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Mitigating Climate Change Induced Increases in Rainfall Intesity and Frequency Using Lid Stormwater Techniques

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MITIGATING CLIMATE CHANGE INDUCED INCREASES IN RAINFALL INTENSITY AND FREQUENCY USING LID STORMWATER TECHNIQUES

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by
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Abstract

A warming climate leads to a moister atmosphere and more rapid hydrologic cycle. As such, many parts of the country are predicted to experience more total rainfall per year and more frequent extreme rainfall events. Most regions of the country have stormwater systems designed to a standard that matches outflow to pre-development values for specified return period storms. Increases in these return period storm depths, as predicted by many global climate models, will stress existing stormwater infrastructure. This thesis examines two issues related to the impact of climate change on stormwater management in the state of South Carolina, namely how will rainfall patterns change over the remainder of the century and how can our approach to stormwater management system design adapt to these changing conditions.

Rainfall simulations from 134 realizations of 21 global climate models were analyzed across the state of South Carolina through 2099. Results show that there will be increases in both annual total rainfall (ATR) and 24 hour design storm depth for a range of return period storms. Across South Carolina, ATR is predicted to increase by approximately 1.5-3.3 inches over the forecast period while the 100 year design storm depth is predicted to increase by 0.5-1.2 inches depending on location. However there are significant regional variations with the Savannah River Basin experiencing smaller increases in ATR compared to the rest of the state.

The impact of these changes in rainfall patterns on the outflow characteristics of various land developments was examined through a series of case studies. Hydrologic

models were developed for three different development sizes at three locations in the state to evaluate the effectiveness of Low Impact Development (LID) technologies in site design. LID usage increases in onsite retention and disposal which significantly reduces total runoff and can also reduce peak runoff rate. Depending on the location-specific infiltration, developments with LID usage can reduce runoff volume to below predevelopment values and significantly reduce peak outflow. The use of LID can also reduce the required pond size by up to 70% allowing for more space for land development or water quality control structures.

Dedication

Dedicated to my family, friends, and most of all, my wife to be, Anna. Without all of the mental, emotional, and spiritual support, I would not have completed.

"I can do all things through Christ which strengtheneth me." Phillipains 4:13

Acknowledgements

I would like to thank Dr. Kaye and Dr. Martin for the constant mentoring and support throughout my time as a graduate student. Completing a MS in under a year is not an easy task and I owe it to them for guidance through the countless edits and comments for this achievement. Dr. Martin taught me how to use Matlab and ArcGIS which were integral to the study. The final member of the committee, Dr. Mishra, gave insight to statistical methods used in this paper. Graduate students Nasim Chowdhury and Ali Tohidi helped with crucial components of the research. Finally I would like to thank the South Carolina Water Resources Commission for funding to make this research possible.

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Chapter 1 Introduction

Natural landscapes are constantly changing and new developments are added daily. Soils are covered by impervious surfaces and plant life is removed. Precipitation still occurs so measures must be taken as the increase in impervious surfaces and decrease in the natural cover combine to increase the peak flowrate and amount of water flowing from a given site.

The original method of stormwater design was to move the water off-site as quickly as possible. This was achieved through the use of conveyance systems-inlets, pipes, and outlets that discharged the runoff off-site. This fast moving water not only increased erosion and caused flash flood type events, it also carried more pollutants downstream than before development occurred. This human impact on the urban environment disrupted the natural hydrologic cycle from the previously existing state of runoff moving downstream to receiving waters by increasing the amount of water and how quickly it moves downstream.

After several large environmental disasters the Clean Water Act of 1972 was passed and the Environmental Protection Agency (EPA) began the process of protecting surface waters, most which are impacted by urban runoff. With over 60 million acres of developed land, and that increasing by an estimated 6000 acres of open space lost to land development each day (USDA 2007), the EPA has a large responsibility. Best Management Practices (BMP) was a term coined by the EPA in the 1977 Clean Water

Act and is used to refer to methods to improve water quality as well as reduce peak outflow and total runoff volume from a site.

State regulatory committees enforce limitations on runoff from a site though the extent of each state's limits can vary greatly. For example, the South Carolina Department of Health and Environmental Control's (SCDHEC) stormwater regulations focus on peak flows and not total runoff.

1.1 Global Climate Change

The increasing concentration of greenhouse gasses in the atmosphere are trapping heat and altering the earth's climate. These changes are modeled through the use of global circulation models (GCMs) which are numerical models that represent physical processes in the atmosphere, ocean, cryosphere and land surfaces and are the most accurate models in predicting climate variables in response to different greenhouse gas concentrations (IPCC, 2013). A three-dimensional grid with multiple layers over the entire globe (with a grid sizing varying from 250 to 600 kilometers) is used to analyze the time varying climate. GCMs vary in some of their inputs and the significance of other inputs. Established in 1995 by the World Climate Research Programme (WCRP), GCMs were used for the Couple Model Intercomparison Projects (CMIP) that used multiple GCMs to validate and compare results. CMIP5 is the fifth and most recent phase of the project that coordinates climate model experiments developed by 20 climate modeling groups from around the world. Previous CMIP issues involved modeling the carbon cycle and clouds are not present in CMIP5. The models were used to compute climate variables for the previous fifty years to form a baseline comparison with historical data to check for accuracy and to quantify any differences between models. CMIP5 also includes projections of future climate change to the end of the century (Taylor et. al, 2009).

GCMs model the earth on a fairly coarse grid. Therefore, to apply these models' output to relatively small entities within each grid the model results must be downscaled for a regionalization that is more accurate and useful for cities, government planning, and impact of precipitation change on a sensitive ecosystem (Weltzin, et. al 2002).

1.2 Stormwater management and LID

With a predicted increase in precipitation over the next century, many areas are at risk for flooding. Flooding can have devastating economic and social impacts on a community (König, et. al, 2002) and can occur routinely for an outdated or under-designed storm drainage system. Flooding is a worldwide problem and is not focused on a certain region or climate. Although flooding in some areas is a recurring problem based on the season, in many locations flooding is attributed to a high density of buildings, parking lots and other non-permeable surfaces that do not allow a portion of the water to be absorbed into the soil.

In today's standards of design, runoff from a storm is accounted for and the appropriate measures are taken to capture and responsibly drain water from an area so as to avoid flooding. However many cities were built decades and centuries ago with a mindset of taking full advantage of all available space and gave little consideration to the prevention of flooding. Throughout the past 200 hundred years there have been significant and notable development booms across the United States of America. Major cities have been growing in population and, although there are no longer issues with slums, developing adequate stormwater infrastructure has been lagging behind. According to the American Society of Civil Engineers (ASCE) 2013 Report Card for American Infrastructure the US has an overall rating of D+ (ASCE). With the most recent, wide-scale boom being almost 50 years ago a lot of the roads, bridge and storm sewer systems are approaching the end of their design life or do not meet the capacity required.

Over the past two decades extensive research has gone into BMP/LID usage as their use becomes more popular. BMP practices that focus on infiltration and percolation practices can greatly improve the downstream water quality. BMP/LID techniques are typically implemented with water quality in mind. However, increasing onsite retention and disposal will decrease total runoff and peak discharge from the site. This benefit is largely ignored in current design practice. In this thesis the potential of LID technologies to reduce runoff and to be more resilient to changes in rainfall patterns is investigated through a series of case studies.

Chapter 2 Future Precipitation Analysis

2.1 Introduction

Over the last century the average global temperature has risen 0.85 degrees Celsius (IPCC, 2014). Forecasting climate changes is important for preparing societies for possible impacts to food supply, water resources, infrastructure, ecosystems, and even human health. Temperature changes are only one aspect of the predicted changes the Earth will experience. Other changes include precipitation patterns and intensities, ice and snow cover, sea level, and ocean acidity. In 2001, the IPCC published strong conclusions in response to evidence of global climate change (IPCC, 2001). The 1990's were reported to be the warmest decade, for the northern hemisphere, since adequate record keeping (IPCC, 2001). Trends in precipitation are increasing slightly, about 1% per ten years, and the number of severe precipitation events is also increasing (IPCC, 2001). The IPCC concluded that the warming that is being observed in the last century is not natural, based on the inaccuracy of models that attempt to predict historical trends based on natural radiation, while models that include increases in atmospheric gas concentrations predict the historical data quite well (IPCC, 2001).

The IPCC made its conclusions based upon a large variety of research and data. Specific to the United States, there has been trend analysis done for precipitation and temperature for major urban areas. Mishra and Lettenmaier (2011) found that there were significant increases in extreme precipitation events in 30% of urban areas from 1950-

2009. Martinez et al. (2012) found increasing trends in temperature and decreasing trends for precipitation for the state of Florida for a similar time period.

In general, climate change models predict a warmer and moister atmosphere resulting in a more rapid hydrologic cycle and more extreme rainfall events. Stormwater systems, some of which are already overloaded, will be stressed even further with increased runoff. As a result water quality will decrease as sediment runoff and flooding will increase.

Current South Carolina stormwater regulations (DHEC, 2002), only regulate peak flows and not total runoff. As such, traditional stormwater designs have reduced infiltration and increased total runoff when compared to original site hydrology. Developing sites often requires significant downstream storm sewer infrastructure. With increased rainfall due to climate change, these design weaknesses will cause a disproportionate amount of the additional rainfall to directly become runoff. Responsible stormwater management is required to maintain the quality of surface water in a climate that will exhibit increased frequency and intensity of rainfall over time.

This paper presents the results of a detailed analysis of rainfall forecasts based on Global Climate Model (GCM) data archived through the Climate Model Inter-comparison Project – 5 (CMIP5). The data is analyzed to examine the change in annual total rainfall (ATR) and 2, 10, 25, 50 and 100 year 24 hour storm depths between now and the end of the century.

Engineers and regulators will better understand the risk a changing climate will present to stormwater infrastructure as a result of this analysis. That is particularly true for state agencies with regulatory responsibilities for defining stormwater design events such as SC-DHEC and SC-DOT.

The remainder of the chapter is structured as follows. The project overview summarizes the main goals of the project and pertinent literature. The sources of data used and the analysis techniques are described in the methods section. The results section presents forecasts for the ATR and 2, 10, 25, 50, and 100 year 24 hour storm depths for the entire state of South Carolina. Conclusions and suggestions for future work are presented in the discussion section.

2.2 Project Overview

As an increase of rainfall intensity and frequency is expected, the responsibility of designing stormwater systems to be effective for their entire design life lies with the designing engineer. However, in order to effectively plan for future rainfall patterns, data on expected changes is required. GCM's typically produce low spatial resolution data that must be statistically downscaled for the purposes of local hydrologic trend analysis. There are a number of approaches to downscaling including Bias Corrected Constructed Analogs (BCCA) and Bias Correction and Spatial Disaggregation (BCSD) (Ahmed et al. 2013). The choice of downscaling technique depends on the application. Downscaling GCMs using Bias Corrected Constructed Analogs (BCCA) provide a higher temporal and spatial resolution (Barsugli, et al, 2009, Maurer & Hidalgo, 2008) and improved estimates of precipitation compared to other downscaling methods (Brown & Wilby, 2012). Using multiple GCMs removes the bias that a certain model may have and improves the estimation of variability that is typically under estimated by using a single downscaled data set (Brekke, et al., 2008). This study uses projected rainfall data from 134 realizations of GCMs with daily temporal resolution and $1/8^{o}$ degree spatial resolution to explore long term trends in rainfall in South Carolina. These data sets include GCM model runs for all four Representative Concentration Pathways (RCPs). That is, they include model runs for a range of different long term atmospheric CO_2 concentration levels. The choice of appropriate RCP would require a prediction of future public policy which is beyond the scope of this paper. As such, all four data sets were lumped together. The results, therefore, represent an average set of predictions of future rainfall patterns. This approach may under estimate the potential changes in rainfall patterns if global CO_2 emissions are not curbed.

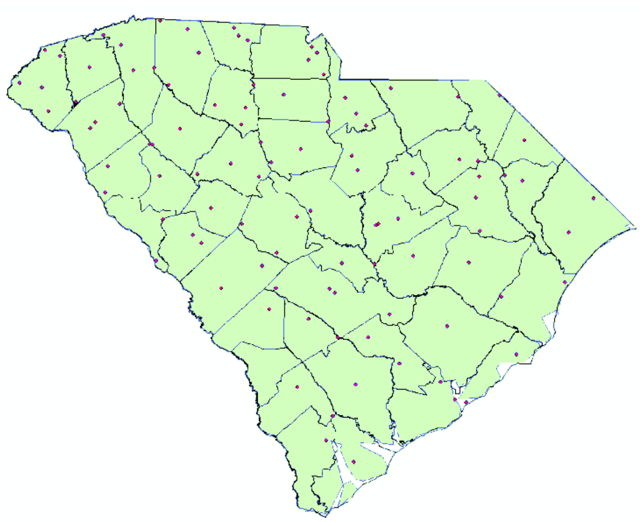

Figure 2-1 NOAA weather station locations in South Carolina for which observed data was collected and downscaled GCM data was analyzed.

2.3 Methods

Downscaled GCM data was analyzed for each of the locations of NOAA precipitation measuring stations, **Error! Reference source not found.**, so that the projected rainfall data could be directly compared to historical data and posted 24 hour storm depths. Historical rainfall data is available for all of the stations through the National Climatic Data Center (NCDC) run by NOAA. While breaks in the data (no data recorded) exist in the data sets they only exist for relatively short periods and are not accounted for in the analysis. The average data set for the historical data from 1950-1999 contained 41.6 years of data. The list of stations was edited to remove duplicate stations (occurring for stations that measured both hourly and daily values), stations located outside the projection grid (occurring for some coastal stations), or stations with region information not specified by NOAA (Bonnin, et al., 2006). BCCA downscaled CMIP5 daily hydrologic projections were downloaded for each station from an online archive (U.S. Department of Interior, 2014). The projections used 21 climate models with various combinations of four representative concentration pathways (RCPs) and different realizations creating a total of 134 different daily rainfall projections for a period of record (POR) from 2015-2099.

A precipitation frequency analysis had already been performed on the historical data by NOAA and was the computational method behind the Precipitation Frequency Data Server (PFDS), which gives the storm depths for different return periods and durations. The NOAA Atlas 14, Volume 2 is based on data from 13 states and covers precipitation frequency estimates for event durations of 5 minutes through 60 days at recurrence intervals of 1-year through 1,000 years. The method is based on converting annual maximum data to partial duration data series and then further "personalizing" by location through regionalization. The analysis herein focused on 24 hour storm depths due to their role in stormwater design regulations.

After importing the data for each station, the maximum daily values were converted to 24-hour maximum values using

$$P_{24max} = P_{max} * t_{24} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(1)}$$

where $t_{24}=1.13$ is the ratio between average daily maxima and average 24-hour maxima. This ratio is empirically derived from 86 stations that had 15 years of concurrent data. Comparing the conversion factors to past NOAA volumes and other studies finds that the conversion value is comparable if not the same. The 24 hour annual maximum depth data set was then converted to partial duration data series using

$$P_{AAmax} = P_{24max} * {}^{T_{AMS}}/_{T_{PDS}} \ldots\ldots\ldots\ldots\ldots\ldots \text{(2)}$$

The parameter T_{AMS}/T_{PDS} is equal to 1.58 and represents the frequency ratio between an annual maximum series and a partial duration series. This ratio allows for multiple large storms in a single year be considered in the final value such as occurred in Clemson, SC in 2013. The partial duration series was averaged and converted into a set of 24 hour storm depths of specified return period using

$$P_{n\_yr} = \bar{P}_{AAmax} * RGF_n \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3)}$$

where n is the return period in years. The Regional Growth Factor (RGF) for each return period depends on the location of the rain gauge and is given in the NOAA Atlas. Distribution of the regions for the RGF can be seen in Figure 2-2. For example, since the station in Clemson, SC (Station ID 38-1770) is assigned to NOAA Region 12, its RGFs for the 2, 5, 10, 25, 50, and 100 year storms are 0.907, 1.196, 1.429,1.801, 2.148, and 2.272 respectively (Bonin, et al., 2006). Using the same frequency analysis technique employed by NOAA allows for direct comparison of the GCM precipitation frequency values to the precipitation frequency values reported by NOAA based on historical rainfall data.

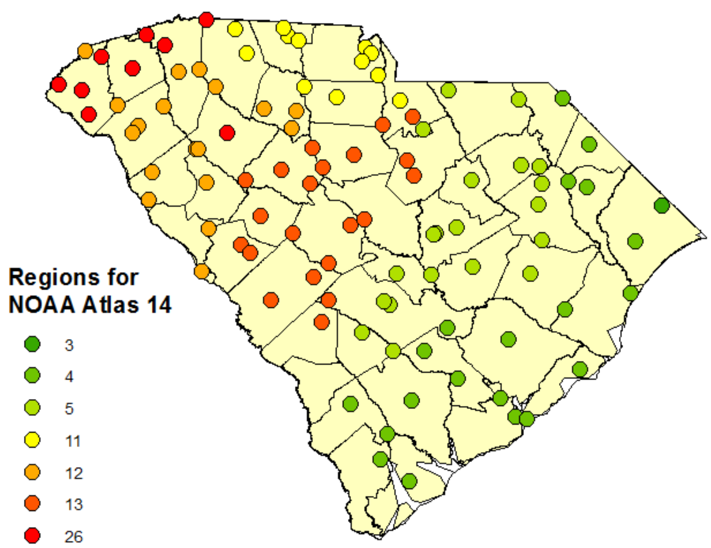

Figure 2-2 Regions from NOAA Atlas 14 for the South Carolina

2.4 Results

Results are presented for changes in Annual Total Rainfall (ATR) and for the 24 hour storm depth for 2, 5, 10, 25, 50, and 100 year return period storms. Because much of the data presented is location specific, Clemson, SC was chosen as a case study and is represented in many of the figures herein to illustrate a typical location. There are also figures that summarize this data for the entire state of South Carolina.

2.4.1 Changes in annual total rainfall

For each NOAA precipitation gauge location the daily time series of historical rainfall data and each downscaled GCM data set was converted into an ATR time series. A plot of the 134 ATR time series from 2015-2099 along with the historical recorded data from 1948-2011 for Clemson, SC are shown in Figure 2-3. The data shows significant year to year variation in the historical recorded data and a similar level of variation across the different GCM data sets presented. There is also a steady increase in the GCM predicted ATR over time. This is seen more clearly in Figure 2-4 which shows the mean and standard deviation of the historical data along with the yearly mean and standard deviation from the 134 GCM data sets. Note that there is a slight jump in average ATR from the historical mean to the start of the GCM time series. However, this discontinuity is well within the range of variability observed in both the historical and GCM projected data.

The downscaled GCM data shows a clear increase in the ATR over time. However, a histogram of the ATR from 2089-2099 for each of the 134 GCMs shows only a slight increase in mean ATR compared to historical records (see Figure 2-5). To verify that the increase is statistically significant a T-test was performed to compare the historical data with the GCM data for the last eleven years of the century (2089-2099).

The T-test showed that the difference in the means was statistically significant with a 97.5% confidence interval

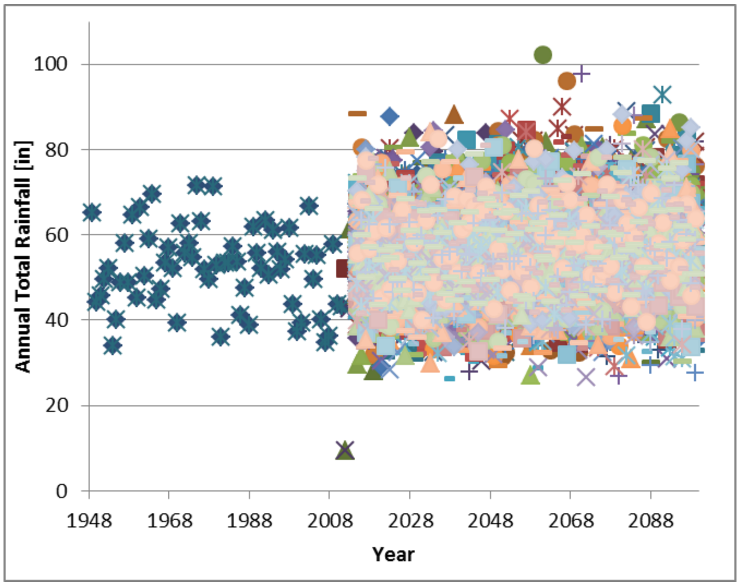

Figure 2-3 NOAA observed historical annual total rainfall (1948-2011) and predicted annual total rainfall (2015-2099) from 134 different realizations of GCMs for Clemson, SC.

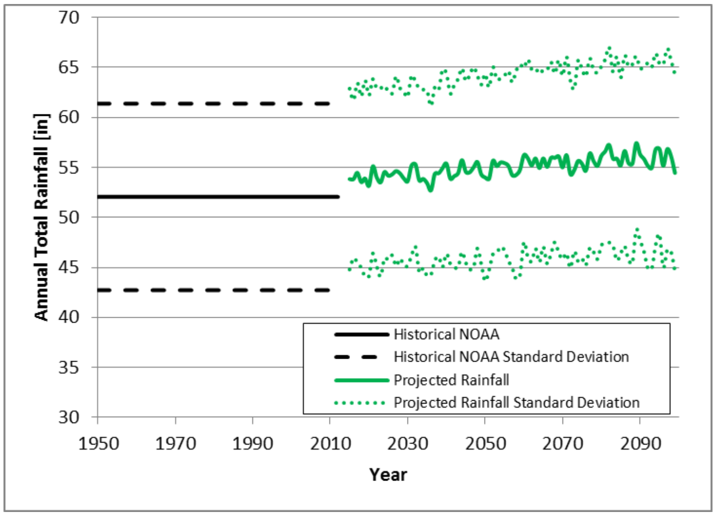

Figure 2-4 Averaged ATR for Clemson, SC based on NOAA observed data (1950-2011) and projected rainfall for 2015-2099 based on 134 realizations of GCMs

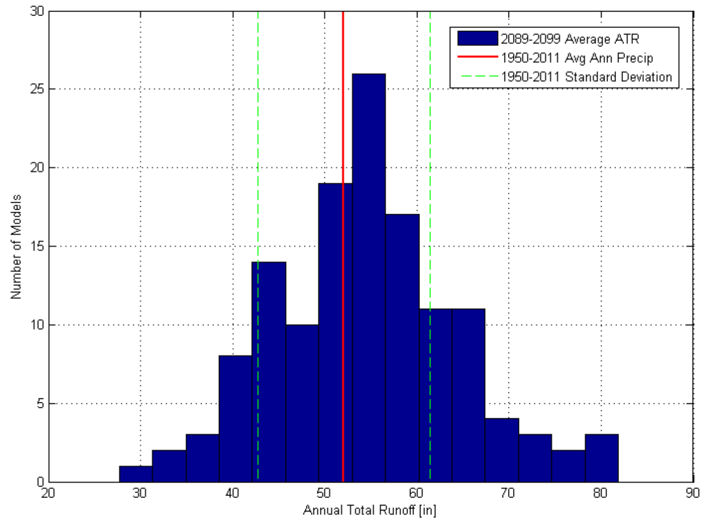

Figure 2-5: Histogram of the average ATR for Clemson, SC from 2089 to 2099 based on 134 downscaled realizations of GCM data sets. The vertical line represents the current average ATR.

The data and analysis above was for a single location, Clemson, SC. Similar analysis was conducted for each of the precipitation gauge locations throughout the state. All locations showed an increase in ATR between 2015 and the end of the century. However, the net increase in ATR varied across the state. There was also an offset between the predicted 2015 mean ATR based on 134 GCM data sets and the historical record. At each gauge location the historical mean and standard deviation in ATR was compared to the mean and standard deviation of the ATR for 2015 based on all 134 GCM realizations. These data are plotted in Figure 2-6 and Figure 2-7. Figure 2-6 shows a scatter plot of historical mean ATR versus 2015 GCM mean ATR. The offset between the historical mean and the 2015 mean varies by location though the 2015 GCM mean ATR is almost always larger than the historical mean ATR. This is would be expected for a climate with increasing mean ATR as the historical record would average over a non-stationary data set and would, therefore underestimate the current mean ATR. Figure 2-7 shows the standard deviation in the historical ATR versus the 2015 GCM ATR standard deviation. Again the difference varies with location though in this case the standard deviation is not consistently higher or lower for the GCM data. The historical data shows a greater range of standard deviations compared to the GCM data, though this is likely due to the smaller number of data points in the historical data sets used in this analysis (average 41 years of data, 14 year standard deviation) compared to the 134 data points for the 2015 GCM ATR standard deviation.

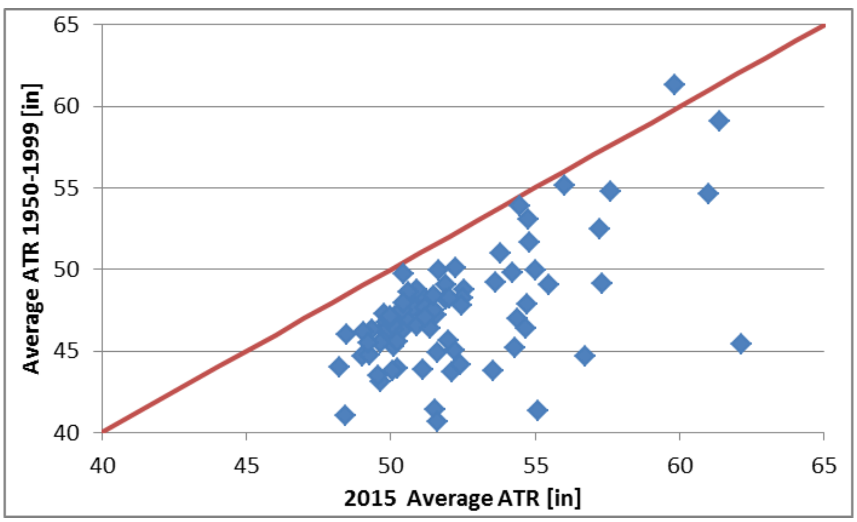

Figure 2-6. Scatter plot of 2015 GCM mean ATR versus historical average ATR for 1950-1999 with the red line showing exact agreement. Each data point represents a station.

Given the variation in both mean offset and predicted standard deviation it might be somewhat misleading to simply present the difference between the historical mean and the mean averaged over the later years of the century. Instead, we present data for the projected change in ATR based on a linear curve fit through the mean ATR for the GCM

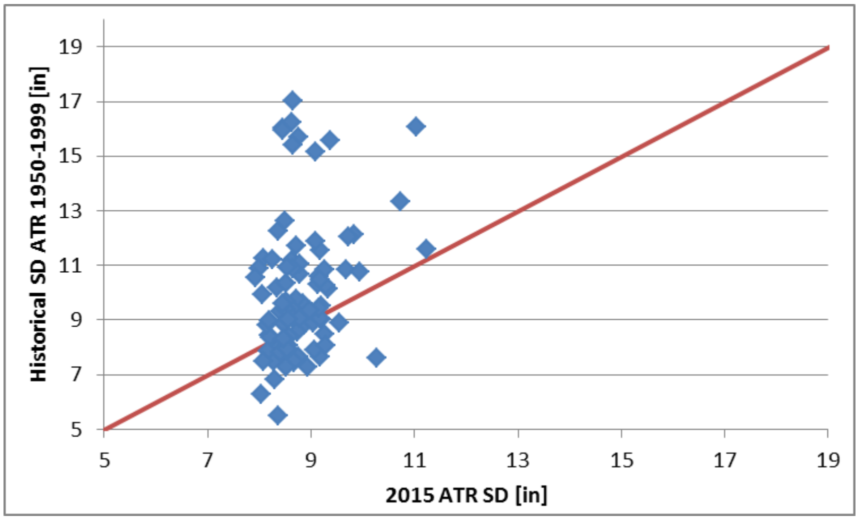

Figure 2-7. Scatter plot of 2015 GCM SD of ATR versus historical average ATR with the red line showing exact agreement. Each data point represents a station.

data from 2015-2099. A straight line was fit through the mean GCM ATR for each location. The slope of this line (in in/year) was then multiplied by 84 years (the GCM POR) to give a projected change in ATR over the remainder of the century. The data from each station was then entered into ArcGIS by ESRI where the geographic data information was interpolated using a tensioned spline method to create contour surfaces. A tension spline interpolation results on a surface that is less smooth but more closely restrained by the inputted data. This contour plot is presented in Figure 2-8.

Figure 2-8 shows significant variation in ATR change from 2.3 in for certain parts of the Savannah River basin to over 3.8 in in the coastal region, especially Charleston and Horry County. Much of the upstate and the length of the Savannah River Basin are all predicted to see lower levels of ATR increase compared to the rest of the state. The exception to this is the northern section of the border between Greenville and Spartanburg counties which will see ATR increases of around 4 in.

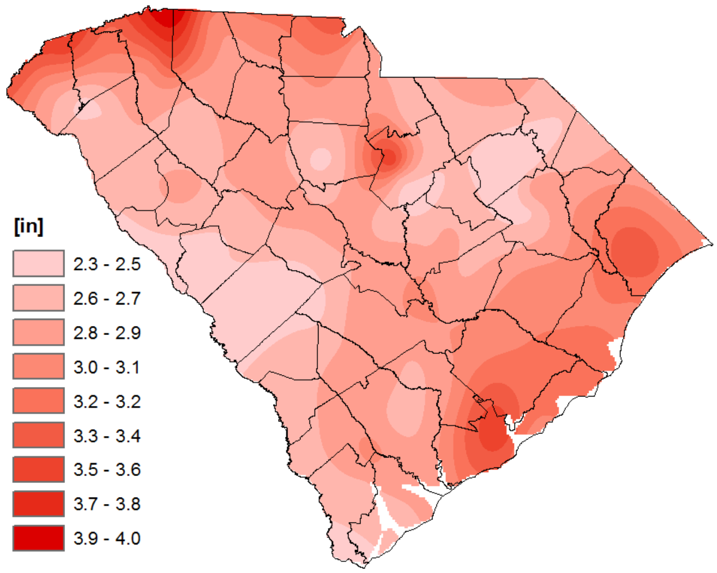

Figure 2-8 GCM simulations of change in average ATR (inches) over the forecast period (2015-2099) using the ATR trendline slope

2.4.2 Changes in 24 hour design storm depths

Stormwater design in South Carolina is generally based on the 2, 10, and 100 year return period storms (DHEC 2002). Therefore, it is important to see how these design storm depths change over time, especially in comparison to the current NOAA return period data. In a changing climate the idea of a return period storm is not clearly defined. However, given 134 annual time series per year it is possible to get reasonable estimates of 2, 5, 10, 25, 50, and 100 year return period 24 hour storm depths for each year in the GCM POR and analyze how they change over time. A sample plot of the variation in storm depth for Clemson, SC is shown in Figure 2-9 along with the current NOAA values for the same return periods.

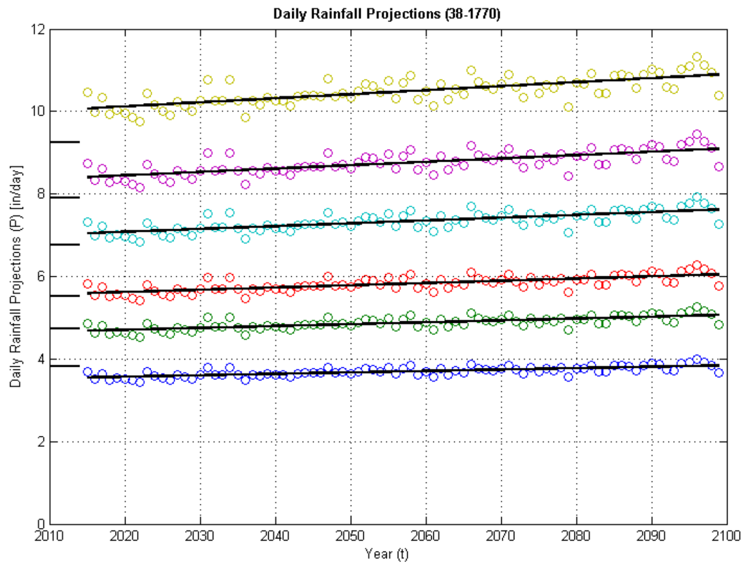

Figure 2-9 Forecast of storm depths versus year based on 134 downscaled GCM data sets. 100, 50, 25, 10, 5, and 2 year storm depth are shown from top in descending order. The horizontal lines on the y-axis show the current NOAA value for the respective storm depth.

As with the ATR, the 24 storm depths are also seen to increase over time for each return period. However, there is also a difference between the historical record and the 2015 GCM projection for the each return period storm. In this case, the 2015 GCM data is lower than the NOAA value for the 2 year storm and above the NOAA value for the 100 year storm. In general the 2015 GCM projections for the 100 year storm were higher than current NOAA values though not always. Figure 2-10 shows a histogram of this difference for the 101 precipitation gauges analyzed as part of this study. The vast majority of locations have a difference of less than 1 in though some exhibit differences of up to 4 in. Twenty stations had 2015 GCM 100 year 24 hour storm depths lower than the current NOAA data.

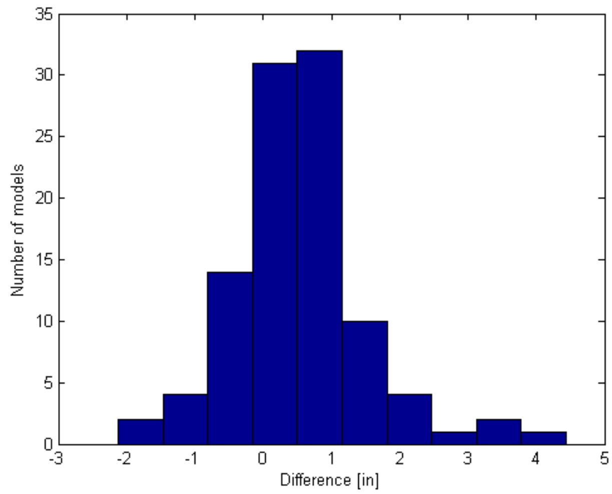

Figure 2-10. Histogram of the difference between the linear trend line value for the 2015 GCM 100 year storm depth and the current 100 year storm depth reported by NOAA for all 101 stations analyzed.

Regardless of the offset between 2015 GCM predictions and current NOAA data there is a clear upward trend in all six return period storm depths. Therefore, as with the ATR data, the projected change in depth is reported. A line was fit through the yearly return period depths for each return period and each precipitation gauge. The slope of these lines was then used to calculate the projected increase in storm depth by the end of the century across the state. As with the mean ATR, there is significant uncertainty in the calculated values of 24 hour storm depth for a given return period. As such, NOAA reports the calculated depth and the depths at the extremes of the 90% confidence interval. For each rain gauge location, the projected year at which the GCM calculated storm depth exceeded the upper range of the 90% confidence interval for the historical data was calculated. Histograms of this year for each of the calculated return period storms are shown in Figure 2-11. The data shows that there is a more significant change in the longer return period storms. For example, most locations will not see the 2-year storm depth exceed the current NOAA 90% confidence interval value until well into the next century whereas most locations will have 100-year storm depths that exceed the current 90% confidence interval in the next few years. The year in which the GCM trendline exceeds the current 90% confidence interval is sometimes greatly outside the simulation period of record and should, therefore, not be taken as predictive. However, the data clearly shows that the larger depth (longer return period) storms will exceed the current 90% confidence interval much sooner than smaller depth storms.

The linear fits for each location and each return period were used to create contour plots of the total change in depth predicted over the GCM POR. The slope of each line was multiplied by 84 (the number of years in the POR) to calculate a change in depth. This approach is the same as that used for calculating changes in mean ATR over the GCM POR and ignores any offset between the 2015 GCM data and historical data. This offset is discussed below. A contour plot of the projected depth change for each return period storm is shown in Figure 2-12. The GCM data projects that the 100 year storm depth will increase by between 0.5 in and 1.2 in over the next 84 years whereas the 2-year storm depths only increase by between 0.2 and 0.5 in. As with the ATR data there is significant variation across the state with the largest increases in similar regions to those that were predicated to have the largest increase in ATR.

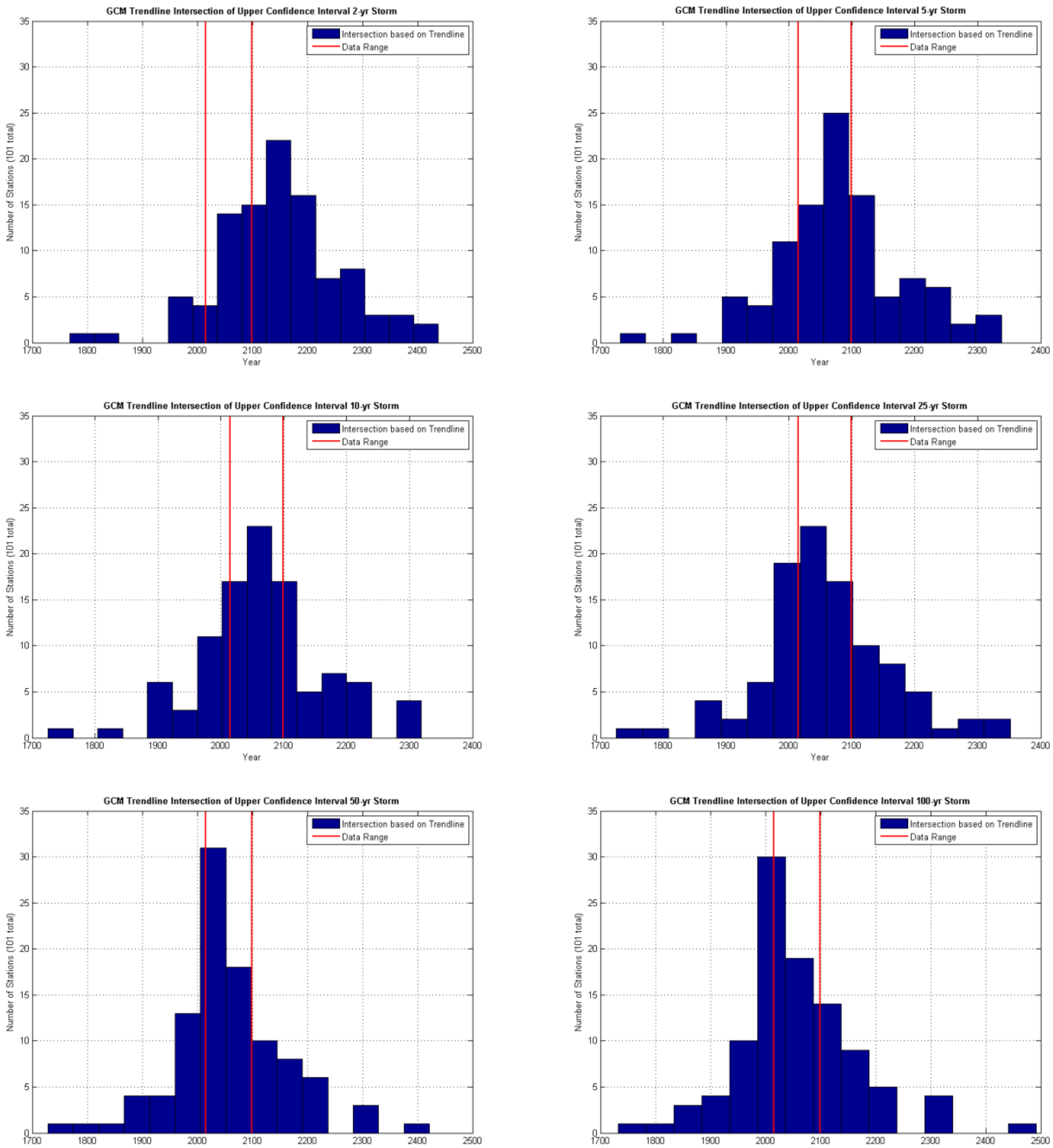

Figure 2-11. Histograms of the year in which the 24 hours storm depth will exceed the current NOAA 90% confidence interval upper limit using the GCM trendline equation. Reading from top and left to right, 2, 5, 10, 25, 50, and 100 year return period storms. The vertical red lines represent the GCM simulation POR.

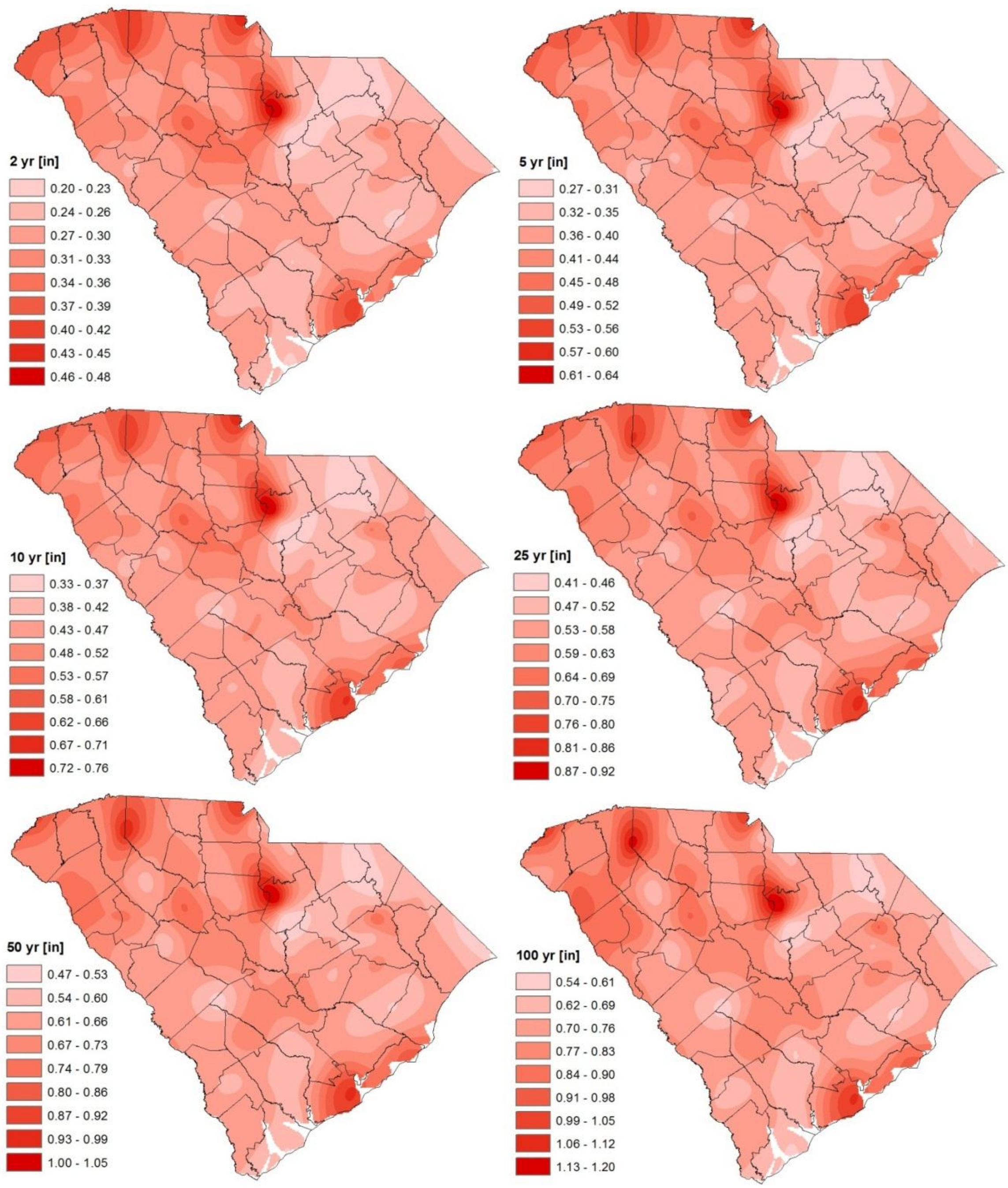

Figure 2-12 Contour plot of the GCM prediction of the change in 24 hour design storm depth (inches) over the forecast period. Reading from top and left to right, 2, 5, 10, 25, 50, and 100 year return period storms.

One possible explanation for the 2015 GCM 100 year storm depth being different, and typically deeper, from the current NOAA data is that the climate has already been changing over time. If this is the case, and the extreme event depths have been increasing over time, then there should be a correlation between the GCM 2015 to NOAA difference and the projected change in 100 year storm depth as plotted in Figure 2-12. Figure 2-13 shows a contour plot of the GCM 2015 to NOAA difference for the entire state. Visual comparison between Figure 2-12 and Figure 2-13 indicates that the regions of higher storm depth growth (darker regions of Figure 2-12) correspond to regions of greater initial difference in depth (darker regions of Figure 2-13). Further evidence of this relationship is shown in Figure 2-14 which shows scatter plots of the initial difference versus projected change for each of the return periods considered. Again, a clear correlation is observed between the offset and the projected rate of increase in storm depth.

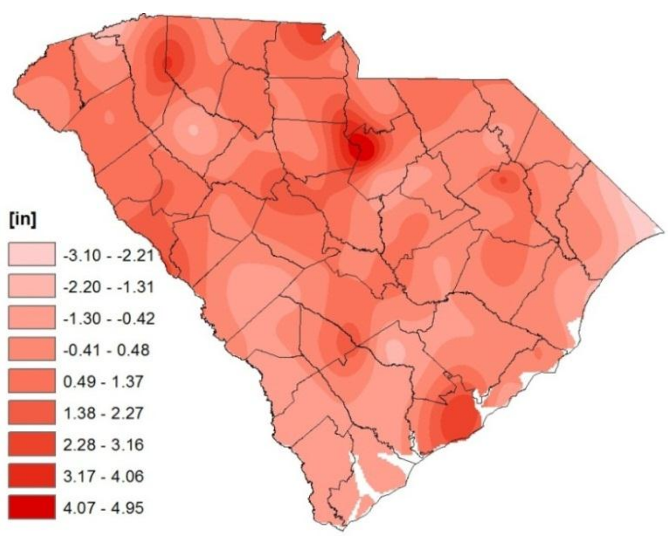

Figure 2-13 Contour plot of the offset between the 2015 GCM 100 year storm and the current NOAA data.

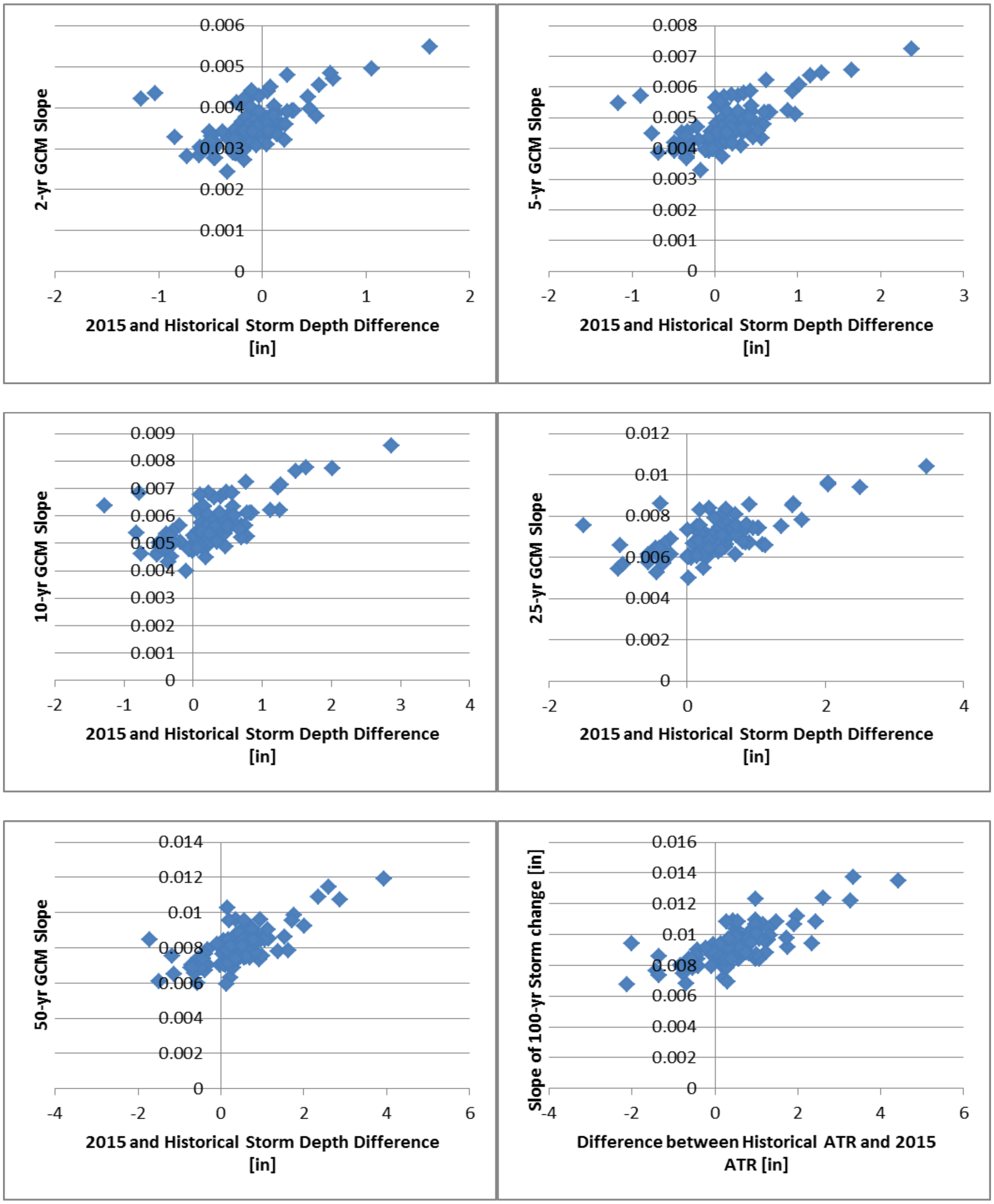

Figure 2-14 Scatter plot of the offset between the 2015 GCM 24 hour storm depth and the current NOAA data versus the projected growth in storm depth over the next 84 years. Reading from top and left to right, 2, 5, 10, 25, 50, and 100 year return period storms.

2.5 Discussion

A detailed analysis of the projected change in rainfall patterns in South Carolina has been conducted using BCCA downscaled GCM data from CMIP5. The GCM data show that average total annual rainfall will increase across the state over the remainder of the century. However, the increase is not uniform across the state with coastal regions predicted to have greater increases than most of the state. The Savannah River Basin is predicted to have below average growth in average annual total rainfall compared to the rest of the state. While the trend toward increasing ATR is clear in the data, the increase is quite small compared to typical year to year variability (see Figure 2-5).

The analysis also shows that the 2, 5, 10, 25, 50, and 100 year 24 hour design storm depths will all increase across the state over the remainder of the century. For example, the 100-year design storm depth is projected to increase between 0.5 and 1.2 inches across the state by 2099. In fact the GCM projections for 100 year return period 24 hour storm depths for most of the state will exceed the current NOAA 90% confidence interval in the next few years. However, the 2-year 24 hour storm depth will not exceed the NOAA 90% confidence interval until well into the next century for most locations in the state.

For both the ATR and the 24 hour storm depths there was an offset between the projected 2015 values and the historical data. In almost all cases the 2015 GCM ATR was greater than the historical mean though well within historical levels of variability. The offset between the current NOAA 24 hour storm depth data and the projected 2015 GCM values were quite varied. A substantial number of the offsets were negative indicating that the GCM storm depths were below the historical calculated values. However, the increase in storm depth over time was clear for every return period throughout the state. Further, the offset between the GCM and historical data was shown to be correlated to the local rate of change in the projected storm depths (see Figure 2-14). In general, the longer the return period of the storm, the greater the rate of increase in storm depth and the sooner the storm depth is predicted to exceed the current NOAA 90% confidence interval upper value.

The projected increases in both average annual total rainfall and design storm depths have the potential to stress existing stormwater infrastructure. The increases may also require regulatory agencies to re-visit their published design storm depths. One possible approach to mitigating the impact of these changes is to require new developments, as well as re-developments and retro-fits, to more closely replicate the predevelopment site hydrology. This could be done through the use of low impact development (LID) best management practices (BMP) to encourage infiltration and on-site runoff management. Such an approach has the potential to make new development more resilient to the projected changes in rainfall patterns.

Chapter 3 LID Usage in Stormwater Systems

3.1 Introduction

Given the predicted increases in ATR and design storm depths, it is important to investigate methods for mitigating their impact on stormwater infrastructure. This chapter examines the potential of Low Impact Development (LID) Best Management Practices (BMPs) to mitigate the impacts of an increase in rainfall depths. Current stormwater regulations for SC focus on hydraulic detention in order to match peak runoff for a 2- and 10- year storm depth. However, if these 2- as 10- year storm depths increase the additional rainfall will generate more runoff, thereby overloading the current systems. While LIDs traditionally focus on water quality they also allow for retention, infiltration, and evapotranspiration that helps control water quantity. This study used LIDs that slow down and remove runoff from a site to decrease the peak discharge and runoff volume. The study did not consider the water quality benefits of these LIDs.

To compare the effectiveness of LID to a standard site design, three different scale developments were analyzed. Both LID and standard designs were produced for the three developments. To investigate how location can impact the results, three different geographical locations representative of South Carolina were chosen (see Figure 3-1) and the site-dependent factors from each location were used in the design.

3.2 Methods

Traditional and LID designs were completed for a small commercial, large commercial and a residential development. For each development type, designs were completed for a location in the upstate, midlands, and coastal regions of South Carolina. Each location differed in design storm depth and soil type (soil class and saturated infiltration capacity). Once completed, each of the 12 designs, along with their pre-development conditions, were modeled in HEC-HMS v4.0 to analyze their performance for storm depths beyond their current design storm depth. HEC-HMS was used as it has flexibility in modeling techniques and is freely available to the public. HEC-HMS does not have built-in models for many LID technologies. The methods used for modeling LID type designs are discussed below in the design section.

3.2.1 Locations

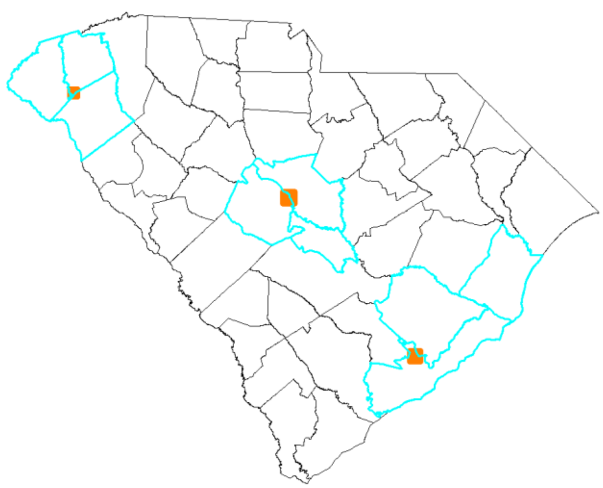

Figure 3-1 Theoretical site locations in orange with included county data highlighted in blue

Three theoretical sites were developed and run for three different locations throughout the state of SC. Stations located in the cities of Clemson, SC, Columbia, SC, and Charleston, SC will characterize the state's profile of piedmont, midland and coastal zones respectively (Figure 3-1). Changing locations will change the type of storm (II or

III), design storm depths, soil type, and soil infiltration capacity. The site topography was kept the same for each of the three locations.

Models were run using the NRCS (formerly SCS) 24 hour design storm hyetographs. The 24 hour rainfall distribution varies with geographic location throughout the United States. Coastal regions will have different distributions of rainfall patterns than non-coastal regions and the storm type accounts for that (Type III for coastal regions and Type II for non-coastal regions in South Carolina).

Soil infiltration capacity also varies across the state of South Carolina. The typical saturated infiltration capacity of the soil at each location was taken from the hydraulic conductivity rates reported in the USDA's online resource websoil survey (Soil Survey Staff, USDA). A three county average of the surface conductivity was taken for each location. The values obtained varied by location and will result in different LID performance for each geographic location (Table 3-1). Hydraulic conductivity is used in the Green-Ampt infiltration model (Green, Ampt 1911). For the Green-Ampt model, in the limit of a fully saturated soil the infiltration capacity of the soil approaches the hydraulic conductivity. Therefore, the hydraulic conductivity of the soil can be used as a conservative estimate of the infiltration capacity.

Table 3-1 Infiltration rates for locations with NOAA designated site names

	Site Name	Infiltration Rate [in/hr]
Upstate	CLEMSON COLLEGE	1.78
Midlands	COLUMBIA WSFO AP	6.70
Coastal	CHARLESTON WSO CITY	4.32

3.2.1.1 Development Scales

The three development type modeled were a small commercial development of five acres, a residential development of 70 acres and a large strip mall of 120 acres.

3.2.1.1.1 Small Commercial

The small commercial site was developed as a class project by different student groups, so four different variations are modeled to compare variability in design. The site was modeled after an empty lot located in Clemson, SC so predevelopment conditions were available for visual in-person inspection. Project design standards required a 15000 square feet building with 90 parking spaces (10 handicapped) and two entrances to the parking lot from adjacent streets. Beyond these guidelines, the designs were not restricted any further and the resulting designs from the class vary greatly. For example, the area of the porous pavement varied as some projects have just the area of the parking spaces as porous while others modeled the entire parking lot as porous pavement in order to meet project requirements of reducing runoff volume. A schematic diagram showing the site layout of one of the small commercial sites used is shown in Figure 3-2.

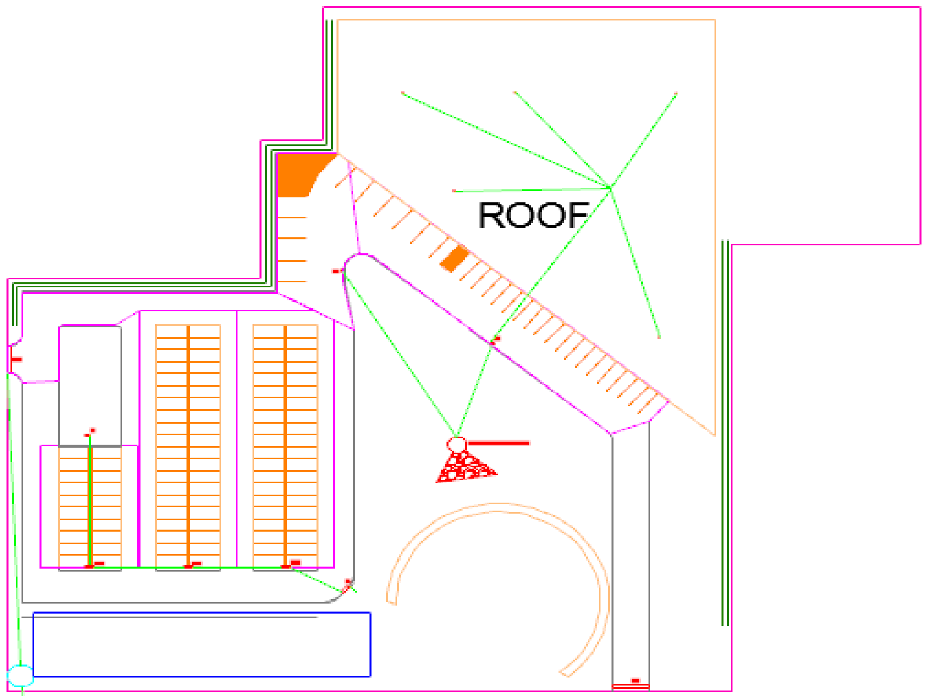

Figure 3-2 Schematic diagram of one of the four site layouts used for the small commercial site showing the trapezoidal building in upper site, porous pavement parking lots, and a semi-circle infiltration trench and detention pond in bottom site..

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3.2.1.1.2 Residential

The residential site design was modeled after a neighborhood in Cayce, SC that is 75.2 acres and has 178 lots, 1.7 miles of roadway, and two stormwater ponds. Two ponds are used based on the topography and original runoff directions of the site. Sidewalks are on both sides of every road and remain impervious but driveways were included as porous pavement for each site for the LID design. Lot geometry was based on homebuilder specifications and county parcel maps. Stormwater structure location and size was gathered from on-site measurements and confirmed using county map information. The site layout is shown in Figure 3-3.

Figure 3-3 Residential site layout with 178 lots and two detention ponds. Storm sewer piping is shown in blue and connects at each stormwater inlet. Due to topography of site, two ponds exist.

3.2.1.1.3 Large Commercial

The mall/big box store was modeled after the South Park Mall in Charlotte, NC. Mecklenburg County has a sophisticated GIS system called VirtualCharlotte. All inlet locations, pipe sizes, and pipe lengths were available online. A combination of VirtualCharlotte and Google Earth were used to construct a to-scale drawing of not only stormwater structures but also the building and parking lot locations. South Park Mall does have parking garages but they were removed from the design and were replaced with ground-level parking. Inflows from other sites are routed into the stormwater system in South Park but they were not considered and inlets begin with no upstream pipes since this analysis was focused on the performance of this individual site. The site layout is shown in Figure 3-4.

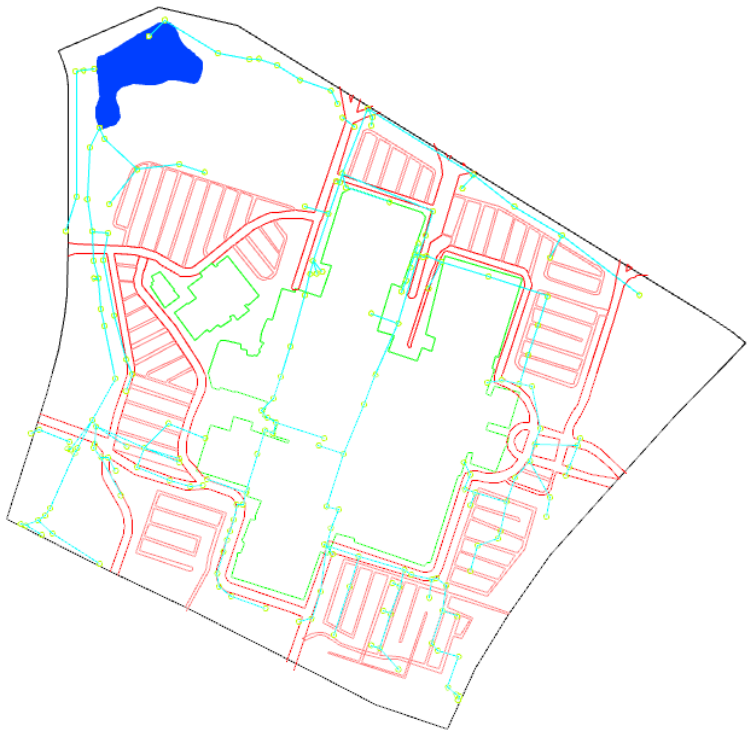

Figure 3-4 Large site development with sewer pipe outlines, structure outline in middle with parking lots surrounding the structure, and internal roadways connecting external access to parking. With a total lot area of 116 acres, a building size of 26.3 acres, and 2100 parking spots, runoff is routed through a 2.1 acre pond before flowing offsite.

3.3 Designs

All roadways were considered local streets having high average daily traffic (ADT) to conservatively set the design frequency standard. The design spread was set to half the width of a driving lane (Brown et al. 1996) which came out to 6 feet to fit all sites' roadway widths. Roadways are also all crowned at 2% that lead to a 6 inch gutter in a composite section (see Figure 3-5). Inlets were spaced based on spatial restraints (intersection, sag points, entrance and exits, etc.) or limitations on spread. The small site was fully governed based on spatial restraints while the other two sites did require inlets based on spread limitations. Inlets were all combination drain-curb inlets. Pipe sizing was restrained by SC Department of Transportation (SCDOT) minimum storm sewer sizes of 18 inches but pipe size was increased when necessary. Grass-lined channels existed on the small and large development sites. Pond sizing was found based on required storage from the site and used to develop a stage-storage relationship for the HEC model.

Each site was delineated based on the site's topography and each drain was assigned an area. This area is represented as a sub-basin in HEC-HMS. NRCS (SCS) curve numbers were used for the loss method and SCS unit hydrographs were used for the transform method. The pre-development runoff curve number (RCN) used was based on a curve number of fair soil conditions for a wooded-grass combination area. Drains were represented by junctions and connected to sub-basins by reaches. Reaches represent the connecting storm sewer pipes and flow routing was done using the Muskinghum-Cunge method (Fleming 2013). Circular pipes were used for all storm sewer pipes and grass lined channels were trapezoidal.

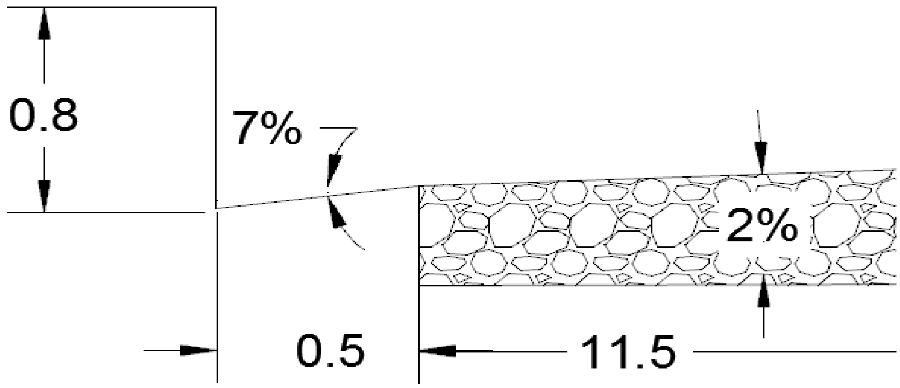

Figure 3-5 Typical cross section of roadway to the crown of the road with curb and gutter and dimensions in feet.

3.3.1 Pre-development Conditions

Outflow from an undeveloped site is often used as a target value while designing a site. Increasing the peak runoff or runoff volume can lead to downstream flooding, increased erosion, and other problems. However not allowing enough water to continue downstream can cause on-site flooding and reduced flow in downstream waterways.

The NRCS (SCS) time of concentration was calculated for the pre-development conditions based on land slope and the hydrologically longest flow path for each delineated basin of the site. HEC-HMS models with these larger natural sub-basins were then run to get the peak pre-development outflow and runoff volume.

The small commercial site lot was undeveloped, so pre-development design was simple. Due to the small size, small differences in curve number or time of concentration values between student models impacted pre-development runoff estimates greatly; therefore these values were standardized for all the models. The residential and large development sites are based on already developed sites and pre-development site characteristics were not available. Therefore, predevelopment times of concentrations for these sites were based on current site grading and a runoff curve number (RCN) for fair soil conditions and a wooded-grass combination.

3.3.2 Standard Design

Standard designs for the small commercial site consisted of calculations for inlet spacing, pipe sizing, culvert sizing, and pond geometry. Height and sizing of outlet structures for the stormwater pond were calculated based on matching the pond outflow with pre-development peak runoff for 2 and 10 year 24-hour storms. SC-DOT regulations have a minimum pipe size of 18 inches so most pipe diameters on such a small site were rounded up to this size.

For the residential site, on-site measurements of inlet location and size, pipe size, and outlet structures setup allowed for an accurate model of this existing developed residential site. For ease of modeling, each inlet had a sub-basin created that covered several lots and a length of pavement. A cumulative curve number was calculated based on the percent of area that was house, grass and pavement. Time of concentration for each sub-basin was based on the hydrologically longest length of flow within the sub-basin. Outflow from both ponds were routed to a theoretical outlet to obtain a cumulative outflow from the site.

For the large commercial site, inlet locations, pipe sizes and lengths, and placement were based on the VirtualCharlotte database. As for the residential site, a sub-basin was created in HEC-HMS for each inlet with its drainage area and time of concentration calculated as before. The site had a very large pond (larger than 2 acres) and no information on the outlet structures in the pond was available online or through Mecklenburg County engineering offices. Therefore, the outlet sizes were calculated based on historical satellite images and matching of the 2 and 10 year storm peak discharges with the calculated pre-development values.

3.3.3 LID Design

The standard design for each site became the starting template for the LID design. The LID technologies used were porous pavement, infiltration trenches, and green roofs. Implementation of each technology varied from each site and is discussed below. Each sub-basin for the HEC-HMS model was divided into areas of pavement, roof and remaining area that would not be used as a LID. Site layouts differed slightly for the small commercial development between standard and LID design due to the nature of class project guidelines, but the areas of the building, pavement, etc. remained the same between the two designs. For the residential site and large commercial development, the layout remained the same and the only changes were the addition of the LID technologies. Sizing of the porous pavement was based on the percentage of area within each previously delineated section. In HEC-HMS, the porous pavement sub-basin, along with green roof outflows and lawn sub-basins, were routed to the porous pavement pond. Residential site design relied on percentage of areas for each component due to each lot having lawn space, green roof space, and porous pavement. The large development design had drains dedicated to pavement or roof space so area percentage was not used for the model construction.

3.3.3.1 Porous Pavement

Porous pavement and infiltration trenches were modeled as infiltration ponds in HEC-HMS to represent their hydrologic behavior. The surface of the porous pavement was modeled as a sub-basin with an area equal to that of the porous pavement surface, a curve number of 98 to account for wetting losses (Schwartz 2010), and 1 min time of concentration. The sub-basin runoff was then routed into the infiltration pond. Another sub-basin was connected to the pond for any adjacent surfaces that sent runoff into the pavement but were not part of the porous pavement. A schematic of this model is shown in Figure 3-6.

Infiltration in the base of the pond was represented in HEC-HMS as a pump. The pump had a constant discharge with elevation and an intake at the base of the pond. The pump discharge (Q_p) was given by

$$Q_p = \varphi_s f_s A_p \tag{1}$$

where A_p is the pond (pavement) area, f_s is the saturated soil infiltration capacity, and φ_s is the porosity of the pavement sub-base stone and accounts for the masking effect of the stone on the soil (Schwartz 2010). The pump outflow is routed as an auxiliary flow to an infiltration outlet so that it was not counted toward the main site runoff.

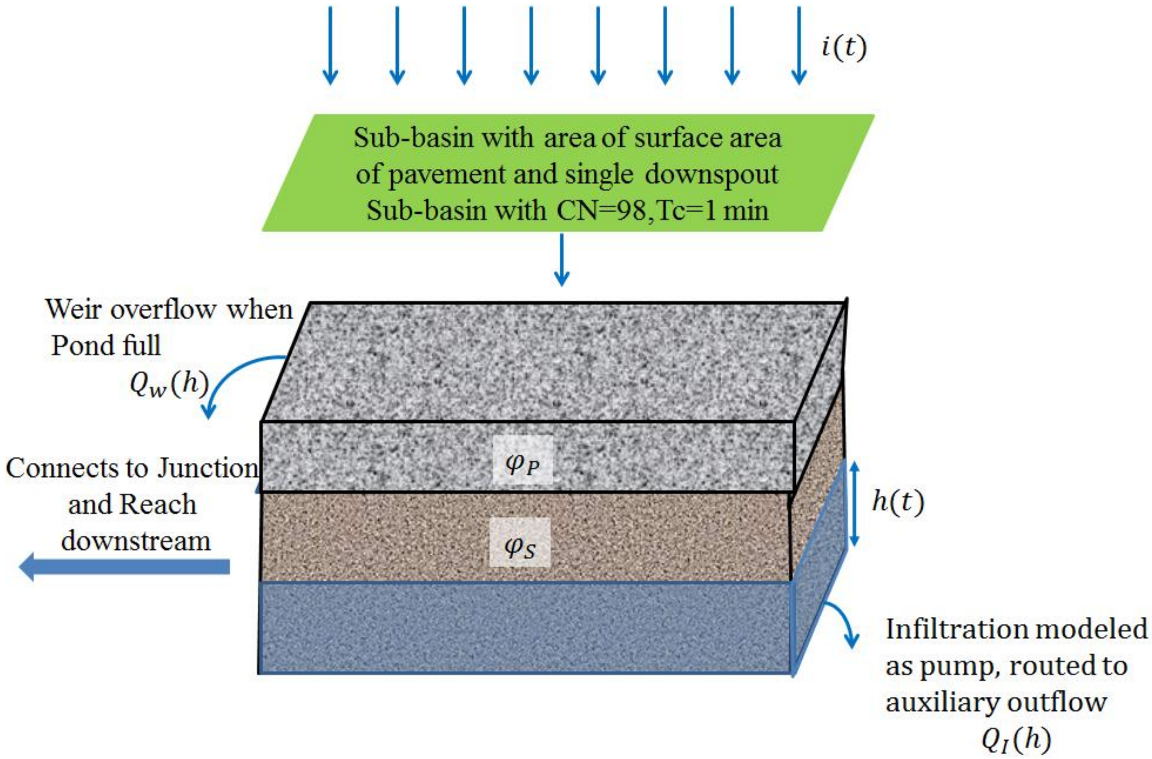

Figure 3-6 Flow path for porous pavement system modeled in HEC-HMS.

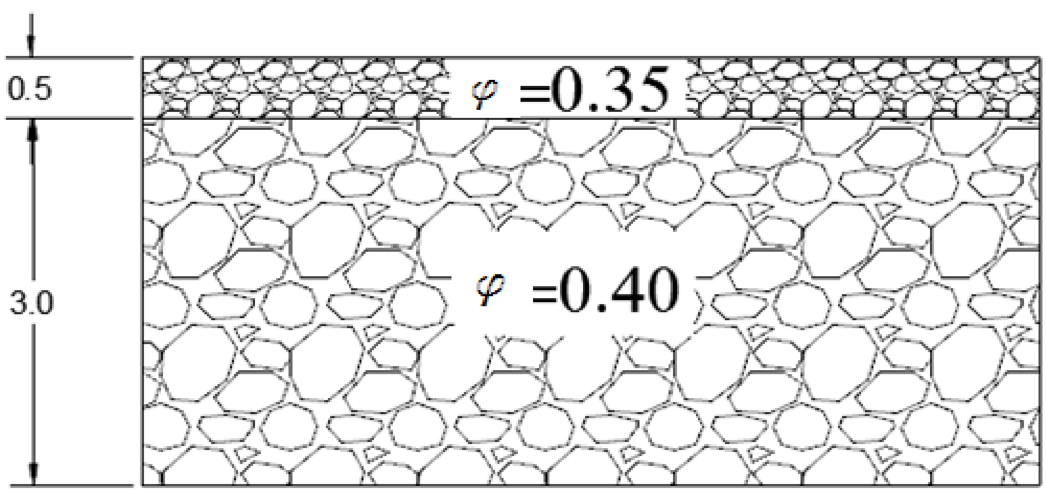

Figure 3-7 Typical porous pavement cross-section with dimensions in feet and the porosity of each section, φ.

Both the porous pavement and infiltration models also have a dam top with level overflow at elevations corresponding to the top of the pavement or trench. The depth of these infiltration ponds is based on the infiltration rate and the 72-hour drawdown limitation. The lowest soil infiltration capacity across the locations considered was used to size all pavement and trench depths regardless of location to keep the designs consistent. A schematic of a typical porous pavement section is shown in Figure 3-7.

Table 3-2 Layer depth and porosities for all LID technologies used

LID Type	Layer	Depth from surface [ft]	Porosity
Porous Pavement	Pavement	0-0.5	0.35
	Subbase	0.5-3.5	0.4
Infiltration Trench	N/A	0-3	0.4
Green Roof	N/A	0-0.33	0.5

An elevation-discharge table was created with the assumption of constant infiltration throughout the entire time window provided there was a finite depth of water in the pond. The storage height function for the pond is based on the area and the porosity of the sub-base or pavement.

3.3.3.2 Green Roof

Green roofs were extensive (Razzaghmanesh and Beecham 2014) for all sites and had a height of 4 inches. Green roof modeling is similar to modeling a porous pavement with the entire green roof area being modeled as a pond. The surface of the green roof is a sub-basin with a curve number of 86 (Carter and Rasmussen, 2006) and a 1 min time of concentration. The runoff from this sub-basin runs into the pond. The outflows of the green roof pond are discharged through an orifice at the bottom of the green roof (at elevation 0.01 feet) and a dam top at the height of the green roof. The orifice was sized based on the required drawdown time of a full (pore space filled) green roof. As each green roof area differed, the paired

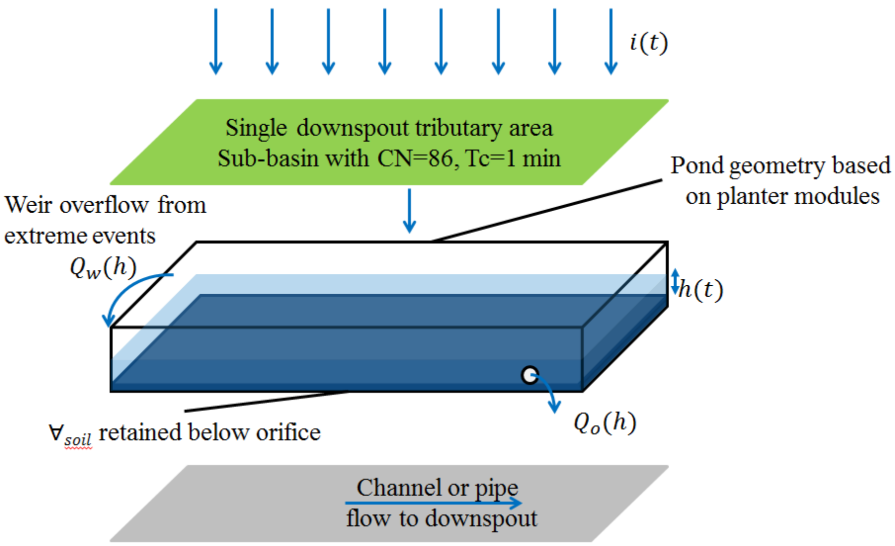

Figure 3-8 Schematic diagram of the flow routing model for an extensive green roof

orifice size differed accordingly. The orifice size was based on a 30 minute drawdown time for a porosity of 50%. The green roof's area was equal to the area of the buildings and flow was routed to roof drains and downspouts which were routed to a drain at the ground level.

For each residential development sub-basin the number of lots determined the green roof area. To incorporate green roof modules, residential houses were assumed to have flat roofs. The average footprint of the houses in the entire development was 2800 square feet so the number of lots times this area determined green roof size. The collective green roof sub-basin in HEC-HMS ran onto a collective green roof pond and then onto the porous pavement pond via a reach. The reach from the roof to the porous pavement pond was calculated after first running the entire site for a 10 year storm depth and then designing each downspout based on the peak outflow from the roof.

3.4 Results

Each development scale at each location was run in HEC-HMS for precipitation depths ranging from 0.5 to 14 inches in increments of 0.5 inches. The highest rainfall storm depth was for the 100 year return period in the state was 10.3 inches (for Charleston, SC) and modeling the developments for a greater than 100 year storm event was not necessary. However, based on the GCM forecasts in Chapter 2, Charleston's 100 year 24 hour storm depth is predicted to increase to 13.94 inches by the end of the century so modeling was performed from 0.5 inches to 14 inches. For pre-development and standard designs, peak runoff and runoff volume only varied by the storm type (i.e. II or III) however LID designs considered location dependent infiltration capacity so each location varied in peak runoff and total runoff. Pond inflow and outflow were recorded for standard designs to capture the impact of a detention pond. LID designs were run for the same pond dimensions as the standard design (to examine the performance of a site retro-fit) and a smaller pond (LID pond) that was designed to match the LID 2 and 10 year peak outflow with the pre-development values. The outflow structures were held constant from the standard design and only the pond dimensions were changed for the

LID pond. Since all of the ponds were detention ponds with no infiltration or extended storage (beyond 72 hours after storm), runoff volume did not vary between inflow and outflow of the pond.

3.4.1 Peak Runoff

Peak runoff for all development scales and locations are shown in Figure 3-9, Figure 3-10, and Figure 3-11. Based on stormwater regulations, the 2 and 10 year storm depth pre-development peak runoffs were matched for each location and beyond that point the peak runoff varies greatly. Inflow to the pond for the standard design was the highest peak runoff for all cases as no volume reduction or outflow control is used. Including the pond for the standard designs brings down the peak runoff curve to match at the 10 year storm depth but then follows the same slope as the standard design without a pond. The same relationship happens for the LID design between the inflow to the pond and outflow to the pond. The LID design does have two options for the pond, the same pond size as the standard design and a resized LID pond, in which the LID pond has higher peak discharge values due to less storage availability. In comparison to the pre-development case, outflow from both LID designs are less for the entire range of precipitation rates.

As expected with a smaller amount of storage, the LID ponds have higher peak discharge values than LID with the standard ponds. Clemson, with the lowest infiltration rate in this study and therefore the largest inflow to the pond, had a 73% reduction in the required pond size. Changes in the outlet structures would have affected this reduction but isolating the change to just the pond size was done for consistency of comparing outflows from site designs.

With the highest outflow regulation at the 10 year storm depth, the behavior after this design point shows how a site reacts in larger storm events and gives insight as to how a site may react after consecutive days of rainfall. With design life often over 50 years, increasing storm depths are likely to occur during the life of a storm sewer structure. To quantify this, a three point forward difference method was used to measure the slope of the peak runoff line to characterize the pond outflow after the 10 year storm depth requirement (Figure 3-3-12, Figure 3-3-13, and Figure 3-14). All locations had return storm depths between the rainfall steps (i.e. 10 year storm for Columbia is 5.28 inches and model was run for 5.0 and 5.5 inches) so the three point forward difference starts at the preceding rainfall step to the return period storm depth.

The results of this analysis are inconsistent. For example, for the small commercial development the pre-development conditions resulted in the largest jump in peak outflow with the standard development being the most resilient to changes in storm depth. In all cases the standard design located in Columbia was the poorest performer with the largest increase in peak flow per additional inch of rainfall for the locations considered. This is likely due to the current design storm depth in Columbia being the smallest of the three such that an additional inch of runoff represents the largest percentage increase of the three locations. In general the larger development LID designs performed better than the standard designs.

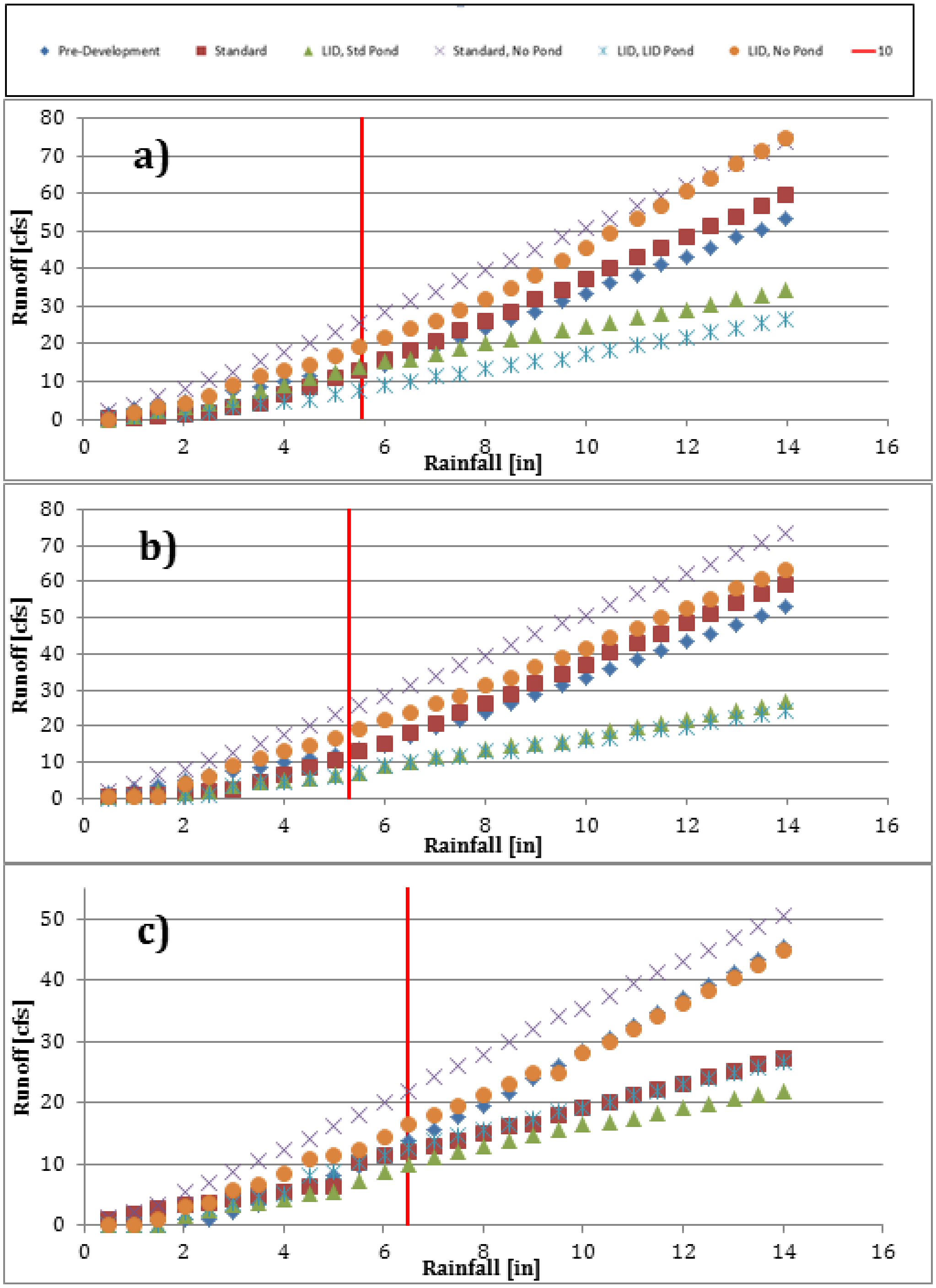

Figure 3-9 Peak runoff curves for small development for Clemson (a), Columbia (b), and Charleston (c). A master legend is located above the graphs.

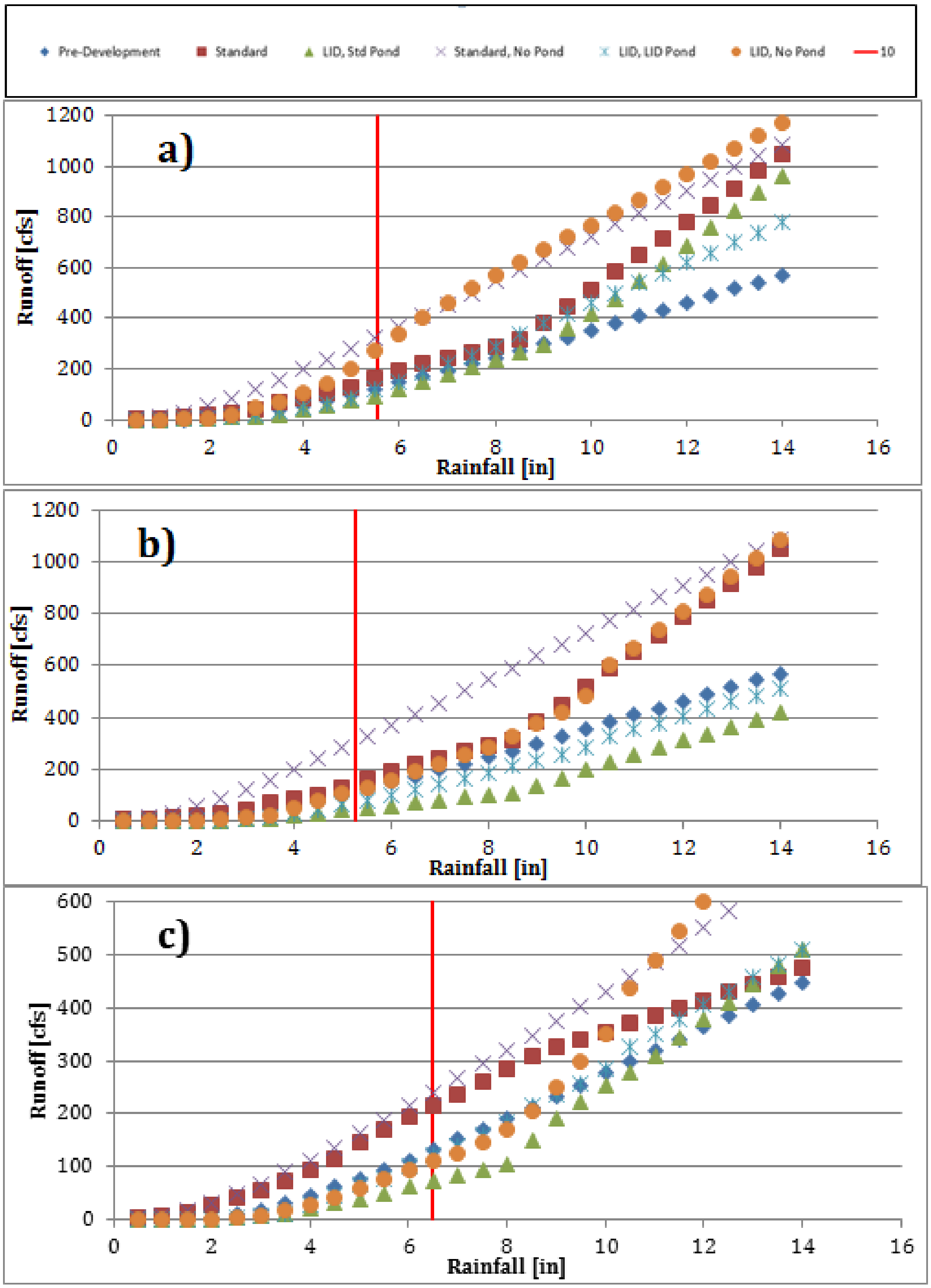

Figure 3-10 Peak runoff curves for residential development for Clemson (a), Columbia (b), and Charleston (c). A master legend is located above the graphs.

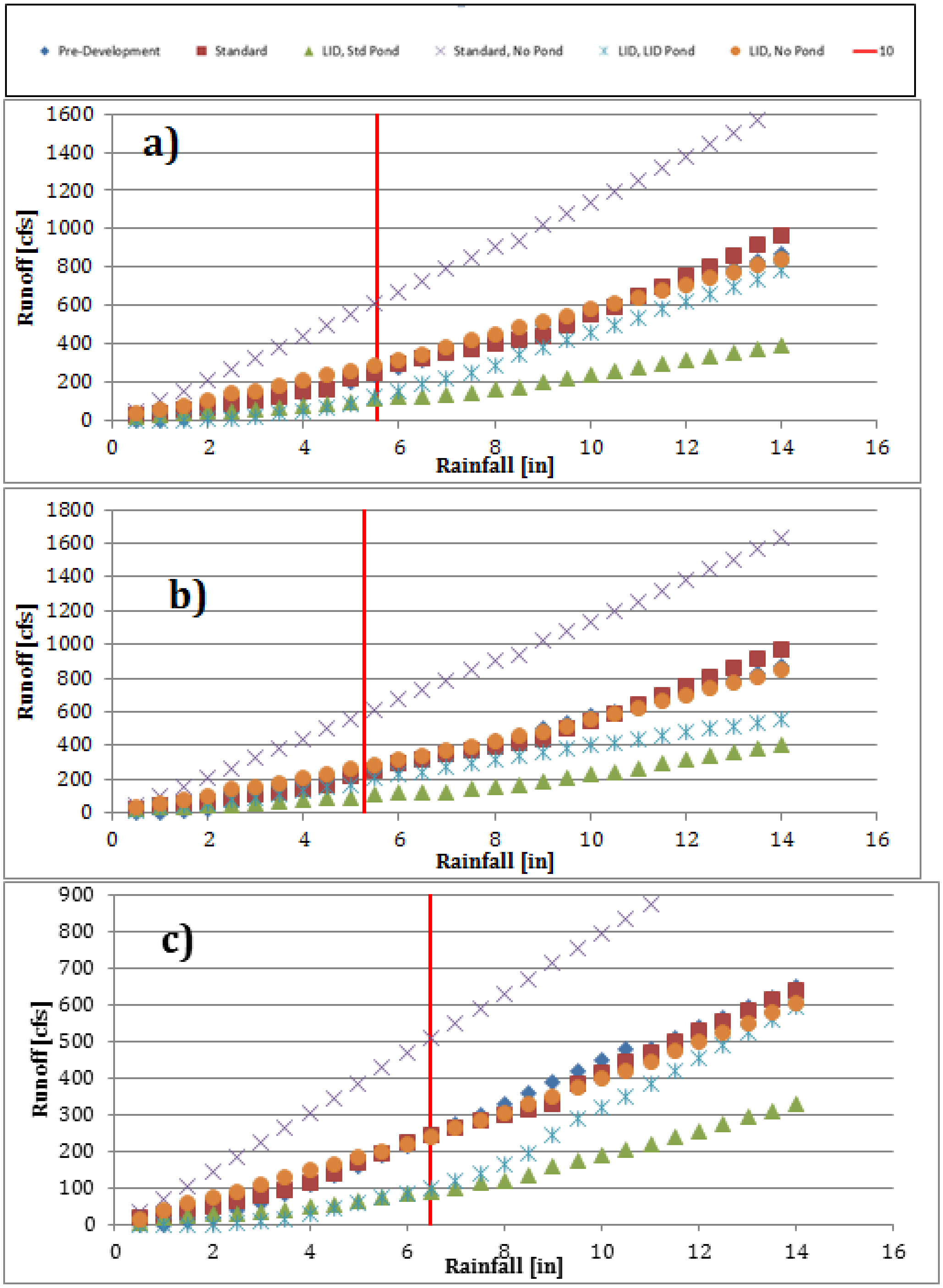

Figure 3-11 Peak runoff curves for large development for Clemson (a), Columbia (b), and Charleston (c). A master legend is located above the graphs.

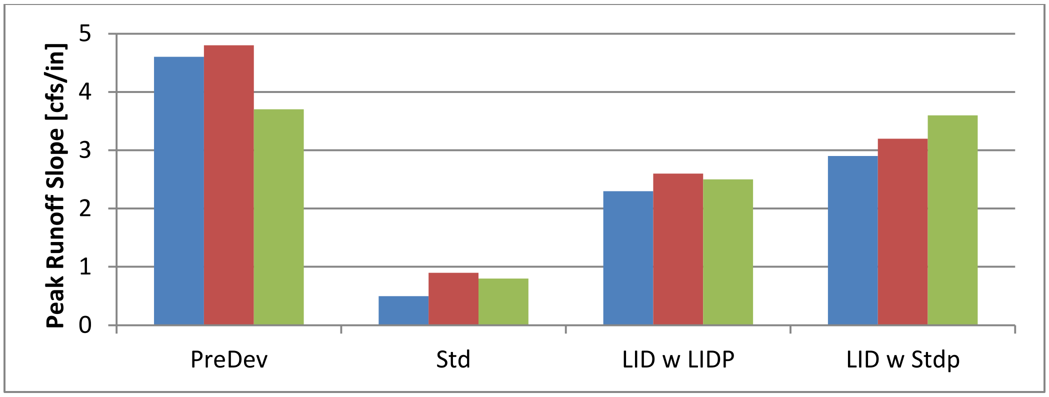

Figure 3-3-12 Comparing three-point forward difference for small commercial site.

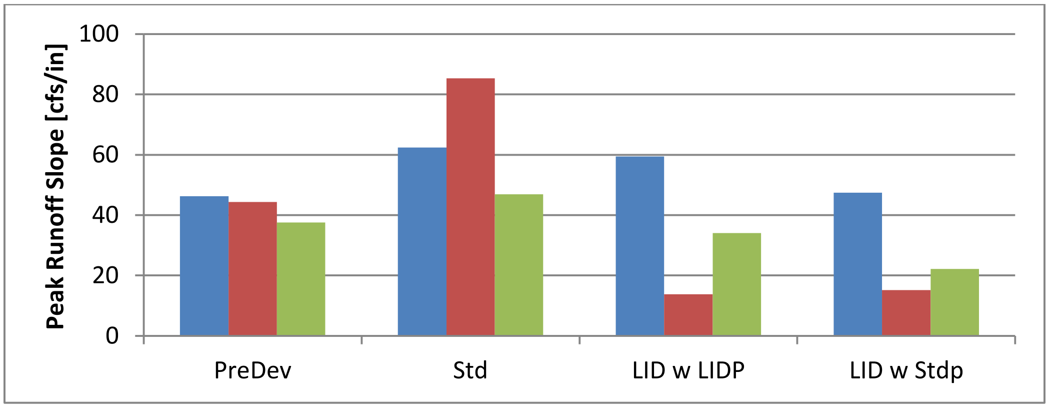

Figure 3-3-13 Comparing three point forward difference for residential site.

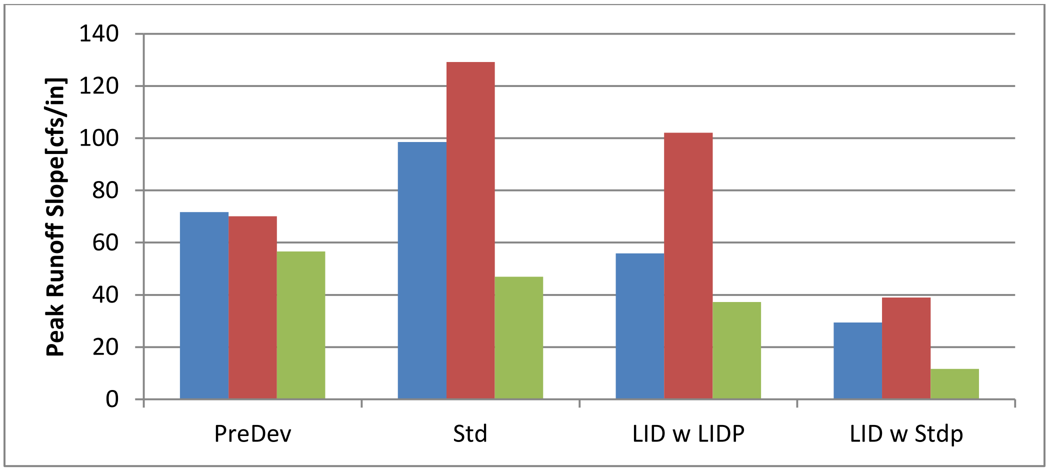

Figure 3-14 Comparing three point forward difference for large development site.

3.4.2 Runoff Volume

As discussed previously, besides small variations due to storm type, the peak runoff did not vary with location for pre-development and standard design conditions. Infiltration was the only variable to have an impact between the locations and only altered the performance of the LID designs. The use of LID technologies led to a significant reduction in the runoff volume, especially in the Columbia location, which had the highest soil infiltration capacity. This volume reduction for these cases only indirectly influences the design as it allows for a smaller pond capacity while still achieving the desired peak load attenuation. However, other states do require volume reduction and all sites would benefit from a reduction of runoff volume. Table 3-3 shows the percent reduction of required pond size for LID designs. Onsite infiltration will improve water quality, replenish groundwater storage, and reduced downstream erosion of channels. Total runoff-precipitation curves are shown in Figure 3-15, Figure 3-16, and Figure 3-17. The volume of runoff for the standard designs were highest for every case and while the LID designs are matching or less than the pre-development runoff. Inflow and outflow from the pond are not compared as the pond does not have any volume reducing factors and therefore the inflow volume matches the outflow volume.

Table 3-3 Percent reduction in required detention pond volume for LID design for 10 year outflow requirement compared to standard pond.

	Clemson	Columbia	Charleston
Small	76%	97%	89%
Residential	40%	99%	86%
Large	72%	89%	81%

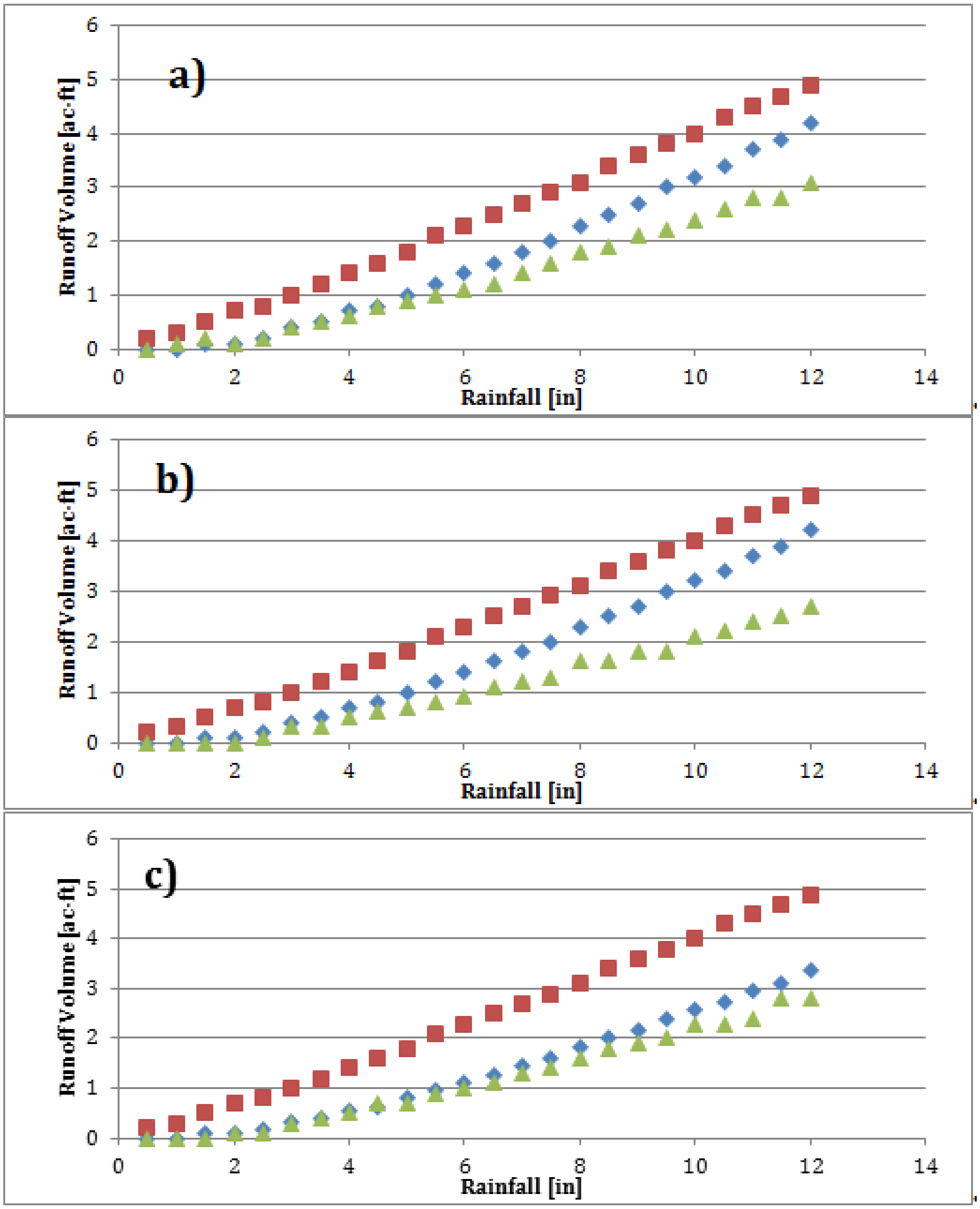

Figure 3-15 Runoff Volume for small commercial sites for Clemson (a), Columbia (b), and Charleston (c). The red squares represent post development, the blue diamonds are pre development, and the green triangles are the LID case.

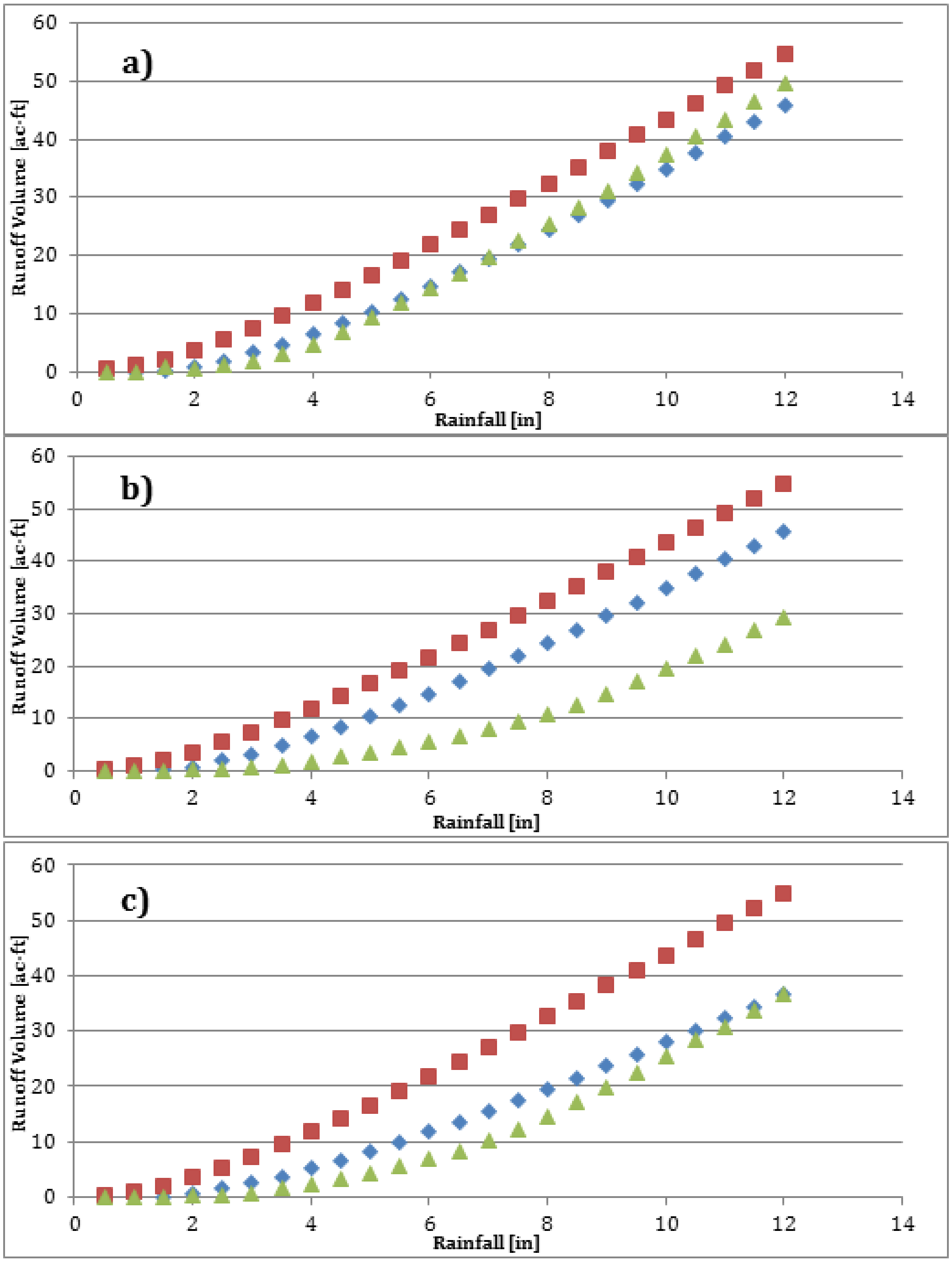

Figure 3-16 Runoff Volume for residential commercial sites for Clemson (a), Columbia (b), and Charleston (c). The red squares represent post development, the blue diamonds are pre development, and the green triangles are the LID case.

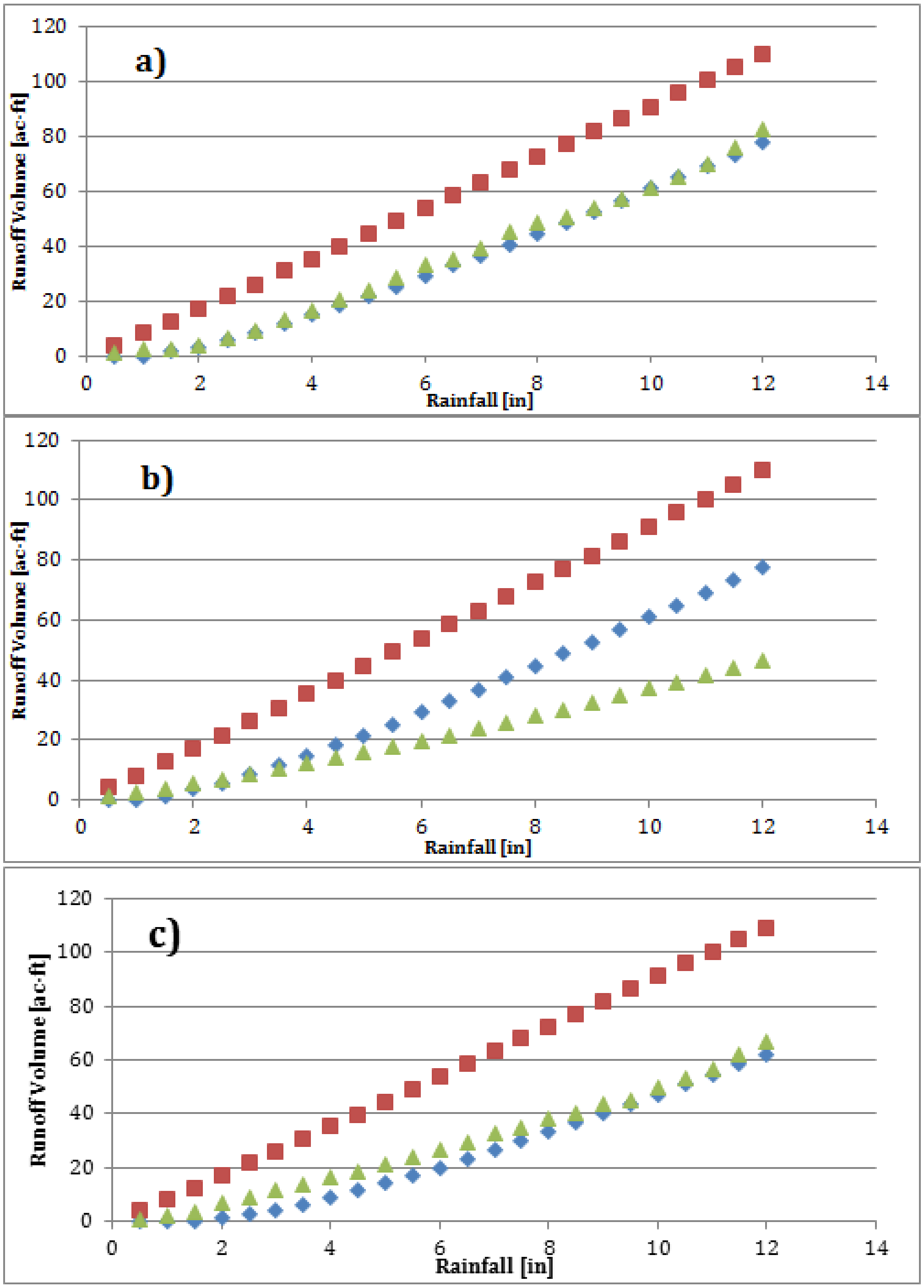

Figure 3-17 Runoff Volume for large commercial sites for Clemson (a), Columbia (b), and Charleston (c). The red squares represent post development, the blue diamonds are pre development, and the green triangles are the LID case.

3.4.3 Comparison of performance of multiple small commercial models

Site design can vary greatly based on the required specifications and site characteristics. As mentioned before, the small development site was developed as part of an undergraduate class assignment so groups of engineering students each created a different design with the same guidelines. This allows for a comparison of the variability in designer approach to meeting the project requirements.

Looking at the runoff volume reduction (Figure 3-18b), the benefits of LID implementation are easily seen as all four LID designs show a reduction in runoff volume compared to the standard design. This means that the use of LID technologies on a site will easily reduce the water volume from a site, but that does not guarantee that the peak runoff rate will be decreased.

Figure 3-18a shows the variability in peak runoff from different designs using LID designs in comparison to the standard and predevelopment cases. All sites (four LID cases and one standard design case) match the predevelopment curve as the 10 year storm depth but beyond that their performance varies widely. Two of the four LID designs have a peak runoff rate that is less than or matches the pre-development rate. A third falls between the pre-development and standard development performance, and the fourth has a peak runoff rate that exceeds the standard development peak rate at larger rainfall depths. This difference in performance is likely due to the different designs of the pond and outflow structure by the four teams. Even though the teams had the same objectives, the size and type of their pond ended up having the largest control on how the runoff rate changes with increased rainfalls.

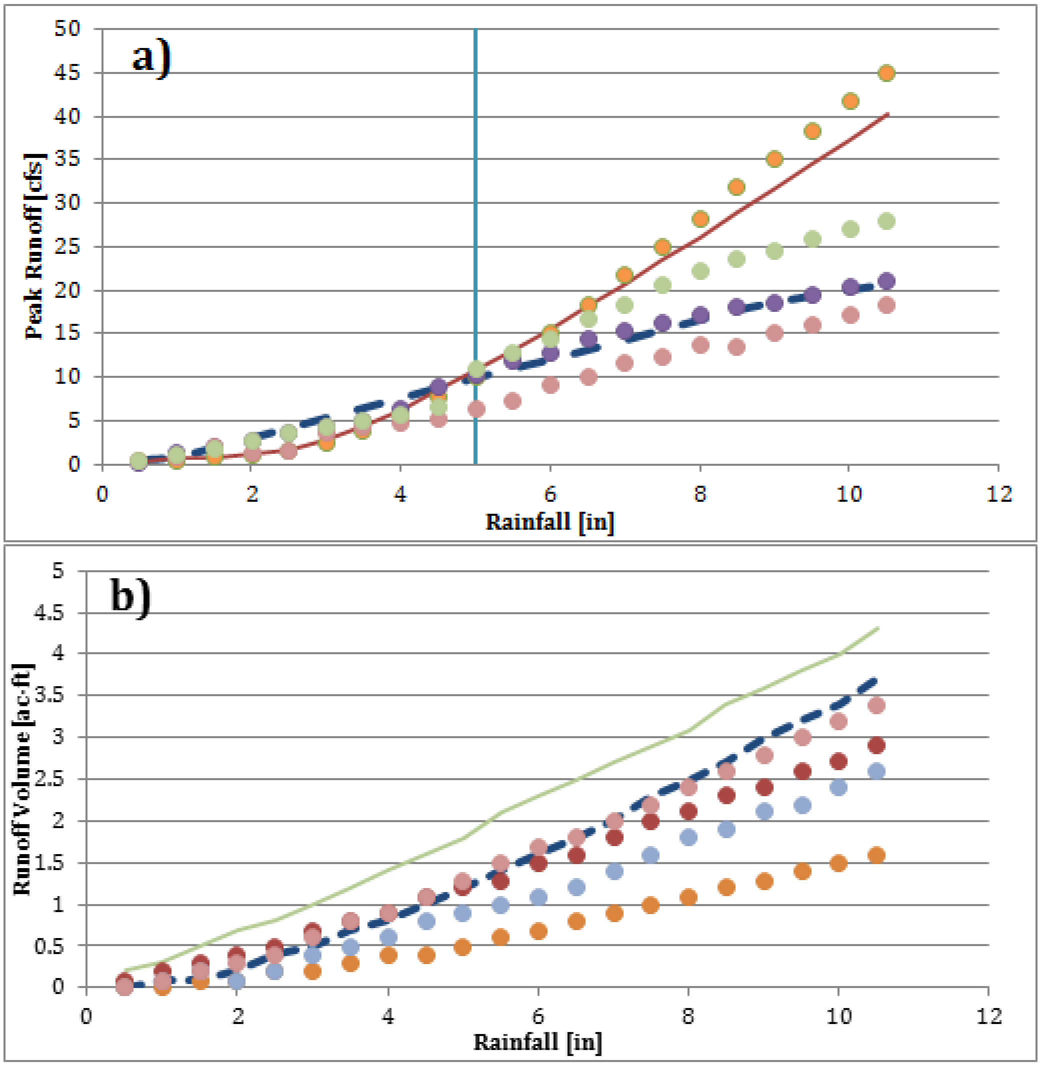

Figure 3-18 a) Comparing variability in design for small commercial site for peak runoff and b) comparing variability in design for small commercial site for total runoff volume. For both sites, upper solid line is standard design, lower dashed line is predevelopment design, and other plots are four separate designs, while the red vertical line is the 10 year design storm value in the a) graph.

3.5 Conclusion

To compare the effectiveness of LID technologies to a standard site design, three different development types, varying from 4 to 120 acres, were constructed. These three development types were modeled in HEC-HMS for three locations in South Carolina. Stormwater regulations in SC only regulate peak runoff from a site for the 2 and 10 year 24-hour storm depth so runoff volume from a site has little to zero consideration in the design.

Volume reduction through the LID design reduced total runoff and peak runoff from the site. Infiltration rates varied based on location soils and the resulting effectiveness of LID varied based on this. At the location with the lowest infiltration rate, implementation of LID methods reduced the pond size by 70%.

Chapter 4 Summary, Conclusions, and Recommendations

Forecasts of future precipitation at 101 locations throughout the state of South Carolina were analyzed using 134 BCCA downscaled, CMIP5 GCMs. Locations were based on existing NOAA precipitation measuring stations so direct geographical correlations could be compared. The GCMs used went from January 1, 2015-December 31, 2099, so 85 years of future precipitation trends were examined. To calculate the standard return storm depths (2, 5, 10, 25, 50, and 100), adjustments and regional factors from NOAA were used. Storm depths for year 2015 for all return periods were offset from 2014 storm depth and these differences were compared. All storm depths increased over the GCM period of record with larger increases predicted for longer return period storms. ATR trends were found through annual averages of the 134 realizations of 21 GCMs for each year. The Savannah River basin, a stressed water region in SC, has the lowest increases in return period storm depths and also is subject to the lowest ATR increases in the state. The coastal regions of SC have the highest increases in intense storm depths. These regions are already flood prone and climate change will only increase this problem. For example, Clemson's (station ID 38-1770) 100 year storm depth is predicted to increase to 10.89 inches in 2099 from its current value of 9.24 inches (NOAA).

With future precipitation increasing, stormwater systems will be more prone to failure. A series of case studies were developed to examine the possibility of using LID technologies to make stormwater infrastructure more resilient to changing rainfall patterns. Three development sizes varying from 5 to 120 acres were analyzed at three locations in South Carolina. Location dependent factors included the soil infiltration capacity, storm type, and design storm depth. Based on state stormwater regulations, stormwater detention ponds were sized based on the 2 and 10 year storm depth peak runoff from the predevelopment case. Sites were modeled using HEC-HMS and LID technologies were represented in HEC-HMS by shallow ponds or infiltration ponds. Each site was run in HEC-HMS for every location and design type from 0.5 inches to 14 inches in 0.5 increments. As expected at 14 inches of precipitation, there is a significant separation between the peak runoffs from different design types. Overall, use of LID technologies greatly decreased the total runoff from a site and had a lesser, but still clear, impact on the peak runoff from a site. This approach of using LID for water quantity management can be applied in a retrofit of an existing site which could lead to the pond size being significantly reduced giving the owner more useable land to develop. If precipitation does increase over the life of the site, a LID site would show greater resiliency and be less impacted than a standard design site. Although the results from the residential and large commercial were each from one design, the effectiveness of LID implementation can be shown by looking at the variability in the small commercial designs, which had an average of 11.6 cfs (28% of outflow) decrease in runoff.

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