the small storm event because, on examination of the SCS equations, it does not account for the contribution of directly connected impervious surfaces. On both the lot and subdivision scale, the runoff generated from the impervious surfaces was near equal (Tom Brown Rd.) or greater than that generated from the pervious surfaces, although the pervious surfaces account for the largest amount of land area (See Table 11 and Table 19).

At the small storm the nonlinear characteristics of the SCS equations may explain the under-prediction in runoff when an average curve number is used to represent a land parcel that consists of areas of a wide range of curve numbers (Grove, Harbor, and Engel 1998). In the urban setting, adjacent land parcels are often assigned curve numbers that differ greatly due to the multiple land uses that are encountered. Since the impervious areas have a high curve number, they will produce a disproportionate amount of runoff than areas of low curve numbers during small storms (See Figure 33). Averaging the curve numbers to determine the composite curve number loses the disproportionate contribution from the impervious surfaces during the water quality storm.

The largest difference between the volume predicted with the composite curve number method and the weighted volume runoff method is seen for the water quality storm at

Georgian Drive in both the lot and subdivision scale analyses (See Table 12 and Table 21). The under-prediction is greatest for areas with a lower pervious curve number (B soils). While the pervious area does not contribute runoff at Georgian Drive, directly connected impervious surfaces contribute 100% of the runoff volume, which is not reflected by the composite curve number. When impervious areas are assigned a curve number of 98, the greatest difference between pervious and impervious curve number is seen in areas of B soils, which were assigned a curve number of 61. Although no runoff is generated when a curve number of 61 is used, the runoff generated from the impervious surface alone on the Georgian Drive lot is 14 times more than what is calculated when the composite curve number is used. Since the pervious surfaces (C soils) in the other subdivisions have a curve number of 74, they will be expected to contribute some runoff to the overall runoff volume at the small storm as seen in Table 12 and Table 19. The volume under-estimation in these study areas, while important, is not as drastic as that seen at Georgian Drive.

As the storm depth increases at Georgian Drive, the weighted volume approaches what would be predicted by the composite curve number (Figure 35). At the water quality storm, the curve number difference of 14 represents a difference of almost 30,000 cubic feet (Figure 36). The volume weighted curve number is much greater than the composite curve number for the water quality storm, but approaches the composite curve number at larger storms (Figure 37). As the storm depth increases, the difference in calculated volume between the composite curve number and the weighted volume method diminishes. The composite curve number method makes the best approximation of discharge where the line crosses zero between the 7.5” and 8.0” inch storm. This value is about an inch less than the 100-year storm (Figure 38).

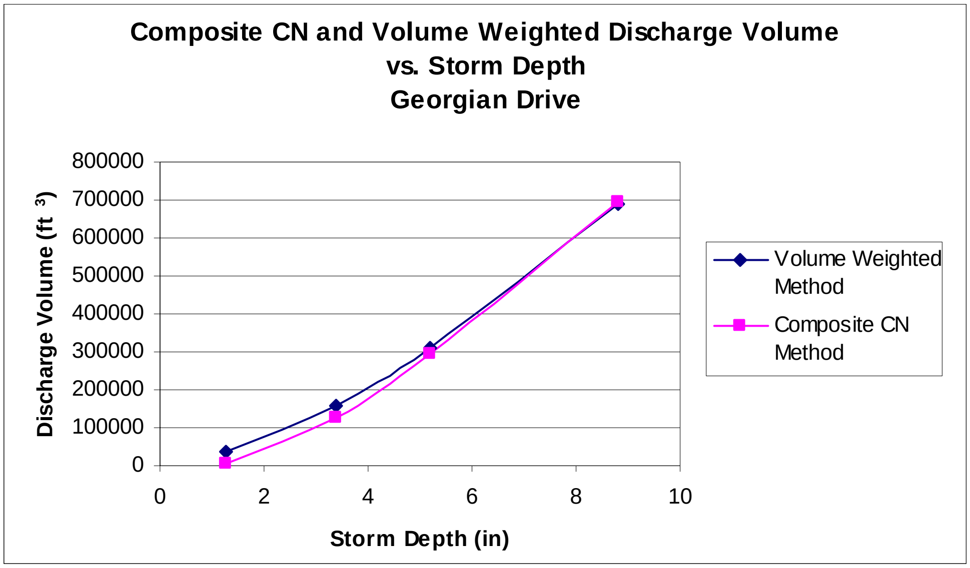

Figure 35: Predicted discharge volume for composite CN and average weighted volume at Georgian Dr. vs storm depth

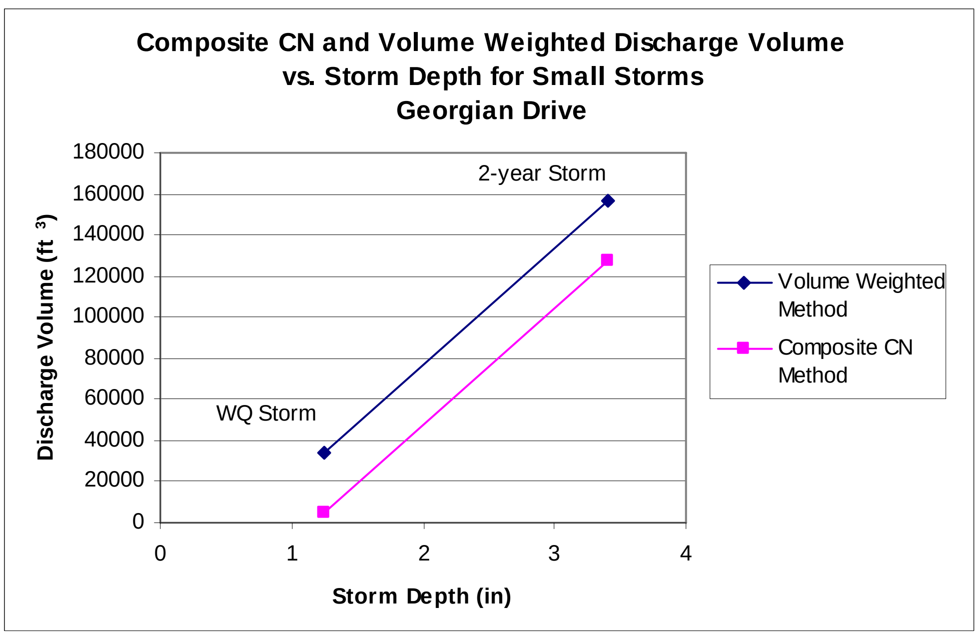

Figure 36: Predicted composite CN and weighted volume discharge at Georgian Dr. vs storm depth for small storms

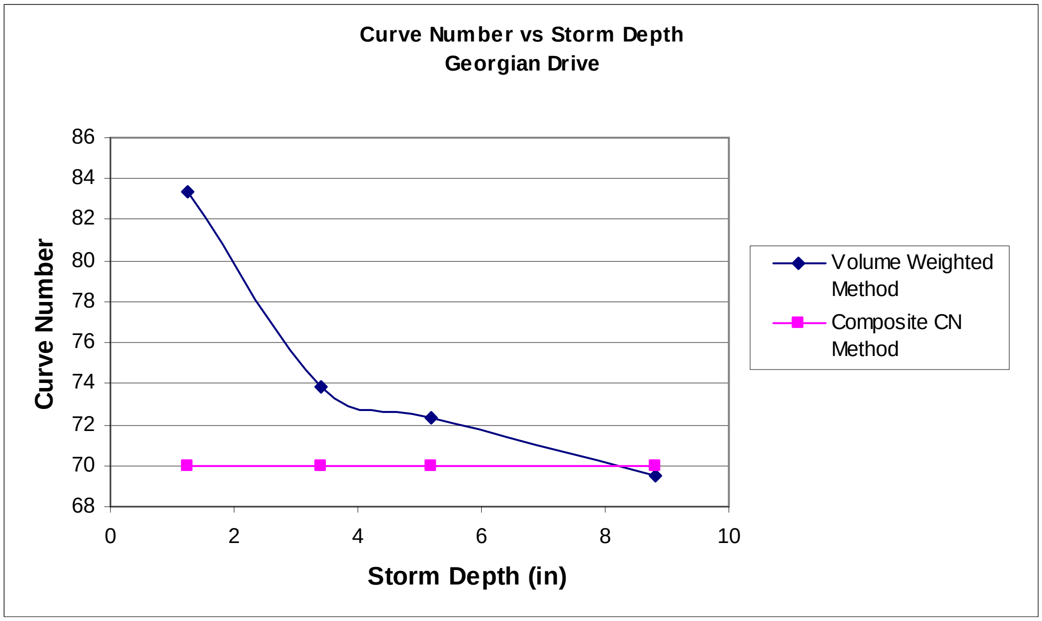

Figure 37: Curve Number vs storm depth at Georgian Dr.

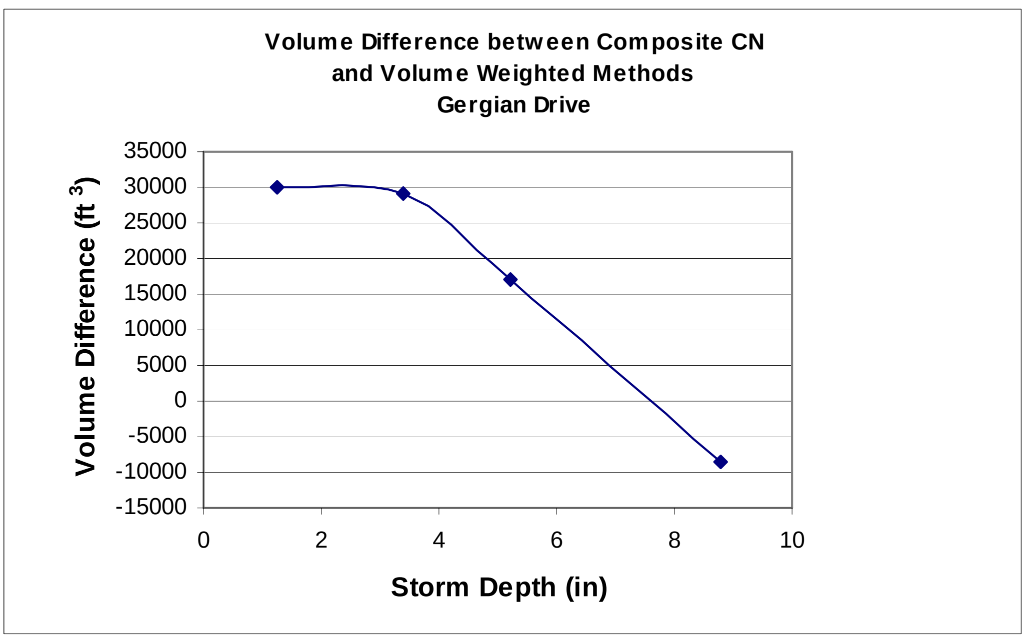

Figure 38: Difference in predicted composite CN and weighted volume discharge at Georgian Dr. vs storm depth

The more linear trend of the curve number runoff response curves for large storms (Figure 34) may explain why the large storm composite curve number volume approaches the weighted volume predicted for the lots and subdivisions (See Table 12). Previous studies also show that as storm depth increases, the volume predicted by the composite method approaches the sum of the volumes from the contributing surfaces (Grove, Harbor, and Engel 1998). As rainfall depth increases, the proportional change in runoff volume decreases (Bonelid, McCuen, and Jackson 1982), so averaging of the curve numbers is more applicable at greater storm depths. At increasing storm depth, the proportional difference in volume from each surface is not as drastic so the curve number discharge relationship is more linear. The curve number method was originally developed to model these larger storms (Kottegoda, Natale, and Raiteri 2000). The effect of the disproportionate contribution from impervious surfaces is dampened at the larger storm because pervious areas begin to contribute a greater proportion of runoff once the infiltration capacity is exceeded.

A footnote to Table 2-2a in TR-55 indicates that all impervious surfaces were assumed to be connected. However, this assumption appears to be incorrect when the weighted volume method is used to simulate a more realistic runoff situation because the curve numbers in TR-55 are composite curve numbers. In the directly connected situation, rainfall is assumed to run from impervious surface to impervious surface. Rainfall falling on the rooftop would be transported directly to the driveway, directly to the streets, into the storm drain network, and directly discharge to the receiving stream. This volume weighted calculation method is more likely what is expected at the smaller storm as compared with the TR-55 composite curve number.

Examination of an average lot at Georgian Drive illustrates the difference between the composite curve number and volume weighted method (See Figure 3, Figure 4, and Figure 5). If the discharge volume from the impervious surfaces is added to the discharge volume from the pervious surfaces for a small storm, a much greater volume of water is encountered than that would be calculated using the composite curve number of 66 (See Figure 3 and Figure 4). The 0.58 acre lot would be expected to contribute 23 cubic feet of runoff for the water quality storm using the composite curve number. A 1300% larger volume, 321 ft^3, is calculated when the runoff contribution from each surface is calculated separately using the volume weighted method. While both techniques attempt to account for impervious surface in the runoff calculations, only the volume weighted method describes the hydraulic significance of directly connected impervious surfaces. In this scenario TR-55 underestimates the runoff from a water quality rainfall event by 298 cubic feet, or 93% for the Georgian Drive lot. In other words, the runoff calculated by the composite curve number method only represents 7% of the runoff that could be expected to be produced from the water quality storm. The curve number for the Georgian Drive lot would need to be increased by 13 form 66 to 79 in order to predict the volume calculated with the volume weighted method.

One way to circumvent the inherent under-estimation of runoff volume at the smaller storm event is to use the volume weighted method when modeling runoff response (NJDEP 2004; Grove, Harbor, and Engel 1998). This is not practical on the watershed scale, for each individual impervious surface area, as well as each individual pervious area, would need to be calculated. Runoff would be calculated for each contributing area and then added together. While calculating the impervious area separately is a better representation of the runoff response for a directly connected drainage area, it can be tedious and time consuming (Hill, Botsford, and Booth 2003; Lee and Heaney 2003).

An analysis of the runoff contribution from each impervious and pervious surface at each lot determined that as the storm depth increases, the percent contribution of the impervious surface decreases. During the larger storm events the pervious surfaces begin to contribute runoff. For the 100-year storm, the percent of runoff volume from the impervious surfaces is approximately equal to the percent impervious area of the lot (Table 12). The volume contribution is also broken down into rooftop and driveway runoff. For example, at the Georgian Drive average lot, the water quality storm rooftop runoff is 195 ft^3 and driveway runoff is 125 ft^3. These impervious surfaces combined account for 99.9% of the total runoff volume from the lot. If all or a portion of this runoff could be disconnected (i.e. routed and infiltrated over the adjacent pervious area), the runoff from this lot could be eliminated for the water quality storm. The volume calculations in Table 12 illustrate the potential volume reductions at the individual lot scale that could be achieved through a disconnection management strategy at the individual lot scale.

The directly connected impervious area can also be described as the effective impervious area (EIA). The EIA of the study areas was analyzed with a method similar to that employed by Alley and Veenhuis (1983). Alley and Veenhuis (1983) found that the ratio of effective impervious area (EIA) to total impervious area (TIA) is independent of lot size for a geographic area. As lot size increased, TIA and EIA decreased, however, the ratio EIA/TIA appeared to be independent of lot size and showed little variation between the basins sampled. In a similar manner, EIA was calculated for the subdivisions studied in the Pompeston Creek Watershed. Rooftops were found through a field survey to be about 50% effective in contributing runoff volume. The ratio EIA/TIA under existing field conditions was in the range of 0.82-0.84 for the four basins sampled, greater than the range of 0.52-0.66 observed by Alley and Veenhuis (1983).

Lot and Subdivision Curve Number Analysis

The lot and subdivision curve numbers increased when the curve number was adjusted to match the runoff volume that was predicted using the volume weighted method for storms smaller than the 100-year design storm (See Table 18 and Table 23). The curve number increase from the composite curve number was greatest on the subdivision scale because the volume nearly doubled at the water quality storm when streets were included as effective impervious area. This increase in volume was reflected by an incremental, though not proportional, increase in curve number. The use of a new curve number to represent the volume that is expected to run off from an urbanizing area is very important to accurate small storm hydrologic modeling. This new curve number can then be used to compare the runoff reductions achieved with different management strategies. This method can also be used to generate new curve number tables to represent direct connection or other watershed conditions for subsequent small storm analyses.

The composite curve number can be useful in predicting the large flood events for flood management design, but falls short when used for design at the water quality event. This under-estimation of the composite curve number may lead to the false assumption that the discharge from the smaller, more frequent storm would not significantly impact urban streams. The weighted volume calculations, however, show that substantially more stormwater is discharged to the receiving streams than TR-55 would predict. This may have lead to the mismanagement of the smaller storm. It also infers that reducing the EIA through disconnection could be a very useful tool in managing the excess discharge from the smaller storm.

Curve number in watershed calibration

The Pompeston Creek Watershed model was calibrated using rainfall-runoff data from three smalls storms of depths 0.65”, 0.76”, and 1.20” collected in the fall of 2005. The composite curve number assigned to each basin in the base model was increased when the model was calibrated to match the observed runoff. The difference between the calibrated and composite curve numbers for the calibrated basins is highlighted in Table 51. As is seen in the table, the composite curve number was increased by 5.9%, 3.5%, 7.5%, and 3.5% for Basins 8, 9, 12, and 13 respectively. These incremental increases in curve number when the model is calibrated account for a 70.2%, 36.4%, 84.6%, and 36.4% increase in volume for the basins when the calibrated curve numbers are used instead of the composite curve number at the water quality storm. This suggests that in order for models to accurately predict basin runoff from the water quality storm in highly urbanized watersheds, urban curve numbers need to be more carefully chosen (Rawls, Shalby, and McCuen 1981). If the appropriate curve number is not chosen, BMPs may be under-designed to control runoff at the water quality storm.

The difference between composite and calibrated curve numbers could be explained by a number of factors. In all six observed hydrographs over the three storms there is an initial runoff response at the beginning of the storm, runoff subsides, and then there is a large peak in the hydrograph. If the relationship between I_a and S ($I_a = 0.2S$) were held constant through the calibration, the model could not be calibrated to reflect the initial response at the onset of the storm. The curve number would need to be set very high (which decreases the initial abstraction) in order for a response in the beginning of the hydrograph. However, the increase in curve number over-predicts the second, major peak in the hydrograph and the overall runoff volume. The initial peak seen in each observed hydrograph may be from roads and other directly connected impervious surfaces within close proximity to the stream gauge. The time of concentration for these areas is probably shorter than for more remote areas and the peaks arrive downstream soon after runoff begins.

An understanding of the relationship between curve number and initial abstraction in the SCS equations is important when the parameters are used in hydrologic modeling. The initial abstraction term is used to express all losses before runoff begins and includes all water retained in surface depressions, water intercepted by vegetation, evaporation, and infiltration. The initial abstraction value can be adjusted and lowered in the model to initiate a quicker runoff response as the abstraction volume is filled with the onset of the storm (Tshrintzis and Hamid 1997). However, when the initial abstraction is lowered independently from the curve number, the calibrated curve number will drop in order to match the volume of the observed storm. If the relationship between I_a and S is violated, the curve number tables in TR-55 are no longer valid, since the tables were designed under the key assumption that I_a=0.2S. Since the purpose of this study is to compare the runoff response of residential urban areas, using the curve number as an indicator, to the suggested and commonly used curve numbers in TR-55, the relationship between I_a and S was held constant for consistency in comparison. The relationship between I_a and S was original generalized by SCS based on data from agricultural watersheds. The approximation may be especially important in an urban application since the combination of impervious areas with pervious areas can imply a significant loss that may not take place (Fennessey, Miller, and

Hamlet 2001). If the modeler intends to use an initial abstraction value other than that suggested in TR-55, the preferred initial abstraction should be entered into the SCS volume equation with rainfall-runoff data and used to recreate the urban runoff curve numbers (Table 2-2 in TR-55). New storage or curve number relationships are then established for each cover type and hydrologic soil group.

The antecedent runoff condition (ARC) is also a factor that may contribute to the early runoff response seen in the observed hydrographs. ARC is the index of runoff potential before a storm event and is important in determining the curve number (SCS 1986). The curve numbers in TR-55 Table 2-2 represent average ARC. The ARC accounts for the variation in curve number at a site from storm to storm. The curve number for the average ARC at a site is the median value as taken from sample rainfall and runoff data. At a site where a storm recently occurred, there would be more moisture already present in the soil, and thus a higher runoff potential before the storm event. Since both the 10-11-05 and 10-24-05 storms had rainfall events approximately two days earlier (See Table 42), the antecedent runoff condition may contribute to the initial runoff response at the onset of the storm events. However, a similar initial response was seen at both sites during the 11-16-05 storm event which had seven days of no rain prior to the storm.

The weighted volume was calculated for each basin by summing the volume of runoff from each soil-cover complex polygon throughout the watershed for the water quality storm. This volume was compared with the basin-wide composite curve number. Basin 12 had a 37% volume contribution from residential areas while Basins 8, 9, and 13 had a 72%, 59%, and 61% of overall volume contribution. The runoff from Basin 12 was 39% from wetlands and 22% from other urban areas, which may account for the difference between the calibrated and basin-wide composite curve numbers.

In the next analysis, all streets were assumed to be directly connected to the receiving streams in the calibrated basins to determine the potential influence of streets for the water quality storm. Street area was approximated by summing the street length throughout the basin and multiplying it by an assumed average street width of 30 feet. The runoff volume from the water quality storm was then calculated from the street area alone to represent direct connection of all the streets. Results of these calculations are in Table 66. Approximately a third of the runoff from the water quality storm is expected to come from the streets in Basins 8, 9, and 13. About 10% of the overall storm runoff may be contributed by streets in Basin 12.

Table 63: EIA of each basin as represented by street area and runoff from the water quality storm

Basin	Total Street Area (acres)	% of Basin Area	Runoff Volume from Streets (ac-ft)	% of Runoff Volume from Streets in Modeled Water Quality Storm
12	36.0	5.1%	3.10	9.8%
8	22.6	8.7%	1.95	32.8%
9	34.6	10.0%	2.99	38.0%
13	17.0	6.7%	1.47	25.5%

The curve numbers for the watershed model needed to be increased to reflect the observed rainfall-runoff data, which emphasizes the need for a modification of urban curve number for use in small storm modeling. The data implies that when the composite curve number is used for the design of water quality controls, the water quality goal will be missed because it is possible that the water quality storm will generate more runoff than expected. Two conclusions can be drawn from this: 1) curve numbers should be modified to reflect urban runoff conditions at the small storm; and 2) the water quality storm needs to be managed through a stormwater control. Disconnection could be used to bridge the two.

Small Storm Management through Disconnection

Since the composite curve number does not accurately represent the runoff volume from small storms, the runoff volumes that were calculated using the volume weighted method were used as the baseline to compare the disconnection scenarios. The effectiveness of disconnection at reducing runoff volume decreases as storm depth increases, however the use of disconnection is promising for the small storm.

Lot Scale Disconnection

Figure 39 and Figure 40 illustrate the reduced efficiency for rooftop and a combination of rooftop and driveway disconnection at the lot scale. These graphs are a representation of data in Table 13 and Table 14. In both situations the capacity for runoff reduction through disconnection is the greatest for the Georgian Drive lot. 194 ft^3, or 60.4% of the runoff from the Georgian Drive lot was disconnected from the storm drainage network with rooftop disconnection. The volume disconnected in the other study areas was not as large a percent with 30.2%, 11.6%, and 36.1% disconnected on the Cardinal Drive, Tom Brown Rd., and N. Washington and E. Central Ave. lots, respectively. Since Georgian Drive (B soil) has a more pervious soil than the other areas (C soil), it can be inferred that the hydrologic group and permeability of the pervious area limit the effectiveness of disconnection. As can be seen in the graph, disconnection is still somewhat effective in reducing volume at Georgian Drive through the 2–yr (3.4") and even the 10-yr (5.2") storms (Table 13, Table 14, and Table 15). The decrease in effectiveness of disconnection with increasing rainfall is due to the greater runoff contribution of the pervious surfaces as rainfall depth increases. As the storage capacity of the soil is filled, all additional rainfall is converted to runoff and the percent of the overall volume reduced through disconnection is diminished.

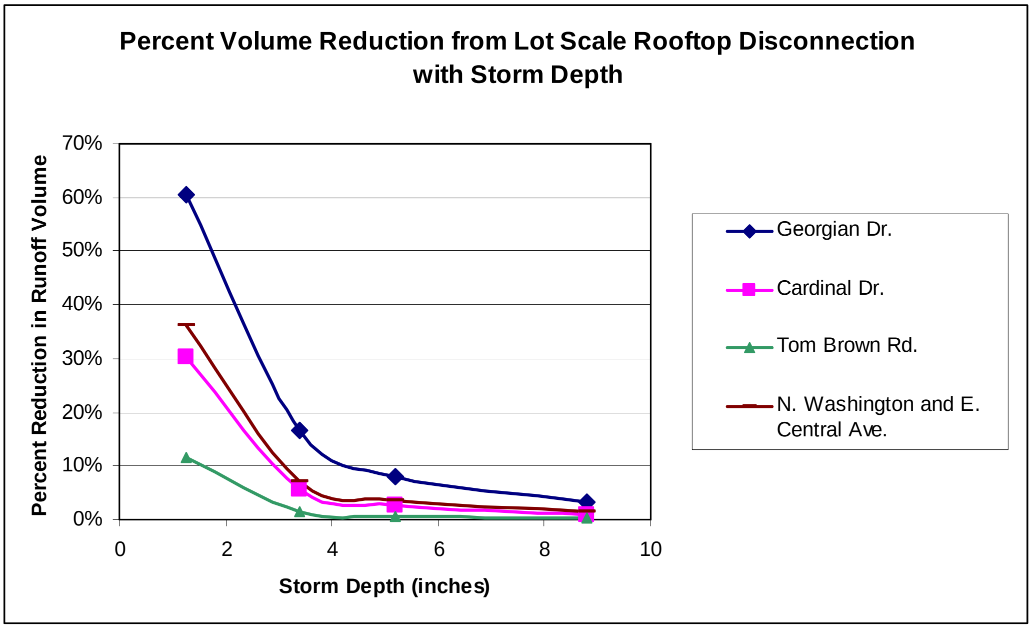

Figure 39: Percent Runoff Volume Reduction from Rooftop Disconnection with Storm Depth

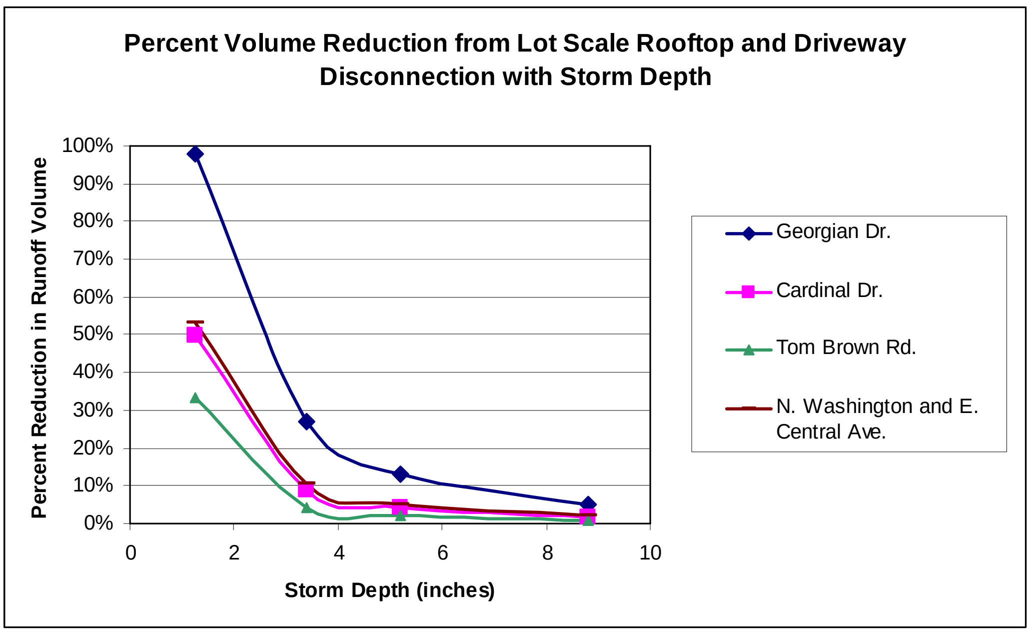

Figure 40: Percent Runoff Volume Reduction from a Combination of Rooftop and Driveway Disconnection with Storm Depth

Table 11 highlights the total amount of runoff from all the impervious surfaces on each lot for the water quality storm. It can be assumed that even for the best case scenario (100% disconnection of runoff) the maximum volume reduction that can be achieved, without the addition of other BMPs, is the impervious surface volume in the table. This volume is the limit of disconnection. In the Georgian Drive lot, 99.9% of the total storm runoff may be eliminated through a disconnection strategy, however, for the other lots (Cardinal Drive, Tom Brown Rd., and E. Central Ave. and N. Washington Ave.) the maximum reduction that can be achieved through disconnection alone is 69.1%, 45.1%, and 74.8%, respectively. It is observed that impervious surfaces dominate the runoff response for the lot with B type soils. Therefore, the hydrologic condition of the pervious area will influence whether or not runoff is generated from the water quality storm.

The "Disconnected Volume" column in Table 14 shows that with increasing storm depth, the amount of volume disconnected tends to approach a constant value. For example, at Cardinal Drive 414 ft^3 is disconnected at the water quality storm, 521 ft^3 is disconnected at the 3.4" storm, and 483 ft^3 is disconnected at the 5.2" storm. The total runoff volume increased much faster from 418 ft^3 to 5,321 ft^3 to 11,072 ft^3 over the increasing storm depth.

Isolation of the runoff from each surface using the weighted volume calculation is beneficial because it targets areas where disconnection could be considered. At Georgian Drive, removing runoff from the rooftops should be targeted since they contribute the greatest amount of runoff (See Table 11). Analysis of the remaining lots, all of which are C type soil, shows that managing runoff from the rooftops on both the Cardinal Drive, and N. Washington and E. Central Ave. lots, and the driveways on the Tom Brown Rd. lot has the potential for having the greatest reduction in runoff volume (Table 11). Disconnection could have the largest impact on runoff volume in areas with small lots and pervious soils. While disconnection can be beneficial on large lots, and should be recommended, it may not have as great an impact on reducing the overall runoff volume. In the large lot situation, the pervious areas may be contributing the greatest proportion of runoff because of their large surface area, so disconnection may not be as useful as a tool to reduce runoff volume for larger storms.

The lot scale disconnection volume is also important in calculating the amount of runoff an individual homeowner can be responsible for disconnecting. Using this method, a homeowner at Georgian Drive who disconnects all roof downspouts can be credited with reducing 194 ft^3 of stormwater runoff from the lot during the water quality storm (See Table 13). If the driveway is also disconnected from the street drainage network, credit for disconnecting 315 ft^3, or 98.1% of the total runoff, can be attributed to the homeowner for the water quality storm (See Table 14). This volume is the difference between the runoff from directly connected impervious surface and the disconnection of that surface. Therefore, disconnection has the potential to entirely eliminate lot runoff from the water quality storm in some areas. Partial reduction and/or elimination of runoff from the small storm is important because the reduction of the number of runoff forming events is a step toward achieving pre-development runoff conditions (Hammer 1972; Hawkins 1975; Hollis 1975; and Booth 1991).

<u>Subdivision Scale Disconnection</u>

Multiple disconnection scenarios were modeled for the four subdivisions in an effort to highlight the theoretical impacts of disconnection on volume reduction during small storms. In the first scenario, the volume from all the rooftops was routed over the pervious lawn areas. The volume from the disconnected impervious surface was uniformly spread over the adjacent pervious area and was expressed as additional rainfall. This volume was added to the storm volume over the pervious area to calculate the pervious runoff after the volume from the disconnected impervious surface was routed over it. Since the driveways and streets are still connected, the volume contribution from these surfaces was left unchanged and added to the pervious runoff. This scenario was repeated adding the cumulative disconnection of driveways and streets. Empirical results are shown in Table 22 for the water quality storm.

The greatest calculated volume reduction from disconnection was a 96% reduction (32,844 ft^3) in volume when the disconnection of all impervious surfaces was modeled in the Georgian Drive subdivision. This is a case that illustrates the importance of the pervious curve number to the amount of stormwater that disconnection can mitigate (Alley and Veenhuis 1983). In all of the subdivisions, except Tom Brown Rd., an impervious surface area equivalent to all the rooftops and driveways and a portion of the streets would need to be disconnected in order to match the volume generated by the composite curve number for the water quality storm. While the greatest reduction was seen at Georgian Drive, the other subdivisions with less pervious soils could also mitigate up to 58% of runoff volumes for the water quality storm when all impervious surfaces were disconnected.

The disconnection scenario was also modeled for the larger 2-, 10-, and 100- year storms. Table 67 and Table 68 display the volume with and without disconnection through increasing storms for the Georgian Drive study area. As the storm depth increased, disconnection was less effective. Hollis (1975) states that at the larger storms, the runoff volume lost to interception and infiltration is minor compared to the overall volume. Therefore, the effects of disconnection at the larger storm would be lost.

Table 64: Georgian Dr. Runoff volumes after disconnection for storm events

	Disconnected Volume (ft^3)			
Storm	**No Disconnection**	**Rooftops**	**Rooftops & Driveways**	**Rooftops, Driveways, & Streets**
Water Quality	34,325	23,522	16,771	1,481
2-year	156,947	139,044	128,415	105,197
10-year	312,063	293,377	282,095	258,616
100-year	687,682	670,606	660,413	639,112

Table 65: Georgian Dr. % runoff reduction by disconnection for storm events

	% Volume Reduction due to Disconnection			
Storm	**No Disconnection**	**Rooftops**	**Rooftops & Driveways**	**Rooftops, Driveways, & Streets**
Water Quality	0%	31%	51%	96%
2-year	0%	11%	18%	33%
10-year	0%	6%	10%	17%
100-year	0%	2%	4%	7%

Although, according to Table 67 and Table 68 , disconnection could remove up to 21,300 cubic feet of runoff during the 100-year storm, it is only 7% of the overall runoff from that storm. While reducing the impervious surface runoff from a small storm could relieve a lot of stress on the receiving stream, or even eliminate runoff altogether, it is only a fraction of the larger storm. Disconnection is not intended to relieve the impacts of large storms on the stream system. Other management strategies would be needed. However, disconnection can effectively reduce the number of bankfull flows and offset the effects of urbanization as described by Leopold (1968), Hammer (1972), Hollis (1975), and Booth (1991). Urbanization not only increases the magnitude of the peak flows and volumes, it also increases the number of channel forming peaks (Booth 1990). By breaking the connection of flow with a pervious area and promoting infiltration during the small storm event, the implementation of disconnection could reduce the amount of nonpoint source pollution carried by the water quality storm by reducing runoff volume and treating it over the pervious areas. Furthermore, the number of water quality runoff events could be reduced and set the stage to return the stream channels to a more natural condition.

The influence of the streets is important to the overall runoff volume. As can be seen in Table 22, even with the disconnection of all the rooftops and driveways in the subdivisions, nearly 50% of the stormwater runoff remains in the Georgian Drive subdivision. By including streets in the disconnection scenario, an additional 25% reduction could be achieved at both the Cardinal Drive, and N. Washington and E. Central Ave. subdivisions. Street disconnection only gained a 5% further reduction in volume at Tom Brown Rd. More realistic scenarios and existing disconnection conditions were modeled in a later analysis. It is important to note, however, the two extreme situations shown in Table 22: runoff volume with no disconnection and runoff volume with all surfaces disconnected. With no other BMPs in place, these scenarios highlight the utility as well as the limits of disconnection.

<u>Application of Field Data to Subdivision Scale Disconnection</u>

In some situations stormwater runoff volumes need to be reduced to meet water quality goals, but some level of disconnection may already exist throughout the watershed. Therefore, in the next phase of analysis, baseline disconnection conditions are established with field data for the study areas. In this way, a more realistic volume reduction can be determined with the implementation of disconnection. Although disconnection was theoretically shown to be effective in reducing runoff volume in the previous simulations where all the roof, driveways, and/or streets were disconnected, it was not simulated using baseline conditions observed in the field, and attainable disconnection goals were not described.

A field survey determined that approximately half of the roof downspouts were already disconnected in each subdivision. This situation was described as “existing conditions” and used as the baseline to compare with further implementation of disconnection. The stages of disconnection in Table 2 were established to simulate more practical applications of disconnection. The first stage of disconnection was determined to be disconnection of 100% of the rooftops. It requires the minimum amount of effort on the part of the homeowners and the municipality, therefore, an analysis of the volume reduction that could be achieved with 100% rooftop disconnection would be valuable for planning. Next, Stage 2 disconnection was determined to be 100% rooftop disconnection with the addition of 50% of the driveways. Most of the driveways sloped toward the street in the field survey. Therefore it was decided that more effort would be required to disconnect driveways, and 50% homeowner participation in driveway disconnection was modeled as the next phase. Finally, the effect of 100% impervious surface in stage 3 disconnection (including the streets) was examined as an ultimate management goal. In this scenario all the streets are also disconnected through the use of a BMP such as a tree box or rain garden. It was impractical to represent the streets as a separate disconnected surface, for all the surface runoff not diverted to a pervious area will eventually drain to the street in a residential drainage area. If the street is to be managed with disconnection, the excess runoff from other impervious surfaces will need to be included in the modeled volume. The data that was compiled for the subdivisions was then applied to the soil-cover complexes in TR-55 to generate curve number tables that could be used to simulate small storms over urban watersheds with various levels of disconnection.

The greatest reduction in runoff volume in from disconnection was a 95% reduction from existing conditions when 100% impervious surface was disconnected during the water quality storm at Georgian Drive. These results were expected based on the lot scale and theoretical subdivision scale analyses. The existing conditions runoff volume from Georgian Drive during the water quality storm was 0.68 ac-ft, which is still nearly seven times greater than what would be predicted if the composite curve number was used (Table 28). Although the greatest volume reduction is seen at Georgian Drive, disconnection of 100% of the impervious surfaces could achieve 51%, 31%, and 52% volume reductions at Cardinal Drive, Tom Brown Rd., and N. Washington and E. Central Ave., respectively. Even if rooftop disconnection is the only achievable management option, 0.12 ac-ft, or 18% of the overall runoff volume, can be reduced through the disconnection of the remaining connected rooftops at Georgian Drive. This reduction in volume would be a simple step toward a 100% impervious surface disconnection goal. While the utility of disconnection in reducing the overall runoff volume decreases as storm depth increases, 100% impervious surface disconnection would result in a 29%, 1.02 ac-ft, reduction during the 2-year storm at Georgian Drive (Table 25).

The runoff volumes after disconnection for the water quality storm were plotted against the percent of the impervious surface that was disconnected in Figure 41. The percent disconnected impervious surface is the impervious surface that is not considered effective impervious area. As the amount of disconnected area increased, the runoff volume from each subdivision decreased linearly. The slope of the line was similar for the Cardinal

Drive and N. Washington and E. Central Ave. subdivisions. This may be due to the fact that the same pervious curve number was used for both areas because they have similar C group soils. Although Tom Brown Rd also has C group soils, the relationship between volume reduction and disconnected area for the subdivision may be different because of the smaller initial amount of impervious surface on the larger lots. Tom Brown Rd. is 6% impervious surface so runoff from the large amount of pervious surface was a large percent of the overall runoff. The slope of the volume reduction line was steeper than the lines for the other subdivisions with C soils, but it may be because the larger lots produce a large amount of volume overall, so reductions in these volumes would produce a steeper line. The steepest line and greatest amount of reduction is noticed for the Georgian Drive subdivision. Georgian Drive is 25% impervious surface cover, and although the percent impervious surface is within the same range of Cardinal Dr (23%) and N. Washington and E. Central Ave. (28%), the more pervious B group soils allow for a greater reduction in runoff volume. The equations from these trend lines are empirical relationships and could be used to predict the volume reduction that could be expected for incremental disconnection of impervious surface for these areas.

When the decrease in runoff volume is normalized to area and plotted vs. percent disconnected area in Figure 42, the relationships between percent disconnected area and percent volume reduction for Cardinal Drive and N. Washington and E. Central Ave are nearly identical. Figure 42 also shows the dramatic effect a more pervious soil can have on the efficiency of disconnection. The greatest reduction in volume is calculated for the Georgian Drive subdivision, therefore, disconnection may be a feasible recommendation for similar urban residential areas with B type soils. Since disconnection has not been shown in these simulations to entirely remove runoff from the water quality storm in areas with C soils, it could be suggested that another BMP should be used to supplement or encourage disconnection.

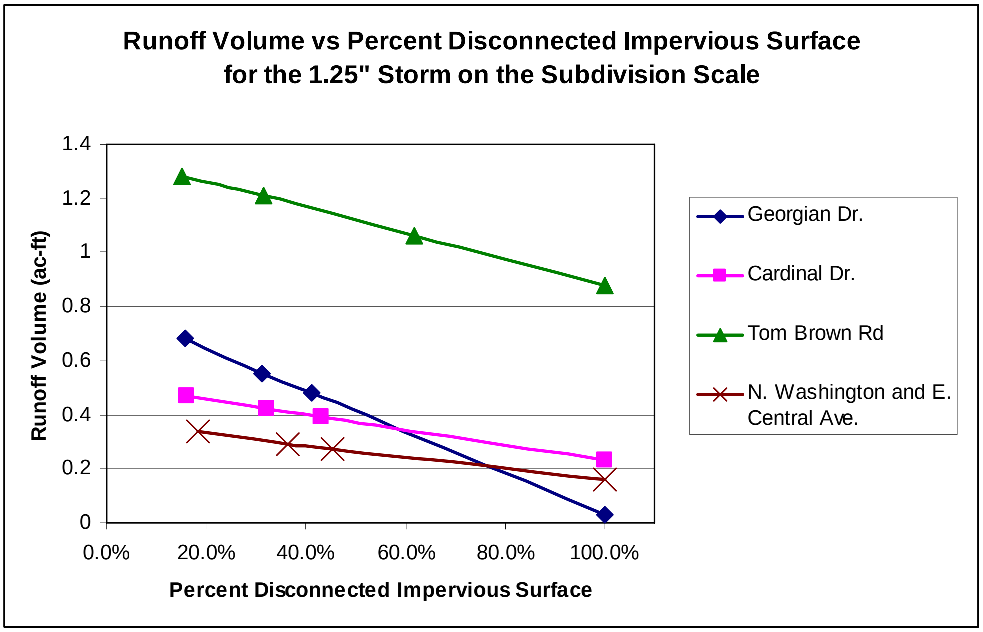

Figure 41: Runoff Volume vs. Percent Disconnected Impervious Surface for the water quality Storm on the Subdivision Scale

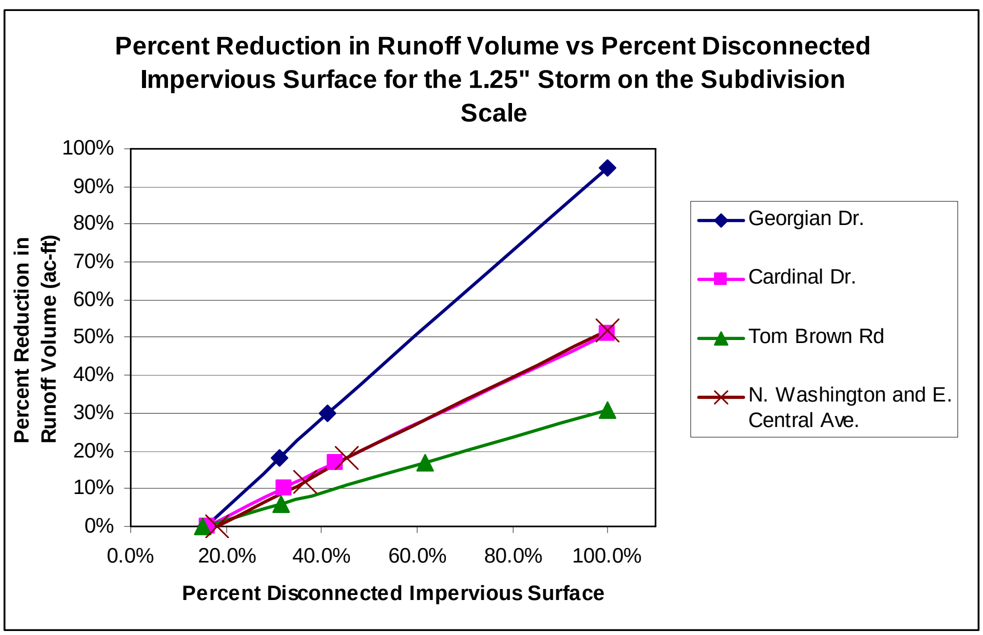

Figure 42: Percent Runoff Volume vs. Percent Disconnected Impervious Surface for the water quality Storm on the Subdivision Scale

Since the composite curve number predicts a smaller amount of runoff volume than the sum of all the runoff volumes in the directly connected scenario, it could be inferred that the composite curve number represents a disconnected runoff situation. Examination of Figure 15 indicates that at the water quality storm, the total runoff predicted by the composite curve number falls somewhere between Stage 2 and Stage 3 on the disconnection continuum. Therefore, the composite curve number represents a situation where nearly all the rooftops and driveways are disconnected. Even if this assumption were true, the composite curve number calculation does not mathematically quantify losses over pervious areas, nor consider the extent of contributing EIA.

An understanding of the effective impervious area in a drainage area and how it can be reduced is critical because physical data demonstrates that as a watershed reaches 10% EIA there is an onset of easily observable aquatic system degradation (Booth and Jackson 1997). Controlling the runoff from the EIA during a small storm event is key to reducing stormwater impacts on the receiving streams. Three of the four subdivisions studied in the Pompeston Creek Watershed are above this 10% threshold. The North Washington and East Central study area is 22.9% EIA, Georgian Drive is 21.1%, and Cardinal Drive is 19.3%. The rural Tom Brown Rd. study area has a 5-acre average lot size and 5.1% EIA (Table 31). The proximity of the residential areas to the receiving stream may also be a factor in their effect on the stream channel. Through disconnection of the rooftops and half the driveways, the EIA can be reduced to 15.3%, 14.7%, 13.1%, and 2.3% of the total land area for N. Washington and E. Central, Georgian Drive, Cardinal Drive, and Tom Brown Rd., respectively (Table 31). Although disconnection has reduced the EIA in all four subdivisions, three are still over the 10% threshold for aquatic ecosystem degradation when rooftops and driveways are disconnected. Further stormwater management control beyond Stage 2 disconnection involving some disconnection of the streets would be necessary to lower the EIA to less than 10%.

Subdivision Curve Number Analysis

Curve numbers were adjusted to represent the new reduced runoff volumes for both the theoretical disconnection simulations (Table 23) and the disconnection simulations using field conditions (Table 30). These curve numbers were compared to the composite curve numbers originally used to represent the subdivisions (Table 18). The greatest difference in curve number across the board for the theoretical disconnection simulation is seen in the Georgian Drive subdivision where the curve number varies from 83 for directly connected impervious surface to 67 for 100% disconnection (Table 23). This may be due in part by the low pervious curve number that was used, and thus supports the argument that the composite curve number method becomes less accurate as the impervious and pervious curve number are spread further apart (Grove, Harbor, and Engel 1998). For the residential areas with lot sizes of approximately two acres or less with greater than 6% impervious surface, the composite curve number represents the runoff response when between 12% and 28% of the total land area can be managed through disconnection. When the lot size becomes larger, as in the Tom Brown Rd subdivision (average lot is 5.84 acres) the effects of disconnection were dampened because the large pervious area contributes the most runoff. Also, the low infiltration capacity of the C soils in this area limit the volume reduction over the pervious area.

The subdivision curve numbers decreased with incremental disconnection and increasing storm depth, which is similar to the observations of the theoretical simulations.

The lots were ordered by increasing lot size, so the trend of decreasing curve number with increasing lot size and decreasing percent impervious surface would be apparent. The decrease in curve number with increasing rainfall is well documented (Bonelid, McCuen, and Jackson 1982; Hjlmfelt 1991; Hawkins 1993; Tshrintzis and Hamid 1997; Kottegoda, Natale, and Hamlet 2003), however, the effects of disconnection on the small storm curve number have not been as extensively analyzed.

For the most accurate runoff calculation, the curve number table derived for the water quality storm should be used when modeling these subdivisions in the future to evaluate disconnection management strategies. The synthesis of the new curve numbers, however, was very time intensive, and it is not feasible to recreate them for every subdivision in a watershed. Curve number modifications that reflect disconnection throughout a drainage area need to be made so that they can be applied to the land use and soil groupings that are given in TR-55 Table 2-2a. The effective impervious area (EIA) was used to relate the lot characteristics of the study areas with the lot sizes listed in TR-55 Table 2-2a.

A New Urban Curve Number Table

New curve numbers were calculated to substitute TR-55 Table 2-2a using the EIA/TIA calculated for each subdivision at the stages of disconnection (See Table 34 through Table 37). The EIA/TIA ratio was appropriate for use to relate disconnection to curve number for the different lot sizes listed in TR-55 because there was not much variation between EIA/TIA for different lot sizes at each stage of disconnection. Therefore, the EIA/TIA ratio was applied to the land use categories given in TR-55 to represent each stage of disconnection. The results of the new curve number tables were similar to what had been seen at the lot scale and subdivisions curve number analysis. The more pervious the hydrologic soil group, the lower the newly adjusted curve number was to represent the volume after disconnection, and the greater the difference between the original composite and new curve number (See Table 38). Also, as previously observed, the disconnection curve number approached the composite curve number as storm depth increased (Table 38). The use of these new curve number tables for the stages of disconnection was analyzed in a calibrated HEC-HMS watershed model for the water quality storm.

<u>Runoff Reduction through Disconnection on the Watershed Scale</u>

The effects of disconnection were not as apparent on the watershed scale as they were on the lot and subdivision scale. The curve numbers generated for the stages of disconnection were applied to the calibrated watershed model to examine if a watershed-wide disconnection strategy in the residential areas of the watershed could be effective in reducing the overall runoff volume. One of the primary obstacles to compare basin wide composite and disconnection curve numbers was that the HEC-HMS model required the input of a single curve number for each basin. The calculation of an area-weighted composite curve number was needed for each basin. However, analysis of the composite curve number on the lot scale determined this method to be inaccurate. The lot scale analysis also indicated that the magnitude of the difference between the curve numbers determined the extent of the under-prediction of the composite curve number at the small storm as did studies by Grove, Harbor, and Engel (1998) Therefore, since the difference between curve numbers in the urban watershed is, for the most part, not as drastic as the difference encountered between the pervious and impervious lot surfaces, the composite curve number may be able to represent the runoff at the basin scale.

The existing conditions basin curve numbers were closer to the calibrated curve numbers in Basins 8, 9, and 13 than the original base model composite curve number. It should also be noted that only residential curve numbers were changed in the model simulations. Highly impervious industrial and commercial areas were not altered, and no credit was given for areas already utilizing stormwater management. The calibrated curve number for Basin 12 was 91, which was higher than the existing conditions curve number of 86.6. Analysis of the land use of Basin 12 shows a higher percent of wetlands (16%) than in the other basins (Table 58). However, it still may be the case that this basin is more directly connected than the other basins, which may explain a greater runoff response.

Stage 1 disconnection reduced runoff volume from the existing conditions for all 13 basins within the watershed by a range of 4.8% to 56.8% (See Table 55). The mean percent volume reduction from each basin was 20.6%. The percent volume reduction achieved for Stage 2 disconnection was between 10.3% and 61.4% over the 13 basins. The mean percent reduction value was 26.9%. When stage 3 disconnection was applied to the residential areas throughout the watershed, there was a 26.2% to 83.0% reduction in runoff volume; mean value was 58.0% volume reduction. These results suggest that the disconnection curve numbers in the proposed tables could be used to predict the volume reductions possible with the implementation of disconnection throughout the basin. Future analysis of these tables using field runoff data is recommended.

CONCLUSIONS AND RECOMMENDATIONS

This study was successful in theoretically showing that disconnection can be a useful strategy for reducing runoff volumes from residential areas in urban watersheds during the water quality storm event. It was also determined that the composite method of calculating urban curve numbers under-estimates the overall runoff volume from residential areas. When the composite curve numbers listed in TR-55 are used to analyze small storms such as the water quality storm, runoff volumes can be misinterpreted as inconsequential events. However, management of the water quality storm is of great concern because even this small storm has the potential to contribute considerable amounts of runoff in an urban setting.

In order to reach a better understanding of the influence of directly connected impervious surfaces to small storm runoff, the SCS method was modified with the volume weighted method to include the runoff volume from directly connected impervious surfaces. At the lot scale analysis, the runoff generated using the composite curve number method was substantially lower than that predicted using the volume weighted method for all study areas during the water quality storm. At the larger 2-year storm, the difference was still substantial for the more pervious soils. However, at the 10- and 100 year storms, there was no difference between the volume calculated by the two methods. This conclusion is consistent with literature that states that the composite curve number method as used in TR-55 is less than accurate when applied to small storms.

Similar results were seen when the analysis was scaled up to the subdivision level to include streets. At the water quality storm, the difference between the composite and volume weighted methods was a function of the soil group and lot size with the greatest difference for the subdivisions with the smallest lots and the most pervious soils. The difference is attributed to the fact that the composite curve number method does not mathematically account for the influence of the directly connected impervious surfaces on overall runoff volume.

Removal of the directly connected impervious areas from the runoff equations was used to analyze the future conditions of the subdivisions if the rooftops, driveways, and/or streets were disconnected. The effectiveness of disconnection in mitigating runoff volumes during the small storm relied on the infiltration capacity of the pervious area. In other words, pervious areas with low curve numbers are ideal for disconnection, predominately hydrologic soil groups A and B. Disconnection was less effective for the larger storms. The greatest reduction of runoff volume was achieved when the greatest amount of effective impervious area was disconnected from the direct runoff. In all scenarios (lot, subdivision, and watershed scale) disconnection was the most effective at reducing the runoff from the water quality storm when runoff was routed over the most pervious lawn areas (B soils). The greatest obstacle to achieve 100% disconnection is the large amount of directly connected street area in residential watersheds.

The calculated effectiveness of disconnection in reducing runoff volume varied based on the scale of analysis. Disconnection was the most effective at reducing total runoff volume at the lot scale. 100% rooftop disconnection alone was calculated to prevent an average of 34.6% of the total runoff volume from the water quality storm entering the drainage network. When rooftops and driveways were disconnected the mean volume reduction was 58.5%. This volume reduction on the lot scale is important because it quantifies the impact that an individual homeowner can have on stormwater runoff. It does not however, describe the entire runoff picture because the effect of streets on total runoff is not included.

Since streets contributed nearly half of the stormwater runoff from the subdivisions, the effect of 100% rooftop disconnection diminished when examined at the subdivision scale.

Rooftop disconnection could reduce an average of 11.5% of the total runoff volume at the water quality storm. When all impervious surfaces were disconnected, the average reduction was a 57.3% reduction. It should also be noted that 95% of runoff volume from existing conditions could be achieved with 100% disconnection in the subdivision with B soils. Approximately half of the urban runoff could be controlled through disconnection in the areas with C soils.

The results seen on the lot and subdivision scale were applied to the watershed scale to create new composite curve number tables for the small storms. Through the application of the EIA/TIA ratio determined for each stage of disconnection in the field, the runoff volume for a disconnection scenario can be represented by a curve number. These runoff conditions were used to create the new composite curve number table for the disconnection scenarios, which included curve numbers based on lot size, hydrologic soil group, degree of disconnection, and storm depth. When these new curve numbers were used in a calibrated HEC-HMS model to simulate disconnection throughout the watershed, the curve numbers calculated for the basins that represented the existing watershed conditions (a baseline disconnection of 50% of all rooftops) were a better representation of the runoff volumes generated by the calibrated model than were the composite curve numbers. This suggests that the volume weighted method was not 100% correct in predicting runoff volume, however, since the composite curve number under-predicted volume, better approximations of runoff were achieved when the curve numbers were adjusted.

Watershed scale disconnection was simulated with the assumption that all residential areas in the watershed were utilizing a uniform stage of disconnection. This is not a realistic situation, however, the simulations could be used to describe management goals. 100%

rooftop disconnection reduced an average of 20.6% of the overall runoff volume from each basin. If all impervious surface was disconnected in each residential area, an average of 58.0% of the basin runoff volume could be reduced. This calculated 58% volume reduction is similar to what is expected at the smaller scales, which confirms the application of the disconnection method to the watershed scale. Although watershed scale disconnection is comparable to the percent reductions at the subdivision scale, it is unlikely that the same level of disconnection could be achieved throughout the watershed. However, the predicted volume reduction for each basin could be used to target basins that would have the greatest benefit from a disconnection management strategy. For example, Stage 3 disconnection could reduce the overall runoff volume from Basin 4 by 10.29 ac-ft which is a 68.9% reduction in total runoff volume. In order to reach this goal, subdivisions within Basin 4 would be pinpointed for a disconnection plan. The street runoff from the subdivisions within Basin 4 would need to be managed in addition to rooftop and driveway disconnection in order to achieve the predicted basin-wide reduction in volume.

Subdivision scale disconnection would most likely have the greatest positive benefit to the localized stream channels. The subdivisions with 1-acre lots and smaller that were analyzed in the disconnection study drained to outfalls that discharged directly to the stream channel. Since all the stormwater runoff from the subdivisions was discharged to the stream through these single points, high flows could be expected from small storms, and erosion is observed around and downstream of the outfall pipes. By reducing or even eliminating the runoff that reaches the stream channel via these pipes during small storm events with a disconnection program for each subdivision, less stress would be placed on the stream channel and downstream erosion could be minimized. As described by Booth and Jackson

(1997), at 10% EIA the 2-year discharge is the runoff threshold that begins to undermine channel stability. If stormwater management strategies such as disconnection can reduce the magnitude and occurrence of these smaller events, bank erosion and other adverse effects on water quality from the small storm may be reduced.

Disconnection could be a useful tool in drafting future stormwater management plans. Future research into the application of the new disconnection curve number tables for prediction of stormwater runoff from the small storm would be beneficial to devising solutions to water quality and quantity issues associated with small storms. If the small storm and channel forming runoff events can be completely eliminated, localized streambank erosion may be able to be controlled. Field studies to verify these curve number tables in the Pompeston Creek Watershed would aid in the understanding of the application of the SCS method to urban watersheds. Also, the disconnection curve numbers generated on the subdivision scale can be used to evaluate the effectiveness of disconnection in reducing streambank erosion, channel widening, and subsequent sedimentation for a hypothetical receiving stream.

It is unknown whether wetlands played a role in the contribution of runoff on the watershed scale. The observed runoff from Basin 12 was greater that that was expected. Either the wetlands contributed a fraction of runoff, or Basin 12 was more directly connected than the other observed basins. Further investigation into the simulation of wetlands in the SCS method may help to determine how to manage basins with characteristics similar to Basin 12.

Time of concentration was not analyzed in this study. Disconnection not only reduces the over volume by promoting infiltration as determined in this study, it can also lengthen the time of concentration. As the EIA is reduced, the time it takes for stormwater to reach the receiving stream is extended. Future studies could analyze this aspect of disconnection and examine how peak flows are reduced when disconnection is implemented.

While disconnection is intended to be a simple management strategy to reduce the runoff volume from residential areas, it is not intended to be the only method for reducing urban runoff. Future studies could examine how a combination of disconnection and rain gardens, or disconnection and tree boxes could provide maximum reduction at the water quality storm. Also, infiltration rates and groundwater recharge were not considered in this study, and further research could include insight as to what benefit to recharge and baseflow could be gained through disconnection.

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APPENDIX

Table 2-2a Runoff curve numbers for urban areas [1/]

Cover description: Cover type and hydrologic condition	Average percent impervious area [2/]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3/]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4/]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5/]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.
2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Figure 2-3 Composite CN with connected impervious area.

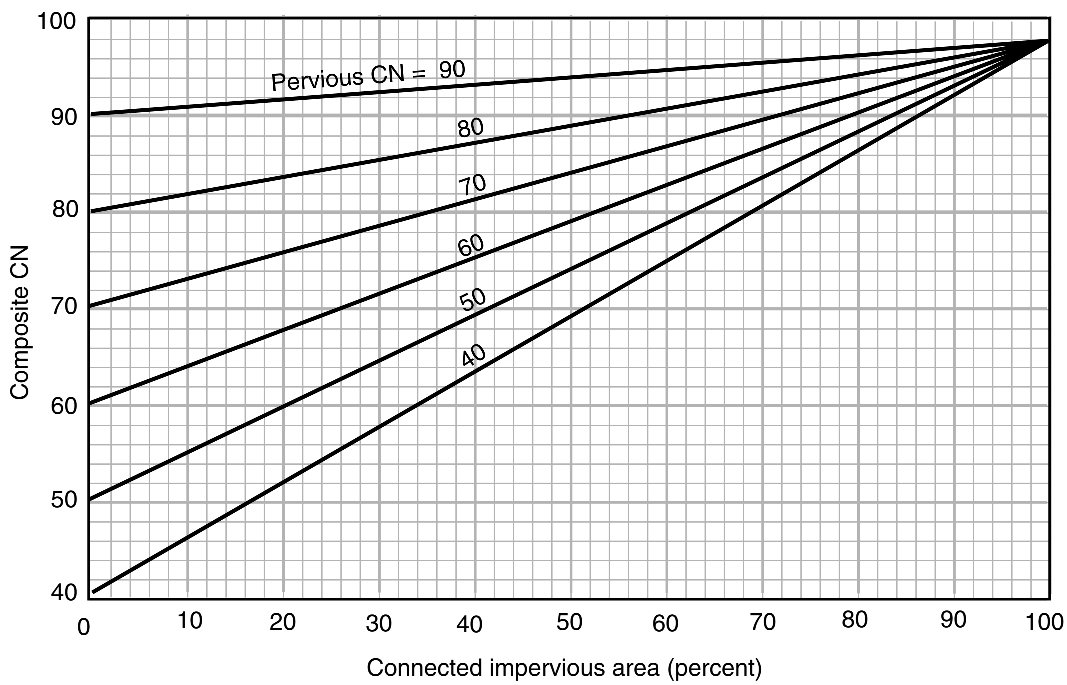

Figure 2-4 Composite CN with unconnected impervious areas and total impervious area less than 30%