AECOM

PRELIMINARY

Mecklenburg County Floodplain Mapping 2008

Catawba Sub-Basin Hydrology Report

4/22/2010

Updated 7/14/2010
Updated 9/25/2010
Updated 1/28/2011
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Updated 9/30/2011

Section 1 Watershed Description

1.1 Watershed Location

The Catawba River Basin is the eighth largest river basin in North Carolina, covering approximately 3,285 square miles. The Catawba River Basin in Mecklenburg County is located in the central area of the Blue Ridge/Piedmont hydrologic region of North Carolina. The sub-basin terrain is characterized by rolling hills with moderate relief and narrow, steep stream valleys in the northern portion and more level terrain in the south. Our study area drains mostly urban areas in the southern part of Mecklenburg County and contains the McMullen, McAlpine, Four Mile, and Six Mile Creek sub-basins.

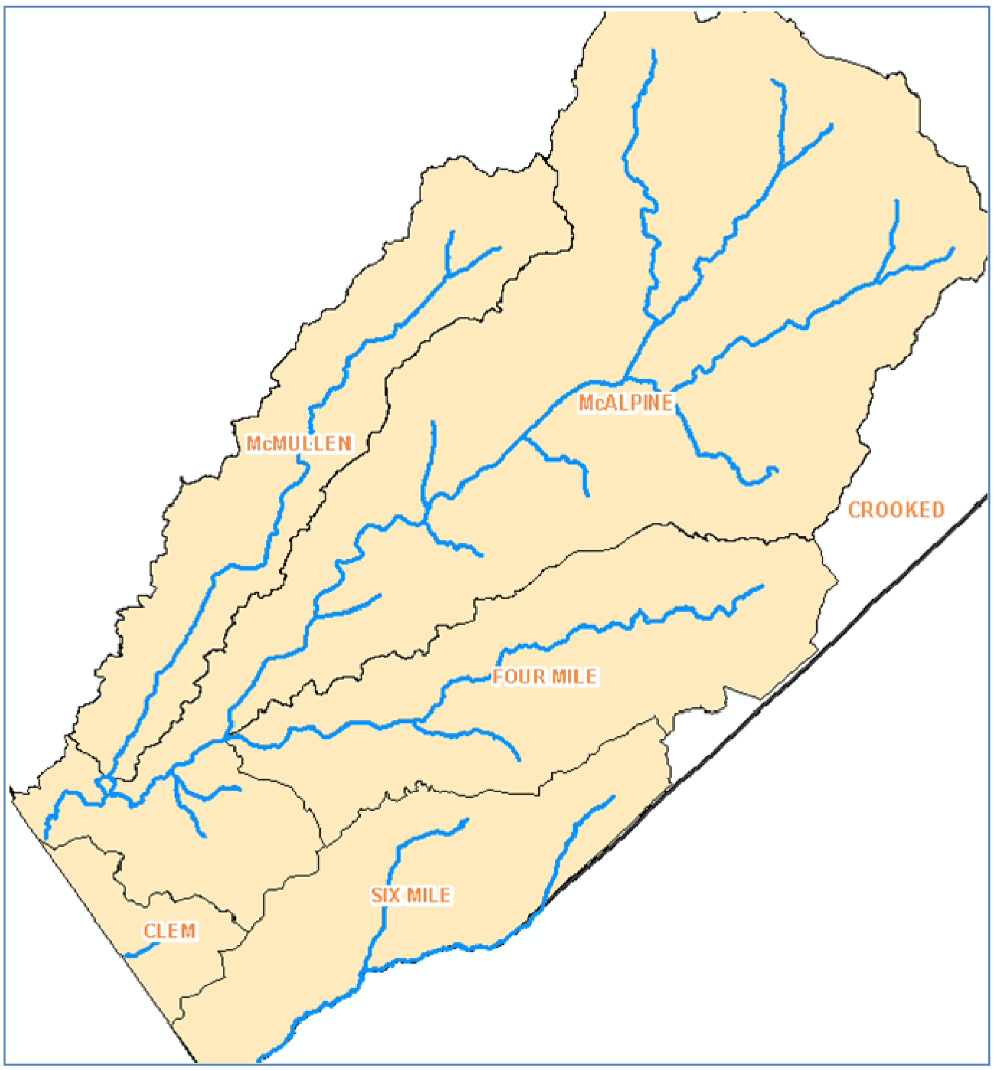


Figure 1. Catawba River Watershed in Mecklenburg County

The study area of the Catawba River Watershed contains 80 miles of detailed study FEMA streams with 137 hydraulic structures. The study limits are summarized below in Table 1.

Table 1: Detailed Study Scope for Catawba River Sub-basin			
Stream Name	**Downstream Limit**	**Upstream Limit**	**Length (mi.)**
Campbell Creek	Confluence with McAlpine Creek	Approx. 750 feet upstream of Barcliff Drive	5.3
Clems Branch	County Line	Approx. 190 feet upstream of Lancaster Highway	0.7
Flat Branch	Confluence with Six Mile Creek	Approx. 0.9 miles upstream of Tom Short Rd	3.1
Four Mile Creek	Confluence with McAlpine Creek	Approx. 1,590 feet upstream of E. John St.	9.9
Irvins Creek	Confluence with McAlpine Creek	Approx. 375 feet upstream of Lawyers Road	6.2
Irvins Creek Trib 1	Confluence with Irvins Creek	Approx. 2400 feet upstream of Independence Blvd.	2.7
Irvins Creek Trib 2	Confluence with Irvins Creek	Approx. 0.6 miles upstream of Lawyers Road	1.3
McAlpine Creek	County Line	Approx. 500 feet upstream of Albemarle Rd.	19.3
McAlpine Cr Trib 1	Confluence with McAlpine Creek	Approx. 0.7 miles upstream of Hwy 521	1.3
McAlpine Cr Trib 1A	Confluence with McAlpine Creek Trib 1	Approx. 485 feet upstream of Ballantyne Commons Pky	1.1
McAlpine Cr Trib 3	Confluence with McAlpine Creek	Approx. 600 feet upstream of Providence Rd.	1.2
McAlpine Cr Trib 6	Confluence with McAlpine Creek	Approx. 1.1 miles upstream of confluence	1.1
McMullen Creek	Confluence with McAlpine Creek	Approx. 0.8 miles upstream of Addison Road	10.9
McMullen Creek Trib	Confluence with McMullen Creek	Approx. 300 feet upstream of Sharon Amity Rd.	0.7
Rea Branch	Confluence with McAlpine Creek	Approx. 210 feet upstream of Sequoia Red Ln.	1.0
Rocky Branch	Confluence with Four Mile Creek	Approx. 0.5 mile upstream of Providence Road	2.1
Sardis Branch	Confluence with McAlpine Creek	Approx. 800 feet upstream of Sardis Road	1.6
Six Mile Creek	County Line	Approx. 0.6 miles upstream of Tilley Morris Road	9.0
Swan Run Branch	Confluence with McAlpine Creek	Approx. 1 mile upstream of Sharon View Road	1.4

1.2 Hydrologic Subdivision of Watershed

The target sub-basin size for this study was determined by the county to be 60 acres. The intent was to reflect more localized hydrologic patterns in the headwaters of the streams to be studied. The overall average size of a sub-basin is 74 acres. This includes the larger main reach basins and basins located outside of the county that drain into the county. We feel that the headwaters are well represented with the smaller basin size. Figure 2 shows the sub-basins as delineated and approved by the county.

Basin delineations and drainage areas were determined using a 10' x 10' grid size digital elevation model (DEM) generated from the Light Detection and Ranging (LIDAR) data collected by the county. Drainage areas from the current effective study were determined using a 50' x 50' grid cell so there may be some differences when compared directly. The effective study was also based on larger scale sub-basins with a typical size between 150 – 200 acres.

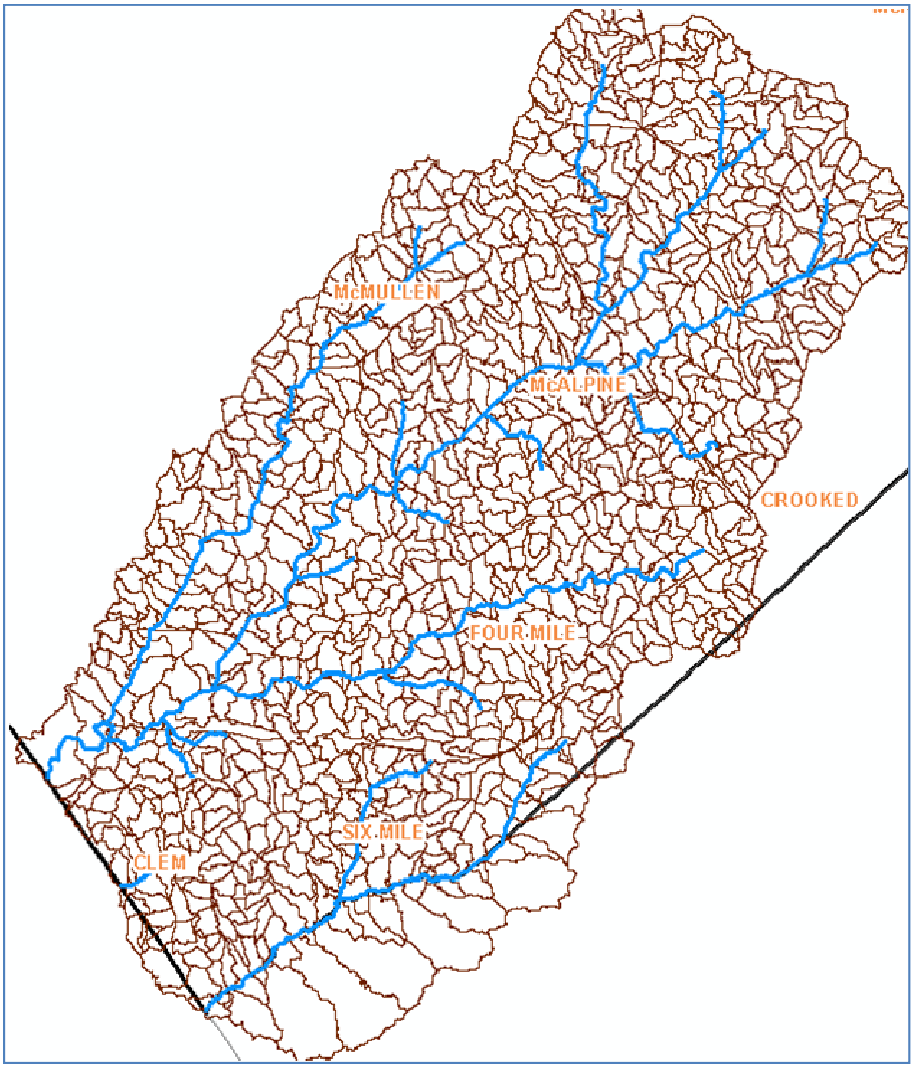


Figure 2. Catawba Sub-Basins

1.3 Soils

Soils in the Lower Catawba River Watershed fall in the central area of the Blue Ridge/Piedmont hydrologic region of North Carolina. These soils are predominately Cecil Sandy Clay Loams and are classified as Hydrologic Soil Group (HSG) B. The Cecil soils make up approximately 60% of the total watershed area.

Other soils located in the Catawba River watershed in Mecklenburg County are the Enon Sandy Loam (En Series), Monocam Loam (MO series), Vance Sandy Loam (Va series), all HSG-C soils. There are also areas of Wilkes soil (Wk series), which belongs to HSG-D.

1.4 Land Use

Land use is often used in floodplain analysis as an indicator of the percent imperviousness of a watershed, which has a significant effect on subsequent surface runoff and associated hydrologic peak flow calculations. The Mecklenburg County Effective Flood Insurance Rate Maps (FIRMs) include floodplain mapping based on both existing and future land use conditions. The existing and future land use layers were used with land use-soil type lookup tables (provided by CMSWS) to develop curve number calculations for hydrologic modeling.

The existing land use layer was obtained from the CMSWS. This layer was used as the base layer and it was reviewed and modified using the most recent aerial photography. Any discrepancies were brought to the attention of CMSWS to resolve. A task force was also involved in the QA/QC of the land use data and over a period of several months, reviewed and verified the data. The task force formally approved the existing land use data on February 17, 2010. Please see the Floodplain Analysis and Mapping Standards Guidance Document (FAMSGD) for more detail.

The future land use layer was obtained from CMSWS for the City of Charlotte ETJ. The towns of Pineville, Mint Hill, and Matthews each submitted separate future land use files. The separate future files were manipulated and then translated into one seamless layer in order to have the same attributes as the existing land use layer. The future layer was then modified and verified using a similar process as the existing layer. The task force formerly approved the future land use data on February 17, 2010. Please see the Floodplain Analysis and Mapping Standards Guidance Document (FAMSGD) for more detail. A detailed description of the fields in the existing and future land use layers is presented in Table 2.

Six Mile Creek drains some area in Union County and the Town of Weddington. As such, an existing land use file was obtained from the Town of Weddington and translated to mimic the updated Mecklenburg County existing land use file. The remaining area in the county was assigned existing land use by using the aerial data provided by Mecklenburg County. Union County did possess a broad future land use file that was used a base file which was again translated to mimic the Mecklenburg County future land use file.

Section 2 Data Used in Analysis

2.1 Mecklenburg County GIS Data

Topographic data was furnished by Mecklenburg County in the form of LIDAR .las files. This data was used in boundary delineation, stream line editing, digital cross section generation, and delineation of the time-of-concentration flow paths. Planimetric data, including streets, streams, and a jurisdictional layer was also furnished by the county.

The storm drainage infrastructure inventory was obtained from archives of the effective study. This file was reviewed and QC'd by the county in the field and each structure survey was verified. If it was not verified in the field it was flagged for a new survey. The 'new' surveyed structures were merged with the approved effective structure data and a new infrastructure inventory file was created.

The aerial photography used is a combination of the 2007 leaf-off imagery and the 2008 leaf-on imagery, both provided by CMSWS. The 2009 data was not ready for use at the beginning of the project and only became available near the beginning of 2010.

Table 2: Field descriptions for Existing and Future Land Use Layers		
	Existing Land Use Layer	Future Land Use Layer
Field name	**Field Description**	**Field Description**
FID	Field created by ArcGIS to provide a unique ID for each row in the table	Field created by ArcGIS to provide a unique ID for each row in the table
Shape	Field created by ArcGIS that indicates the type of geometry (i.e. Polygon)	Field created by ArcGIS that indicates the type of geometry (i.e. Polygon)
ObjectID	Field created by ArcGIS to provide a unique ID for each row in the table	Field created by ArcGIS to provide a unique ID for each row in the table
ACRES	Area of polygon	Area of polygon
LU_CODE	Number assigned based on 12 land use categories	Number assigned based on 12 land use categories
LU_DESC	Land use description (i.e. WOODS/BRUSH, etc)	Land use description (i.e. WOODS/BRUSH, etc)
LU_SOURCE	Source of land use description (i.e. TASKFORCE, etc)	Source for land use description (i.e. TASKFORCE, etc)
DATE_CRRNT	Contains the most recent date that the LU_DESC was edited.	Contains the most recent date that the LU_DESC was edited.
NOTES	Notes were inserted into the field if applicable.	Notes were inserted into the field if applicable.
PERCIMP	EXISTING percent of a catchment area that is made up of impervious surfaces such as roads, roofs, etc. (i.e. Transportation has 80% impervious area)	EXISTING percent of a catchment area that is made up of impervious surfaces such as roads, roofs, etc. (i.e. Transportation has 80% impervious area)
NOTES2	N/A	Notes were inserted into the field if applicable. NOTES2 was added if additional space was needed
PAST_DESC	N/A	Original Land Use description before translation, preserved for reference.
CRRNT_DESC	N/A	One of the twelve land use descriptions assigned after translation
Futr_Imper	N/A	FUTURE percent of a catchment area which is made up of impervious surfaces such as roads, roofs, etc. (i.e. Transportation has 80% impervious area)
ChngInImpe	N/A	Change in percent impervious area from Existing to Future Land Use

2.2 SCS Soil Data

Soils information was obtained from the Mecklenburg County Soil Survey (US Department of Agriculture, October, 1975). This information was intersected with the basin and land use files, and then the look-up tables were applied to get a composite curve number for each sub-basin.

2.3 Rainfall Data

Intensity-Duration-Frequency (IDF) information presented in the Charlotte-Mecklenburg Storm Water Design Manual (CMSWDM) (dated 1993) specifies precipitation depths to be used for the various design storm events (e.g. 2- through 100-year storms) and patterns. The rainfall depths presented in CMSWDM were compared with results of a recent United States Geological Survey (USGS) precipitation study (SIR 2006-5017) prepared in 2006. The USGS study developed several independent families of IDF curves based on different precipitation gage networks and data samples. Based on a comparison and evaluation of precipitation depth sources and recommendations in the USGS publication, it was deemed that the 24-hour precipitation depths from the combined “NOAA dataset plus aggregated USGS site representing the “CRN initial dataset” family with no areal reduction factors (presented in Table 3), hereafter referred to as the “combined” dataset, should be used for the Floodplain Mapping Project.

Table 3. Precipitation Depths for the Floodplain Mapping Project	
Storm Event	Precipitation Depth (inches)
50%	3.06
20%	4.08
10%	4.80
4%	5.76
2%	6.51
1%	7.29
0.2%	9.23
1/3 PMP	13.5

NOTES: Precipitation values taken from combined "NOAA dataset plus aggregated USGS site" IDF presented in SIR 2006-5017

The USGS combined precipitation depths are slightly higher in the 100-year storm, but equal to or slightly lower in the smaller (higher frequency) storms, than those presented in the CMSWDM for a 24-hour storm duration. The 1/3 Probable Maximum Precipitation (PMP) was also applied to the HEC-HMS models, a precipitation depth of 13.5 inches was provided by the county and applied to all models.

2.4 USGS Stream / Rainfall Gages

Mecklenburg County has an extensive collection of USGS gages in and around the county. Rainfall data in 5 minute increments were requested from 26 rain gages throughout the county. Data were received for the following storms:

August 28, 1995
July 24, 1997
February 1, 2008
April 26, 2008
August 27, 2008
November 30, 2008

February 28, 2009
July 27, 2009

It was determined that we would calibrate to the August 27, 2008 and the July 27, 2009 storms. The August 1995 and July 1997 storms were used to compare our calibrated models to after the fact. Those historical events were not used directly in the calibration process. Please see the calibration section for more detail.

The county also has an extensive stream gage network, flow and stage data were requested from 11 USGS stream gages throughout the county. Stream gage data in 15 minute increments were received for the same storms as mentioned above.

The Catawba watershed used 7 stream gages and 17 rain gages in its calibration routine, which is discussed below.

2.5 Time of Concentration / Lag Time

Time of Concentration values were calculated using the method described in Chapter 3, Urban Hydrology for Small watersheds (Technical Release 55), Natural Resource Conservation Service (1986). The time of concentration is computed using sheet flow, shallow concentrated flow and channel flow. A maximum flow length for sheet flow in urban areas is 100 feet and in rural areas is 300 feet.

Section 3 Description of Hydrologic Modeling

3.1 Model Used

The hydrologic modeling for the Catawba River Watershed in Mecklenburg County was performed using the USACE Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS), Version 3.40. Peak flood discharges with 50%, 20%, 10%, 4%, 2%, 1%, and 0.2% annual chance exceedance were modeled for this study.

Future Conditions Model: A future conditions HEC-HMS model was created in a similar fashion as the existing conditions model. The only parameter adjustment in the initial creation of the future conditions model was the use of the future land use layer to calculate future conditions curve numbers. These curve numbers were used in the model to create full build out of the watershed. The time of concentration and initial abstraction used in the future conditions model were taken from the calibrated existing conditions model.

3.2 HEC-HMS Model Assumptions and Limitations

The HEC-HMS model is a mathematical representation of the hydrologic process and it is to be used to perform the computations for three basic functions;

- Compute losses and generate a runoff hydrograph;
- Combine hydrographs;
- Route hydrographs through channels, structures, ponds, and detention basins.

These functions are combined in a logical manner to model a particular watershed. In order to use the HEC-HMS model correctly and evaluate the results, it is important to understand the limitations of the models use and its underlying theoretical assumptions. The general assumptions and limitations of the HEC-HMS model are as follows:

- Stream flow routings use hydrologic routing methods and do not reflect the full Saint-Venant equations;
- Simulations are limited to a single storm event. The model does not have the capability of accounting for soil moisture storage or depletion between rainfall events, and;
- Storage facilities must be described with a single stage – discharge and stage – storage relationship.

The theoretical assumptions that govern the model's applicability to a specific watershed are as follows:

- The watershed can be represented as an interconnected group of catchment areas;
- The hydrologic process can be represented by the model parameters which reflect average conditions within a catchment area;
- Model parameters represent temporal and spatial averages;
- Rainfall and losses are uniformly distributed across the catchments per a weighted gage analysis, and;
- All runoff from a catchment area (sub-basin, basin, and watershed) eventually goes to the same outfall point.

Additional model assumptions specific to the Catawba River Watershed in Mecklenburg County are:

- The modeling procedure used in this project followed the "SCS Methodology". This terminology covers a wide range of procedures relating to rainfall and losses, runoff and hydrograph routing, and use of the SCS Unit Dimensionless Hydrograph to develop runoff hydrographs.
- The 24-hour Type II rainfall distribution was used for all design frequency simulations.

3.3 HEC-HMS Model Parameter Development

Rainfall Data: Rainfall depths for the 50%, 20%, 10%, 4%, 2%, 1%, and 0.2% annual chance exceedance storm events were obtained from the FAMSGD and are listed above in Table 2. These depths were converted by HMS into a Type II rainfall distribution that was used in the modeling.

Drainage Areas: Drainage basin boundaries for the Catawba River Watershed in Mecklenburg County were delineated using a 10' x 10' grid size digital elevation model (DEM) generated from the LIDAR data collected and processed in 2008 and supplemented with data from LIDAR flown in 2004. From this hydrologically correct DEM a total of 1006 basins were delineated based on stream crossings and location in the watershed. The sub-basins averaged 76 acres in size and ranged from 2 acres to 828 acres. Each sub-basin was assigned a unique numeric identifier. See Figure 2.

Runoff Curve Numbers: A weighted runoff curve number was calculated for each sub-basin by using an intersection of soils data, land use data, and sub-basin boundary data. The intersection references a 'look-up' table of curve numbers for various soil and land use category combinations and assigns a runoff curve number to each polygon within a sub-basin. For a given sub-basin, the individual runoff curve numbers are multiplied by the drainage area of the polygon they represent and the results are summed and divided by the total drainage area of the sub-basin. The resultant runoff curve number is the weighted runoff curve number for the sub-basin. See the attached spreadsheet *'Catawba Curve Numbers'* for individual basin curve numbers and comparison of existing and future curve numbers.

The 'look-up' table of curve numbers was created using TR-55 Table 2-2a ***Runoff Curve Numbers for Urban Areas*** as a base but then adding in the percent impervious assumptions from the land use data, i.e. woods/brush is 5% impervious. The adjusted look-up table can be seen in Table 4 below.

Time of Concentration / Lag Time: Time of concentration is the time required for a drop of water (during a 50% chance event) to travel from the hydraulically most remote part of a catchment to its outfall. The time of concentration has three associated flow path components:

1. Sheet flow,
2. Shallow concentrated flow, and
3. Channel flow.

These three components are calculated individually and summed to obtain the time of concentration for the sub-basin. The length of the sheet flow segment for a sub-basin is limited to 100 feet for urban areas and 300 feet for rural or undeveloped areas. The shallow concentrated flow segment extends from the downstream end of the sheet flow segment to a defined swale or pipe system.

Table 4: Master Curve Number Table

		Curve Number for hydrologic soil group with AMC2 conditions						
Land Use Code	Land Use Description	A Ex/Fut	B Ex/Fut	C Ex/Fut	C/D Ex/Fut	D Ex/Fut	U Ex/Fut	W
1	Woods/Brush	33	57	71	75	78	57	98
2	> 2 ac Residential & Open Space	44	65	77	80	82	65	98
3	0.5 to 2 ac Residential	51/53	68/70	79/80	82/82	84/84	68/70	98
4	0.25 to 0.5 ac Residential	56/59	71/74	81/82	83/84	85/86	71/74	98
5	< 0.25 ac Residential	59/64	74/77	82/84	84/86	86/88	74/77	98
6	Institutional Areas	69	80	86	88	89	80	98
7	Industrial-Light	74	83	88	90	91	83	98
8	Industrial-Heavy	81	88	91	92	93	88	98
9	Commercial-Light	83	89	92	93	94	89	98
10	Commercial-Heavy	92	94	95	96	96	94	98
11	Standing Water	98	98	98	98	98	98	98
12	Transportation	86	91	93	94	94	91	98

The channelized flow segment extends from the downstream end of the shallow concentrated segment to the outfall of the sub-basin.

$$T_C = T_S + T_{SC} + T_{CH}$$

The time of concentration routine uses the triangular irregular network (TIN) and calculates the longest path for each sub-basin and stores them in a database and a shapefile. For each sub-basin this routine produces a single shallow concentrated flow path, categorized as either paved or unpaved. Each flow path therefore represents the area that it spends the most time traversing. The shallow concentrated flow paths were verified using aerials and contours to make sure they represent the majority of the sub-basin. However, if the shallow concentrated flow paths traveled over a different surface for greater than 20% of the total distance, an attempt was made to capture that change of land cover in the calculations by dividing the shallow concentrated flow path into separate sections of paved and unpaved, with subsequent calculations then being performed accordingly.

The equations used in the time of concentration calculations are as follows:

1. Overland Flow

$$T_i = [0.007(nL)^{0.8}] / [P_2^{0.5} * S^{0.4}]$$

Where: n = sheet n based on land use
L = Length (100' or 300')
P_2 = 2yr. 24hr rainfall = 3.06
S = Slope

2. Shallow Concentrated Flow

Velocity Calculation for Paved Surfaces: $V = 20.3282 * S^{0.5}$
Assumes n=0.025 and r=0.2

Velocity Calculation for Unpaved Surface: $V = 16.1345 * S^{0.5}$
Assumes n=0.05 and r=0.4

Where: S = Slope

3. Channel Flow

Velocity Calculation: $V = (1.486/n) * R^{2/3} * S^{1/2}$

Where: n = Manning's roughness based on drainage area
R = Hydraulic Radius based on drainage area
S = Slope

The flow paths and associated travel time calculations through ponds and lakes are calculated using a constant velocity of 1.0 ft/s.

Lag time (T_L), or the time which elapses between the center of mass of the rainfall and the peak runoff, is derived from the time of concentration based on the empirical relationship of $T_L = 0.6*T_C$ documented in the HMS User's Manual.

Time of Concentration results for individual basins can be seen in the attached database called '*Catawba_TC_Database.mdb'.*

Channel / Structure Routings: The modified puls method was used for routing calculations in all stream channels because we feel that it gives the modeler the most versatility. In streams that have an effective HEC-RAS model, the storage-outflow parameters were initially used to balance the new model. See the FAMSGD for more detail. In the upper headwater reaches of the watershed where no effective RAS model exists, Manning's equation is used to calculate a range of discharges based on a range of water depths in the routing cross section in that sub-basin. This routing cross section is considered to be an average or "representative" cross section, characterizing the general geometry of the floodplain in that sub-basin. In some cases more than one cross section was placed to get a better representation of the channel. The maximum elevation along the cross section is divided by 10 to come up with a range of water surface elevations, with each elevation then being used to calculate an associated storage volume and discharge. From this, a storage / outflow rating curve for the sub-basin can be developed. New updated RAS models have been created and have been used to balance the HEC-HMS peak discharges with the HEC-RAS peak water surface elevation results, as recommended in the FAMSGD, until the difference between peak discharges in successive runs is less than 10%. In reaches where there is an updated RAS model the storage – discharge curves were taken from the RAS model itself and input into the HMS model.

Section 4 Model Calibration

Model calibration refers to adjustment of model parameters so that simulated stream flow computed using observed rainfall as inputs to the hydrologic model is in agreement with observed stream flow. Model calibration is outlined in a systematic procedure in the FAMSGD. For watersheds with historical precipitation and gage data this procedure suggests that curve numbers be adjusted by +/- 4 so that total runoff volume matches as close as possible at measured locations. The next step is to adjust time parameters to help match time to peak and then cross check with regression equations. Finally, other hydrologic parameters can be considered if necessary and justifiable.

There are several USGS stream and rain gages for the calibration of model parameters in the Catawba River sub-basin portion of the study area (Tables 5 & 6). The stream gages are located on McAlpine, Irvins, Campbell, McMullen, and Six Mile Creeks. The precipitation gages are located within or in close proximity to the Catawba sub-basin.

Table 5: Stream Gages used for Catawba River sub-basin Model Parameter Calibration

Gage Station ID	Gaged Stream and Location	Drainage Area (square miles)
02146562	Campbell Creek near Charlotte, NC	5.6
0214657975	Irvins Creek at SR 3168 near Charlotte, NC	8.4
02146700	McMullen Creek at Sharon View near Charlotte, NC	7.0
0214655255	McAlpine Creek at SR 3150 near Idlewild, NC	7.5
02146600	McAlpine Creek at Sardis Rd near Charlotte, NC	39.6
02146750	McAlpine Creek below McMullen Cr near Pineville, NC	92.4
0214685800	Six Mile Creek near Pineville, NC	20.3

The precipitation and stream flow data for several large storms that occurred between 2003 and 2009 were reviewed and considered for use in model calibration and parameter calibration procedure. Criteria for selection of storm events were:

- Complete data sets;
- Simple, single peak hydrographs;
- Sufficient separation between storm events; and
- Some range in peak rainfall accumulation.

Table 6: Precipitation Gages used for Catawba River sub-basin Model Parameter Calibration

Gage Station ID	Gaged Stream and Location
351812080445545	CRN-07 Raingage at Fire Station 9 Charlotte, NC
351540080430045	CRN-08 Raingage at St Matthews Church
351302080412701	CRN-09 Raingage at Fire Station 15 Charlotte, NC
351218080331345	CRN-16 Raingage at Reedy Creek Park Enviro Center

351455080374445	CRN-17 Raingage at Piney Grove Elementary School
351028080385545	CRN-20 Raingage at Fire Station 14 Charlotte, NC
352000080414645	CRN-31 Raingage at Elon Parks and Rec Center
350627080410645	CRN-32 Raingage at, Bain Elementary School
351536080410645	CRN-39 Raingage at NCDOT Facility Matthews, NC
350857080383245	CRN-47 Raingage at Winterfield Elementary School
351145080371945	CRN-48 Raingage at Old Providence School
350627080410645	CRN-56 Raingage at South Meck High School
351536080410645	CRN-57 Raingage at Lebanon Rd Elementary School
350857080383245	CRN-67 Raingage at Thompson Road Mint Hill, NC
351145080371945	CRN-69 Raingage at Matthews Elementary School
351145080371945	CRN-70 Raingage at Providence High School

After review of available precipitation and stream flow data, two storm events were selected for use in the model calibration parameter calibration exercise. The storm event peaks occurred near August 27, 2008 and July 29, 2009 (Table 7). These storm events were selected to include a representative range of peak discharges that varied from the 50 percent annual chance event to just under the 0.2 percent annual chance event as measured in inches of rain for the specific event.

Table 7: HMS Control Specifications

Storm	Begin	End	Rainfall (In)	Hypothetical Storm Event
Campbell Creek Aug 2008	8/25/2008 12:00	8/29/2008 00:00	8.76	<500-yr
Campbell Creek July 2009	7/27/2009 12:00	7/30/2009 00:00	3.36	>2-yr
Irvins Creek August 2008	8/24/2008 00:00	8/29/2008 00:00	6.28	<50-yr
Irvins Creek July 2009	7/26/2009 12:00	7/31/2009 15:00	4.11	>5-yr
McMullen Creek August 2008	8/25/2008 00:00	8/29/2008 00:00	9.01	<500-yr
McMullen Creek July 2009	7/26/2009 12:00	7/29/2009 15:00	3.21	>2-yr
McAlpine Creek US Aug 2008	8/25/2008 12:00	8/29/2008 00:00	8.04	>100-yr
McAlpine Creek US July 2009	7/26/2009 00:00	7/31/2009 00:00	4.61	<10-yr
McAlpine Creek DS Aug 2008	8/25/2008 12:00	8/29/2008 12:00	5.89	>25-yr
McAlpine Creek DS July 2009	7/26/2009 12:00	7/31/2009 00:00	2.41	< 2-yr
Six Mile Creek Aug 2008	8/25/2008 00:00	8/29/2008 00:00	4.01	<5-yr
Six Mile Creek July 2009	7/27/2009 00:00	7/31/2009 00:00	1.27	< 2-yr

With the watershed in question being so large and diverse it is very difficult to get consistent storm data over the entire area. The July storm does not register as a 2-year event in the downstream end of McAlpine and Six Mile Creeks. The difficulty of modeling to two separate storms can be seen in the Campbell Creek model. The August storm registers as about a 500-year event while the July storm is merely a 2-year event. The parameters of the watershed will act differently for these two storms where the initial abstractions are usually higher for larger events over a shorter time period. We tried to balance the models so the model could handle a short large event, like the August 2008 storm, while also handling a longer, smaller event, like the July 2009 storm. Although, we may not have exactly achieved the 10% tolerances after the overall calibration was applied, we believe we have found a middle ground so the models produce stable results for both small and large rain events.

Table 8 represents observed runoff and flows used in the calibration runs. It is interesting to note that McAlpine Creek has an observed runoff which decreases as it progresses further downstream for the August 2008 storm (3.50 to 3.01), even though the observed peak flow increases. Such an occurrence can happen when precipitation in the upstream sub-basins is of a higher intensity than those toward the lower reaches. Figure 3 shows the total rainfall amounts over the McAlpine watershed for the August 2008 storm event as calculated using the Thiessen polygon method on the available rain gages.

Calculation of observed runoff is done by taking the entire volume of runoff at a precise location, typically measured in cfs, and dividing it by the total amount of contributing drainage area, usually in sq. mi. These computations are made with the proper unit conversions and result in a final runoff, normally expressed in terms of inches.

Table 8: Peak Discharge Events used for Catawba River sub-basin Model Parameter Calibration

Date	Gaged Stream and Location	Observed Runoff (inches)	Observed Peak Discharge (cubic feet/second)
8/27/08	Campbell Creek at SR 3181 near Mint Hill, NC	3.51	2,000
7/28/09	Campbell Creek at SR 3181 near Mint Hill, NC	0.92	475
8/27/08	Irvins Creek below I-485 near Pine Ridge, NC	2.25	2,090
7/28/09	Irvins Creek below I-485 near Pine Ridge, NC	0.71	689
8/27/08	McMullen Creek at Sharon View near Charlotte, NC	5.37	4,020
7/28/09	McMullen Creek at Sharon View near Charlotte, NC	0.85	710
8/27/08	McAlpine Creek at SR 3150 near Idlewild, NC	3.50	3,100
7/28/09	McAlpine Creek at SR 3150 near Idlewild, NC	1.19	680
8/27/08	McAlpine Creek at Sardis Rd near Charlotte, NC	3.38	6,330
7/28/09	McAlpine Creek at Sardis Rd near Charlotte, NC	1.18	2,790
8/27/08	McAlpine Creek below McMullen Cr near Pineville, NC	3.01	7,870
7/28/09	McAlpine Creek below McMullen Cr near Pineville, NC	0.75	1,940
8/27/08	Six Mile Creek near Pineville, NC	0.68	433
7/28/09	Six Mile Creek near Pineville, NC	0.07	121

In the headwater reaches of the McAlpine watershed there are higher amounts of precipitation as compared to the lower reaches. When these precipitation quantities are used to factor the runoff value, it is understandable that the observed volume at the upstream gage will be greater than the observed volume at the downstream gage since there is significantly greater drainage area measured there.

4.1 Calibration Precipitation Input

An area-weighted, spatially distributed precipitation record was developed for use as precipitation input for the model calibration and parameter calibration process. The observed point precipitation data at selected USGS precipitation gages (Table 6) was transformed to an area-weighted, spatially distributed precipitation record using an area weighted Thiessen polygon method. Thiessen polygons are defined as a set of polygons that enclose the areas around a set of point locations (such as a group of rain gages) so that for a given point location the associated Thiessen polygon includes all the area that is less than half way between the selected point and all the remaining points. As such, all locations within a given polygon are closer to the associated rain gage than to any of the other rain gages. Thiessen polygons for the selected precipitation gage location were developed using GIS tools.

The Thiessen polygons were then intersected with the drainage sub-basins for each studied watershed. The weighted precipitation for each sub-basin is computed as the weighted average of the observed rainfall at each gage for which the sub-basin intersects an associated polygon. The weighting factor for the associated rain gages is computed as the percent of the total area of the sub-basin that is contained in the associated rain gage polygon. In order to develop a weighted, distributed precipitation input, the weighted average was computed for each time step in the rain gage record.

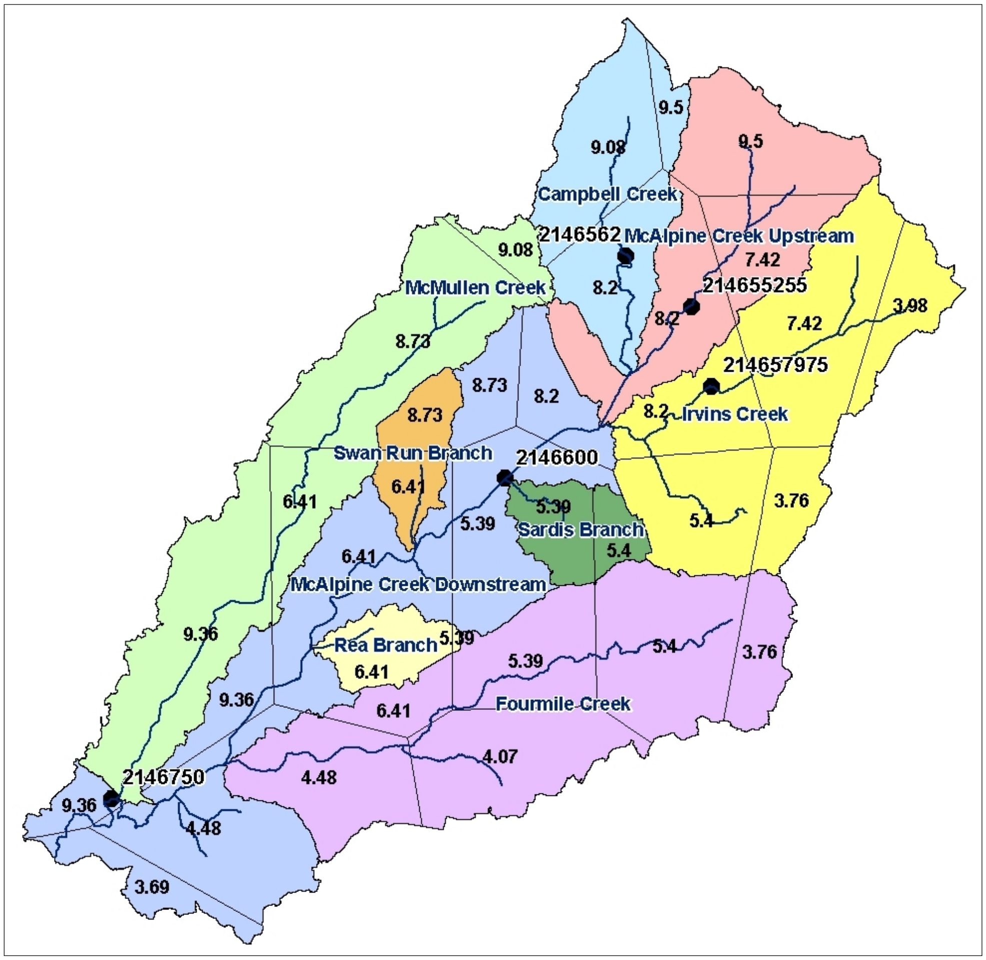


Figure 3. McAlpine Creek Watershed Precipitation Coverage – August 2008

After some investigation into the shape of the McMullen Creek watershed and into how the precipitation fell in both storms, it was decided to adjust the McMullen Creek gage weights. As seen in the graphic above which shows the Thiessen polygon weights, the 6.41 inches that was measured at CRN-48 protrudes into the McMullen Creek basin and lowers the overall average rainfall depth. Upon further investigation, it appears that the August 2008 storm concentrated its intensity along the McMullen Creek corridor and had less substantial rainfall totals to the south. It was decided to throw out the CRN-48 gage and just use the larger rainfalls for our weighting. The result was a total rainfall of 9.07 inches instead of 8.68 inches. The July 2009 storm was similarly impacted and adjusted from 2.32 to 3.21 inches.

4.2 Calibration Methodology

Calibration of the Campbell, Irvins, McMullen, McAlpine, and Six Mile Creek sub-basins in the Catawba River watershed was initiated by following the steps laid out in the FAMSGD. In general, "hydrologic calibration is typically performed by adjusting sub-basin lag times, initial abstractions, curve numbers, and/or peaking coefficients, as justifiable, to better match computed peak flows and hydrograph time to peaks with observed values or previous studies." In order to take advantage of the amount of gage data and to acknowledge that every sub-watershed reacts uniquely to each storm event, it was thought best to keep the calibration of each watershed separate and apply an average of the calibration to each of the remaining un-gaged watersheds in the Catawba watershed.

Calibration of Campbell, Irvins, Six Mile, McMullen, and McAlpine Creek US models were directly evaluated in the basic manner outlined above because a single stream flow gage is located on those streams. Campbell and Irvins are tributaries of the McAlpine Creek watershed, which complicates the overall McAlpine modeling procedure. Another degree of difficulty occurs due to the existence of numerous stream flow gages located within the McAlpine Creek watershed, we looked at a total of 5. It was decided that the McAlpine Creek watershed would be modeled and subsequently calibrated in distinct sections. These divisions have been made at a tributary sub-region level as follows;

Campbell Creek
Irvins Creek & Tribs
Sardis Branch
Swan Run Branch
Rea Branch
Four Mile Creek
McMullen Creek

In addition, the main stem of McAlpine Creek was also separated into two portions, an 'upstream' and 'downstream' component. The 'upstream' begins at the confluence of Irvins Creek with the furthest downstream point corresponding to sub-basin "*BASIN926*", and continues to the most upstream end of McAlpine Creek. For the 'downstream' section the outlet in sub-basin "*BASIN1020*" is where it commences, and concludes at the confluence of Irvins Creek in sub-basin element "*BASIN938*".

A supplementary purpose to those listed above for dividing the McAlpine Creek watershed is the processing time which arises in HMS due to the sheer size of the McAlpine Creek area. HMS is a data intensive tool; meaning it executes copious amounts of calculations, particularly in the calibration routine, in order to attain results. Typically the program is utilized to perform these analyses on small regions. McAlpine is not a typical small area. Due to limitations on the size of individual sub-basins there are approximately 831 sub-basin elements, representing approximately 93.45 sq. miles, which exist in a single model. Couple those with the abundance of necessary routing reaches, as well as adding in subsequent junctions located at confluences of each smallest tributary, the weight of data being processed becomes immense. Each aforementioned element contains numerous data parameters, all of which are taken into account in the calculative sub-routines of the program, thus resulting in excessive calculations and processes, which in turn require time and effect staggering delays in HMS modeling functionality. By extracting pertinent zones of the whole watershed into several models, the full sum of necessary calculations is decreased exponentially, thus reducing the delay in program performance.

A concern with having the watershed divided into various sectors is that there may be a dis-connect in the overall study, which may result in inaccurate evaluations. Efforts must and have been taken to ensure universal congruity. These efforts include an essential inclusivity of all tributary models' results within the central McAlpine Creek stream, in this case the referred *McAlpine Creek Downstream* model. HMS provides a straightforward way of addressing this issue; it allows for source input elements to reference and consequently incorporate results from the models representing the assorted tributaries. This procedure was done for *McAlpine Creek Downstream* as well as *McAlpine Creek Upstream*. The upstream segment of McAlpine Creek integrated the results from the *Campbell Creek* model, while the downstream portion included all others listed above, as well as the upstream segment of McAlpine.

The process in calibrating the McAlpine Creek watershed in its entirety became more multifaceted with the detachment of the large tributaries into their own models. Since some of these streams do not contain an observation gage, there is no calibration routine available to the sub-basins contained therein. However, they must be factored to some degree similar to those sub-basins in the McAlpine Creek main channel. In order to effectuate a proper adjustment to the parameters of elements in the tributary models, the main branch of McAlpine Creek was calibrated normally; and the resolved factors regarding curve numbers and initial abstraction were then applied to those of the contributing streams that do not have gages. Those models were then rerun, with the appropriate results being imported back into McAlpine Creek main branch for a new regular simulation run to analyze overall effects of the calibrated factors.

We began each calibration routine by running the models without any calibration and those results are noted below as compared to observed flow at the respective gages.

Table 9: Initial Simulated and Observed Runoff and Peak Discharge for Stream Gages Used in Catawba River Sub-basin before Calibration

Date	Gaged Stream and Location	Observed Runoff (inches)	Simulated Runoff (inches)	% Difference from Observed Runoff	Observed Peak Discharge (cubic feet/second)	Simulated Peak Discharge (cubic feet/second)	% Difference from Observed Peak
8/27/08	02146562 (Campbell Ck)	3.51	6.15	75.2%	2,000	2,857	42.9%
7/29/09		1.06	1.78	67.9%	475	899	89.2%
8/27/08	0214657975 (Irvins Ck)	2.25	3.00	33.3%	2,090	2,524	20.8%
7/29/09		0.81	1.40	72.8%	689	1,279	85.7%
8/27/08	0214655255(McAlpine Ck)	3.50	5.56	58.9%	3,100	3,990	28.7%
7/29/09		1.19	2.18	83.2%	680	1,637	140.7%
8/27/08	02146600 (McAlpine Ck)	3.39	4.51	33.0%	6,330	10,922	72.5%
7/29/09		1.18	1.51	28.0%	2,790	4,192	50.3%
8/27/08	02146700 (McMullen Ck)	5.37	6.42	19.6%	4,020	4,132	2.8%
7/29/09		0.85	1.31	54.1%	710	892	25.4%
8/27/08	02146750 (McAlpine Ck)	3.02	4.15	37.4%	7,870	15,064	91.4%
7/29/09		0.75	1.07	42.7%	1,940	3,910	101.5%
8/27/08	0214685800 (Six Mile Ck)	0.68	0.89	30.9%	433	826	90.7%
7/29/09		NA	NA	NA	NA	NA	NA

As displayed in the table above, un-calibrated results show that all volumes and peaks are initially high as compared to observed gages for the August 2008 and July 2009 storms. We wanted to make sure that these results were reliable so we double checked the precipitation data and how we applied the Thiessen polygon weights. Essentially, the Thiessen polygon weighting routine works as an areal reduction factor. It uses a weighted average rainfall from the nearest rain gages and applies that to each of the sub-basins in the watershed. The rain totals can be seen in table 7 and compare favorably to the USGS Rainfall Distribution map provided for the August 2008 storm in Appendix A.

As suggested in the guidance document, we first reduced all curve numbers in each watershed by 4 and recalculated the initial abstraction using the default equation IA = 0.2*S, where S is based on the curve number. In most cases the percent differences were 20% or more, but in the McMullen Creek watershed that was not the case. The McMullen Creek watershed is a unique watershed and it appears it does not require much calibration to match the volume and peak of the observed storms. This is true for both the August 2008 and the July 2009 storm as the peak discharges are overestimated by the smallest amount for the respective storms in the McMullen watershed. Initially, we were concerned that there was a gage reporting error or perhaps some other factor that produced these abnormal results, but when we looked at the median flood as unit discharge data for gages in the area, it confirmed our findings that McMullen Creek produced very high flows and volumes. See Table 10 for the median flood as unit discharge information for surrounding streams.

The McMullen Creek median flood is more than twice that of McAlpine Creek and is even 40% higher than Little Sugar Creek. These calculations are based on annual flood peaks from stream gauging observations from 1962 to 1995. We believe there are many factors that contribute to this and they could include:

- the watershed shape,
- how and when the watershed was developed,
- storm drain pipe density, and
- the watershed is considered to be at built out conditions.

Table 10: Median Flood as a unit discharge

Sub-Watershed	Gage	DA (mi^2)	Median Flood CFS/mi^2
Little Sugar	2146507	42.47	18.95
McAlpine	2146600	39.77	11.73
McMullen	2146700	6.95	26.86
Sugar Irwin	2146300	30.50	16.63
Long	2142900	16.60	12.14

Note: Table adapted from "The Regional Hydrology of Extreme Floods in an Urbanizing Drainage Basin", Smith et al.

An additional gage analysis of the McMullen gage along with the two gages on McAlpine also pointed to the abnormalities in the McMullen watershed. The direct quote from our senior hydrologist was "… the problem is the data (for McMullen Creek) continues to trend upward, so you can't really include the entire period of record." His best estimates of 100-yr flows using gage analysis can be seen in table 11. Q100 represents the estimated 100-year storm event and q100 represents a unit discharge per square mile for the 100-yr event. These estimates also conclude that McMullen Creek is an aberration as compared with the other gaged watersheds. This flood frequency analysis was performed on the gage records using the log-Pearson Type III distribution, in accordance with Bulletin 17b guidelines. The station skew option was used because the flows are all affected by urbanization to some degree and as such are not reflective of the generalized skew values computed for unregulated rural streams in the area. Because of the non-uniform nature of the flow record, a number of periods were analyzed and the 1975-2009 period was determined to produce the most reasonable results. These results represent a balance between achieving sufficiently long record length and capturing a relatively homogenous period of basin characteristics.

Table 11: AECOM Gage Analysis Results

Gage ID	Gaged Stream and Location	Dates Analyzed	DA ($miles^2$)	Q100	q100
02146700	McMullen Creek at Sharon View near Charlotte, NC	1975-2009	7.0	5,140	740
02146600	McAlpine Creek at Sardis Rd near Charlotte, NC	1975-2009	39.6	8,910	225
02146750	McAlpine Creek below McMullen Cr near Pineville, NC	1975-2009	92.4	11,600	126

We continued with the calibration routine for each gaged watershed as suggested in the guidance document. We first reduced all curve numbers in each watershed by 4 and recalculated the initial abstraction using the default equation IA = 0.2*S, where S is based on the curve number. As seen in the calibration spreadsheets, this adjustment did not reduce the volume or the flow enough of any watershed except for McMullen. Therefore, a larger adjustment was needed, and in order to justify changing the curve numbers by more than the guidance document suggests, we initiated a direct percent impervious calculation of all drainage areas draining to each gage. This calculation involved obtaining existing percent impervious layers from the county and supplementing them based on the 2009 aerials and the transportation layer. Using the 2009 aerials and the transportation layer, an updated impervious layer was created for all area that drained to the gages. Once the impervious layer was complete a simple calculation of the impervious layer area divided by the total area draining to the gage supplied us with an actual percent impervious for all watersheds that contain a gage. That comparison is seen in Table 12.

Table 12: Percent Impervious Estimated vs. Calculated

Watershed	Original CN	% Imp from Land Use	% Imp from Imp Layer	Percent Difference
Campbell	78.39	42.30	33.07	27.9%
McAlpine US	75.90	34.46	26.15	31.8%
McMullen	78.37	39.04	34.22	14.1%
Irvins	69.97	24.21	17.00	42.4%
Average	75.66	35.00	27.61	29.1%

The initial curve number for each sub-basin is a composite of the intersection of three layers; land use, soils, and sub-basins. This intersection file was used to calculate the overall percent impervious at each gage used in the original calculation. The land use file estimated a percent impervious from each land use and this estimate was used in the composite calculation for curve numbers. As shown in Table 12, the estimated percent impervious calculations from the land use layer results in a minimum over-estimate of 14.1% and a maximum of 42.4%. This over-estimate of percent impervious should allow for the reduction of the curve numbers in calibration by more than +/- 4 as recommended in the guidance document.

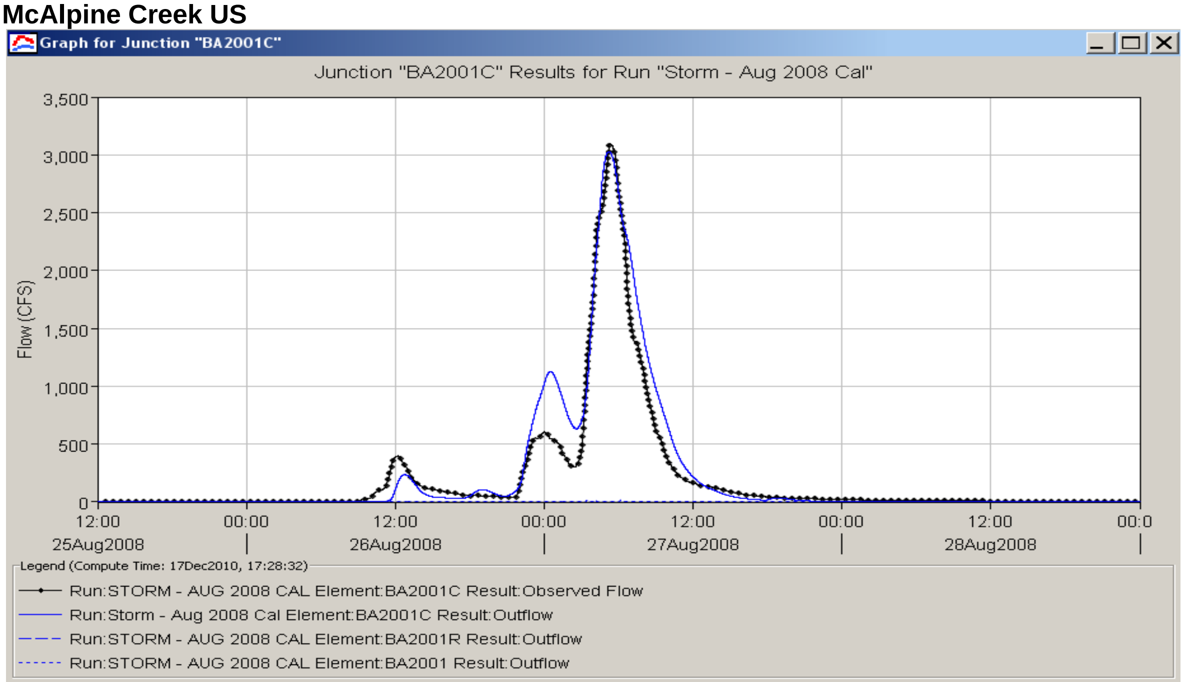
Campbell Creek: Campbell Creek appears to present another anomaly in the data from the observed flow gages. Campbell Creek is due west of the McMullen Creek headwater sub-basins and initially appears to be similar in nature to McMullen and McAlpine US. The observed flows for the August 2008 storm at their respective gages however tell a different story. Whereas McMullen Creek displays a higher flow than anticipated, Campbell Creek displays a much lower flow with an exact overall curve number. We have explained why we believe the McMullen watershed reacts the way it does but we had a more difficult time explain why Campbell Creek reacts with much lower flows. We do want to note, however, that these same trends can be seen in the effective flows as well. Table 13 reflects some of the data used in comparing the three watersheds.

Table 13: McMullen, Campbell, and McAlpine US Watershed Comparison

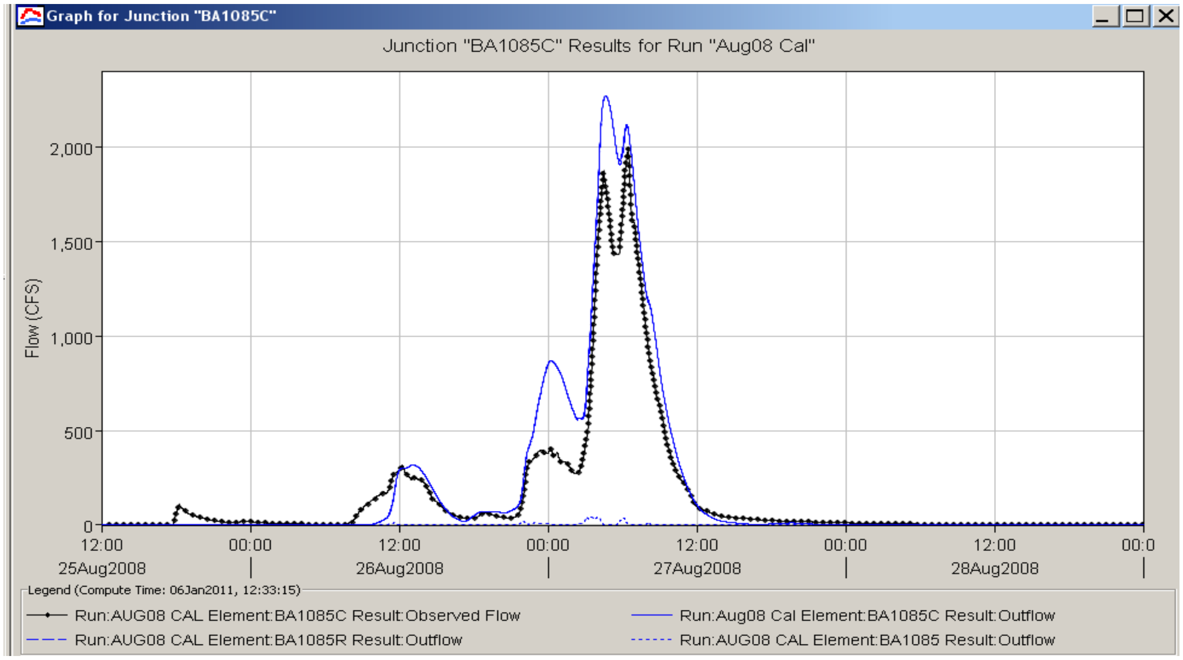
Watershed	DA @ gage (mi^2)	Aug 08 Rain (In)	CN @ Gage	Actual % Imp @ Gage	Avg Lag (min)	Obs Runoff (In)	Obs Flow (cfs)	Obs Flow per mi^2	DA, Rain, and CN equal	DA, Rain, and CN =, per mi^2	FIS Eff DA	FIS Eff flows
Campbell	5.6	8.76	78.4	33.1%	23.2	3.51	2,000	357	2,000	357	7.5	2,571
McAlpine US	7.5	8.04	75.9	26.2%	41.0	3.50	3,100	413	2,605	465	7.3	3,294
McMullen	7.0	9.01	78.4	34.2%	29.6	5.37	4,020	574	3,127	558	7.5	4,561

For the August 2008 storm event, if all parameters (drainage area, curve number, and rainfall) were equal for each basin, the Campbell Creek flow would be 30% lower than McAlpine US and 56% lower than the McMullen Creek flow. Again, we want to point out that the effective data does trend in a similar fashion, and we want to try and provide a possible explanation why. The Campbell Creek watershed initially seems similar to McMullen and McAlpine US but when the land use data is scrutinized some distinct patterns are revealed that may provide an explanation of why the Campbell Creek watershed reacts the way it does. We would like to present two separate but coupled theories, which may cause the noted discrepancy in flow. First, although the curve numbers are very similar for each watershed, we contend that specific land use details in how that curve number was formulated are vastly different for Campbell Creek. And second, that the non-homogenous make-up of Campbell Creek contributes to the dual peak that is seen in the hydrograph at the gage and that non-coincident peaking causes the peak flow to be lower in Campbell Creek versus McAlpine or McMullen, which display more traditional, single peaks for the August 2008 storm event. Let's look at the hydrographs at the gage for all three watersheds:

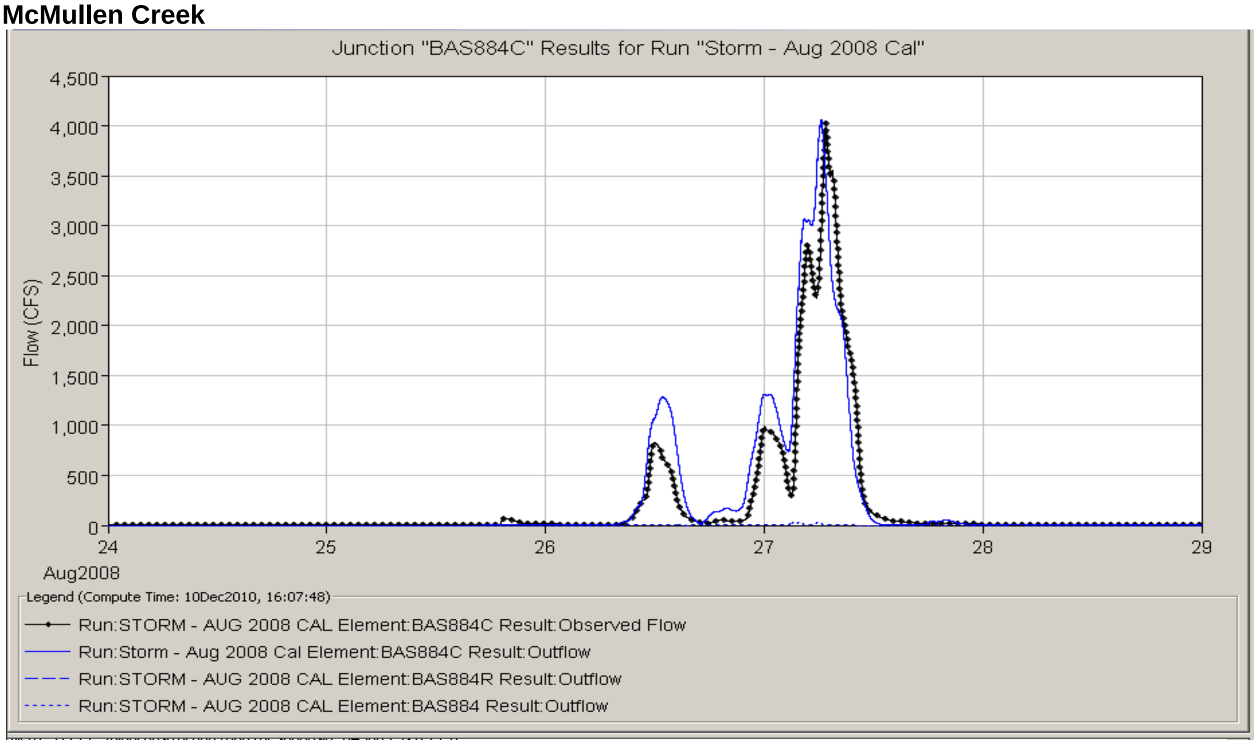
McAlpine Creek US



Campbell Creek



McMullen Creek



The black dotted line represents the observed flow data from the USGS at each respective gage. The hydrographs are similar in nature with two smaller peaks before the larger main peak. This demonstrates that the rainfall was similar for each of these watersheds. The McAlpine and Campbell Creek observed hydrograph is especially close, the two smaller peaks come at about the same time and with similar magnitude but then the Campbell Creek main peak is separated into a double peak. We believe this is due to how the physical watershed was/is built.

The watershed to the gage is basically split into two distinct drainage areas, the area upstream of Albemarle Road and the area from Albemarle Road downstream to the gage. The total area draining to the gage is 5.6 square miles with about half (2.9 square miles) of that upstream of Albemarle and the rest (2.7 square miles) downstream. Please note that the entire industrial corridor along Central Ave to the intersection with Albemarle (Eastland Mall) drains the downstream portion of the study and does not pass through the main channel culvert at Albemarle Road. The actual curve number and lag time breakdown for each of these separate areas is presented below:

Location	Avg CN	Avg Lag
US of Albemarle	77.1	26.8
Albemarle to Gage	79.9	18.7

For areas that drain only approximately 3 square miles, the curve number difference of 2.8 raw points is significant. But even more significant, is the 8 minute difference in lag time. We believe that this lag time difference causes the double peak in the hydrograph seen at the gage. The first of the major peaks is from the sub-basins representing Albemarle Road to the gage, while the second major peak is the flood wave from upstream of Albemarle Road.

An observation of simulated hydrographs at key locations seems to reinforce this assumption. The simulated hydrograph at sub-basin 884, immediately downstream of Albemarle Road shows one distinct

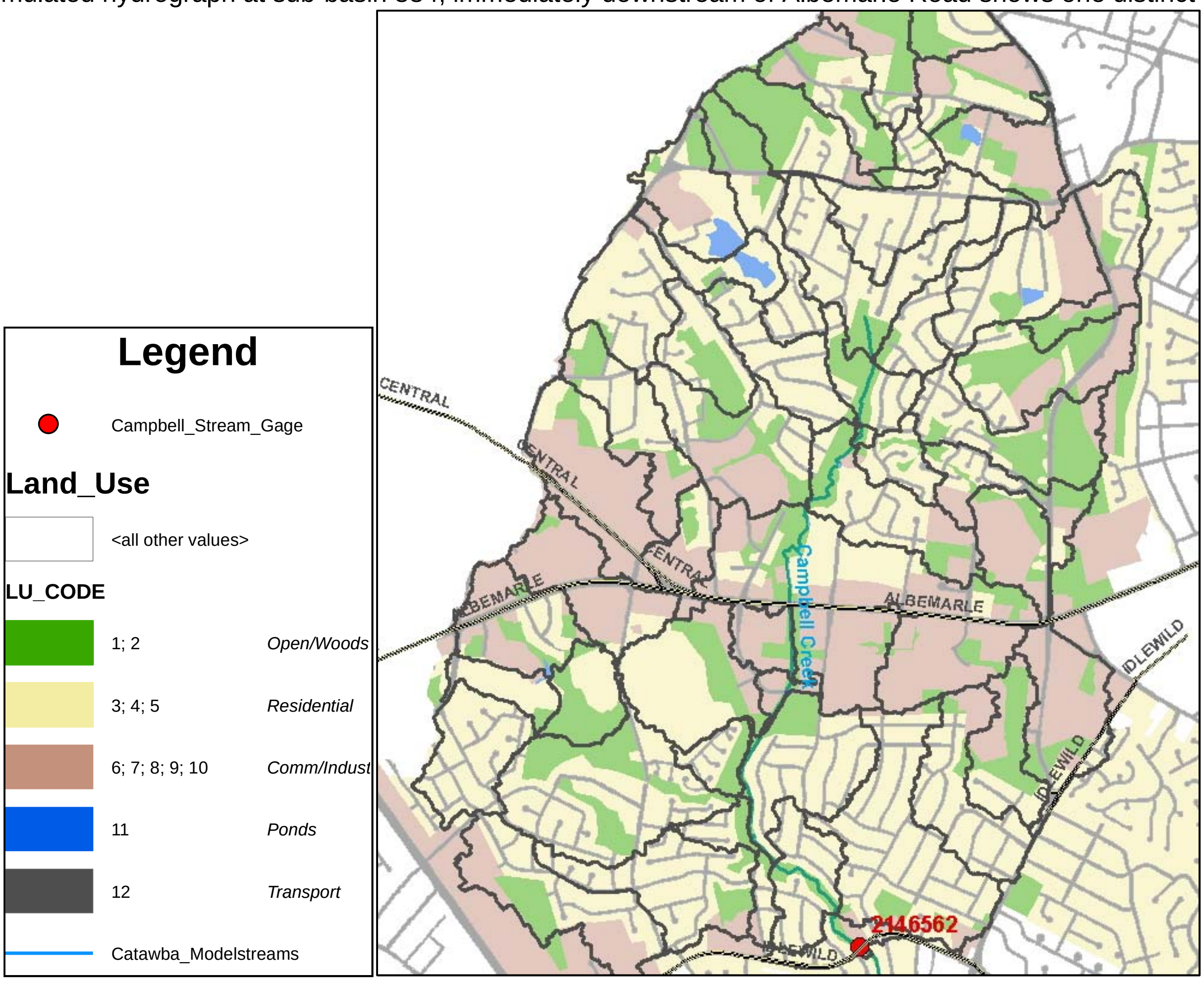


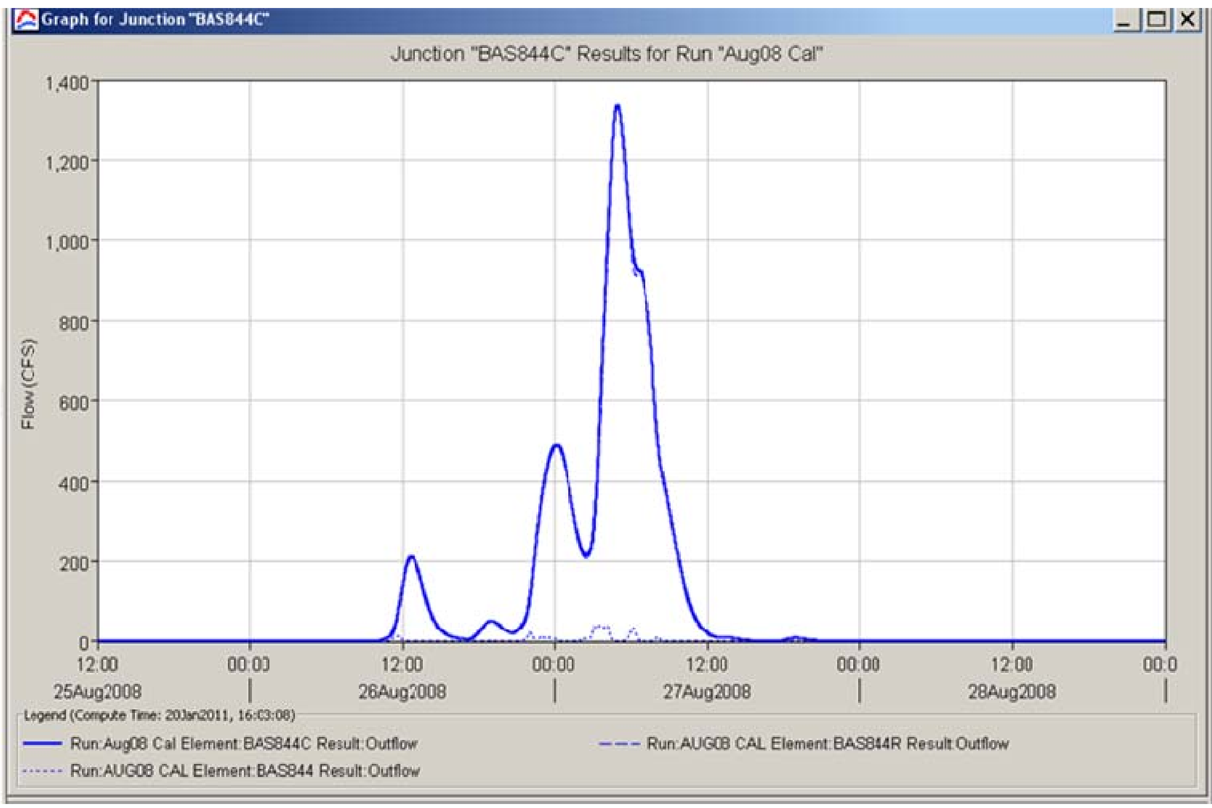
Figure 4: Campbell Creek Gage Analysis

peak flood wave with a time of peak at 4:56. The simulated hydrograph at the gage sub-basin, about 1.5 miles downstream of Albemarle Road, has a dual peak with peaks at 4:37 and 6:18. See Figure 5. The simulated hydrograph closely resembles the observed hydrograph at the gage. Obviously, the 4:37 peak cannot be the same flood wave that is represented just downstream of Albemarle Road as the times do not sync up. Therefore we contend that the first peak at the gage at 4:37 represents the flow from the sub-basins that are downstream of Albemarle Road and the second peak at 6:18 represents the main flood wave from the sub-basins upstream of Albemarle Road. The non-coincidental peaks therefore result in a lower peak flow at the gage than expected.

Initially the lag times were not adjusted from the original calculations by using any global factor but original time of concentration flow paths did progress through several iterations to find the best fit for overall time to peak of the observed gage data. The time of concentration iterations included:

- a manual redraw of flow paths through pipes as noted in the inventory file, pipe flow was calculated using the open channel flow equations
- a velocity assumption of 1 ft/sec through ponds as recommended by the county review
- a redraw of the flow path to find a true longest time flow path. The overland or sheet flow calculations had the biggest impact on overall time of concentration calculations. Extra care was taken to find the longest time flow path versus the longest distance flow path.

Sub-Basin 844C Downstream of Albemarle Road – Peak is at 4:56



Sub-Basin 1085C At Gage (about 1.5 miles downstream of Albemarle) – Peak is at 4:37 and 6:18

PRELIMINARY

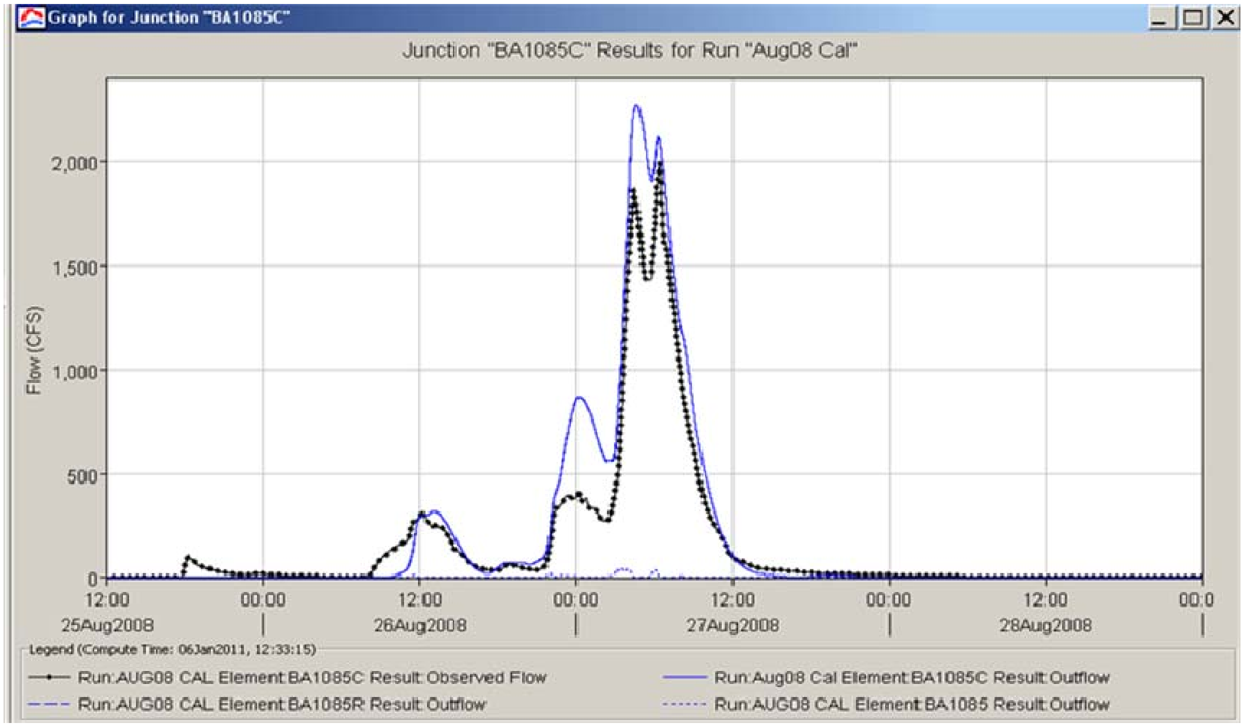


Figure 5: Simulated Hydrograph Comparison for Campbell Creek

After initial individual calibrations were performed for Campbell, McAlpine US, and Irvins Creek watersheds the main channel of McAlpine Creek was run to take a look at gage 02146600, just downstream of the confluence of McAlpine and Irvins Creek. This gage initially produced good results in time to peak and volume but the peak flow values were still very high. This observation led us to try and separate the peaks more with lag factors and to perhaps look at another possibility that would allow for a reduction in flow without an impact on volume. The initial Modified-Puls channel routing calculations in HMS set the initial sub-reach value as suggested in the guidance document. The number of sub-reaches in a routing reach affects attenuation where one sub-reach provides maximum attenuation and increasing the number of sub-reaches approaches zero attenuation. It was determined that we should entertain reducing the sub-reaches to 1 in all basins in order to assist with reducing the extremely high flows simulated at the gages on the McAlpine Creek DS model. Combined with the lag factors applied to the Irvins and McAlpine US models, we have decreased the flows significantly while maintaining stability in the models and not reducing the curve numbers to unrealistic ranges.

Specific calibration iterations for each gaged watershed can be seen in the respective spreadsheets named “***Watershed Calibration***”. For each watershed, save McMullen, we began by applying the curve number reduction as recommended in the guidance document. McMullen required only an initial abstraction calibration with no adjustment to curve numbers and no lag time factor.

4.3 Calibration Results

The model parameter calibration process for Campbell, Irvins, McMullen, McAlpine, and Six Mile Creeks resulted in slightly different initial abstraction (IA) and curve number (CN) scale factor values for the respective storms events used in calibration. Each gaged stream has its own individual calibration routine because each watershed is unique and can have a significantly different response to a similar rainfall event, as the McMullen Creek results demonstrate. In most cases we used average calibration factors for curve number and initial abstraction as calculated for both the August 2008 storm and the July 2009 storm for each model. All watersheds that drain to the McAlpine Creek DS gage at 02146750 had the number of sub-reaches reduced to 1, which increased overall attenuation, and reduced peak flows.

Table 14: Results of Catawba River Sub-basin Calibration

Date	Gaged Stream and Location	IA Starting Value	IA Calibrated Value	IA Final Value	CN Starting Value	CN Calibrated Value	CN Final Value
8/27/08	0212466000 (Campbell Ck)	0.2*S	0.35*S	0.275*S	1.00	0.85	0.85
7/29/09		0.2*S	0.2*S		1.00	0.85	
8/27/08	0212430293 (McAlpine Ck US)	0.2*S	0.4*S	0.325*S	1.00	0.90	0.875
7/29/09		0.2*S	0.25*S		1.00	0.85	
8/27/08	0212430293 (Irvins Ck)	0.2*S	0.3*S	0.2375*S	1.00	Raw - 1	0.92
7/29/09		0.2*S	0.175*S		1.00	0.85	
8/27/08	02146700 (McMullen Ck)	0.2*S	0.3*S	0.275*S	1.00	1.00	1.00
7/29/09		0.2*S	0.25*S		1.00	1.00	
8/27/08	02146750 (McAlpine Ck DS)	0.2*S	0.275*S	0.275*S	1.00	0.875	0.875
7/29/09		0.2*S	0.2*S		1.00	0.875	
8/27/08	0214685800 (Six Mile Ck)	0.2*S	0.15*S	0.15*S	1.00	0.9	0.9
7/29/09		NA	NA		NA	NA	

When the above final initial abstraction and curve number values are applied to the respective models, the final simulated runoff and peak discharge values shown below in Table 15 are the result:

Table 15: Comparison of Simulated and Observed Runoff and Peak Discharge after calibration

Date	Gaged Stream and Location	Observed Runoff (inches)	Simulated Runoff (inches)	% Difference from Observed Runoff	Observed Peak Discharge (cubic feet/second)	Simulated Peak Discharge (cubic feet/second)	% Difference from Observed Peak
8/27/08	02146562 Campbell Creek	3.51	4.40	25.4%	2,000	2,234	11.7%
7/29/09		1.06	0.80	-24.5%	475	435	-8.4%
8/27/08	0214657975 Irvins Creek	2.25	2.30	2.2%	2,090	2,123	1.6%
7/29/09		0.81	0.92	13.6%	689	961	39.5%
8/27/08	0214655255 McAlpine Creek	3.50	3.98	13.7%	3,100	3,029	-2.3%
7/29/09		1.20	1.11	-7.5%	680	827	21.6%
8/27/08	02146600 McAlpine Creek	3.38	3.28	-3.0%	6,330	8,134	28.5%
7/29/09		1.18	0.95	-19.5%	2,790	2,682	-3.9%
8/27/08	02146700 McMullen Creek	5.37	6.22	15.8%	4,020	4,010	-0.2%
7/29/09		0.85	1.16	36.5%	710	806	13.6%
8/27/08	02146750 McAlpine Creek	3.02	3.46	14.6%	7,870	11,256	43.0%
7/29/09		0.75	0.65	-13.3%	1,940	2,007	3.5%
8/27/08	0214685800	0.68	0.66	-2.9%	433	604	39.5%

7/29/09	Six Mile Creek	NA	NA	NA	NA	NA	NA

Specific hydrograph comparison for each storm in each basin is shown in the calibration spreadsheet for each watershed and is taken directly out of the HMS model results. The time to peaks for each storm and basin can be seen in Table 16.

There is no discharge gage on Four Mile Creek. The gage only reports stage or height and does not report discharges. The stage data would have been helpful in the calibration routine, but during the August 2008 storm, high watermarks were surveyed both upstream and downstream of the bridge at which the gage is located. Therefore, we used the surveyed high water mark data instead of the stage data from the gage. Four Mile Creek and Irvins Creek are similar in physical parameters and overall average curve number; therefore they should require similar calibration factors. A curve number factor of 0.92 and an IA factor of 0.2375*S was applied to the Four Mile Creek model.

The main body of the McAlpine Creek DS HMS model consisted of the sub-basins downstream of the confluence with Irvins Creek and also included the smaller un-named tributaries identified as McAlpine Creek Tributary 1, 1A, and 3. The remaining named tributaries, Rea Branch, Sardis Branch, and Swan Run had there own HMS models and were included in the McAlpine Creek DS model as inflow hydrographs. The calibration applied to the main body and the named tributaries was carried out to assist in lowering the flow and volumes seen at gages 02146600 and 02146750. A curve number factor of 0.875 and an initial abstraction of 0.275*S was applied to these sub-basins as seen in the *Calibrate.McAlpine.Downstream.xls* spreadsheet. These factors were seen as the most conservative option from the calibration of similar sub-basins like Campbell and McAlpine Creek US.

Table 16: Calibrated Time to Peaks

Date	Gaged Stream and Location	Observed Time to Peak	Simulated Time to Peak	Difference	Lag Factor Applied
8/27/08	02146562 Campbell Creek	27Aug08,06:30	27Aug08,06:23	-0:07	none
7/29/09		29Jul09,00:14	29Jul09,00:17	0:03	none
8/27/08	0214657975 Irvins Creek	27Aug08,06:00	27Aug08,05:40	-0:20	1.5
7/29/09		29Jul09,00:30	29Jul09,01:17	0:47	1.5
8/27/08	0214655255 McAlpine Creek	27Aug08,05:15	27Aug08,05:13	-0:02	1.5
7/29/09		29Jul09,00:30	29Jul09,00:41	0:11	1.5
8/27/08	02146600 McAlpine Creek	27Aug08,09:00	27Aug08,08:32	-0:28	1.5
7/29/09		29Jul09,02:15	29Jul09,03:04	0:49	1.5
8/27/08	02146700 McMullen Creek	27Aug08,06:45	27Aug08,06:16	-0:29	1.25
7/29/09		29Jul09,22:00	29Jul09,22:33	0:33	1.25
8/27/08	02146750 McAlpine Creek	27Aug08,11:45	27Aug08,11:14	-0:31	1.5
7/29/09		29Jul09,11:15	29Jul09,14:05	2:50	1.5
8/27/08	0214685800 Six Mile Creek	27Aug08,08:30	27Aug08,08:10	-0:20	none
7/29/09		NA	NA	NA	NA

Since the gages on Campbell, McAlpine US, and Irvins Creek directly impact the McAlpine Creek DS gage at station 02146600 it was determined that a lag factor should be applied to the overall models of

McAlpine US, Irvins, and McAlpine DS. Initial results at gage 02146600 from the calibrated runs of the watersheds above the gage showed that the volume was close but the peak flows were still very high. Since the volumes were so close it was determined that the curve number and initial abstractions of the watersheds above the gage were acceptable. But, we now needed to focus on the timing of the hydrographs of the tributaries in order to decrease the peak flow at the gage. A lag time factor of 1.5 was applied to the McAlpine Creek upstream and Irvins Creek models. This factor did not impact each individual model significantly but did allow for more separation of the peaks when they confluence and this did lower the flow at the gage. It was determined that any larger lag time factors would have an adverse impact on the individual models and make them unstable. Although the final flow at gage 02146600 is still 28.5% high for the August storm and 3.9% low for the July storm we feel that this calibration gives us the most balanced and stable results.

Similarly, the flows at the downstream end of McAlpine Creek, at gage 02146750 are 43.0% high for the August storm and 3.5% high in the July storm. We feel that, although outside of the 10% range suggested in the guidance document, this calibration is the most stable and balanced we could attain due to the size of the watershed and the uniqueness of the individual tributaries that drain this area.

4.4 Model Flow Comparison

The 1% annual hypothetical storm was input into the calibrated and un-calibrated models and compared to effective flows. Please see Appendix B for flow comparison details. Generally, the calibrated simulated flows were lower than effective flows at locations noted in the effective FIS. The calibrated simulated flows are generally lower than regression flows as well but closer than effective flows. We see a similar trend in the Catawba basin as we did in the Yadkin basin; proposed flows tend to be closer to effective flows toward the downstream end of the models but severely, in some cases, under-estimate flows in the headwaters. Again, this is probably due to the amount of detail in the modeling of the headwaters in this study.

Another trend that bears mentioning is the fact that wherever there is a gage that was used in the effective study, the new proposed flows are closer to effective. There are three gages that were active for the effective study, one on McMullen and two on McAlpine. Near those gages, the new proposed flows are at the most 11% off of the effective flows. The effective study was a much broader study and encompassed the entire county. The objective of the effective study was to try and apply global factors to all of the watersheds at one time in order to calibrate. Where active stream gages existed, the flows were “calibrated” to the best available data. In order to be conservative, not much calibration was performed on reaches that did not have an active stream gage. Although, it appears that Four Mile and Irvins Creek were identified as watersheds that could be calibrated and were because they did not have as much development pressure as McAlpine and McMullen.

Since we have three more stream gages in the headwaters of the McAlpine watershed for this current study, the calibration has become much more specific and widespread. The comparison to effective flows exemplifies this. This fact also contributes to some of the major differences in flows as compared to effective discharges.

Historical Event Model Comparison: Historical storm events were input into the HMS models for observation. The summer storms of 1995 and 1997 are the storms that always come to mind when flooding events in Mecklenburg County are discussed. Rainfall and stream flow data from the USGS was obtained and incorporated into the HMS models to test the models versus other historical rainfall events. There was, however, some data missing or just not correct from the USGS data dump. The 1997 stream flow data from gage 02146700 on McMullen Creek was only partially available and since it was missing a significant amount of data near the peak it was deemed unusable. Also, it was discovered that rain gage CRN-23 at Charles T. Myers Golf Club had reported low rainfall totals for the 1995 event and this was not consistent with the USGS Rainfall Distribution map for that storm. Therefore the rain totals from that gage were discarded from the calculations. Table 16 and 17 display the raw and calibrated results of the historical storm event analysis.

Table 16: 1995 Comparison of Runoff, Peak Discharge, and Time to Peak

Date	Gaged Stream and Location	Observed Runoff (inches)	Simulated Runoff (inches)	% Difference from Observed Runoff	Observed Peak Discharge (cfs)	Simulated Peak Discharge (cfs)	% Difference from Observed Peak	Observed Time to Peak	Simulated Time to Peak
Raw 95	02146700 McMullen Creek	4.86	6.06	24.7%	3,470	3,493	0.7%	27Aug95,08:00	27Aug95,07:36
Cal 95		4.86	5.86	20.6%	3,470	3,360	-3.2%	27Aug95,08:00	27Aug95,07:44
Raw 95	02146600 McAlpine Creek	4.71	4.93	4.7%	8,980	12,408	38.2%	27Aug95,10:00	27Aug95,09:38
Cal 95		4.71	3.67	-22.1%	8,980	8,449	-5.9%	27Aug95,10:00	27Aug95,09:38
Raw 95	02146750 McAlpine Creek	4.87	5.38	10.5%	12,500	19,504	56.0%	27Aug95,12:28	27Aug95,18:15
Cal 95		4.87	4.35	-10.7%	12,500	13,398	7.2%	27Aug95,12:28	27Aug95,17:41

Calibrated volumes were 10.7%, 13.5%, and 22.1% lower in the McAlpine Creek models except at gage 02146600 in the 1997 storm where the calibrated volume is 16.7% high. In the McMullen model the calibrated volume is 20.6% high. Calibrated peak flows range from 5.9% low in the McAlpine Creek model at gage 02146600 in the 1995 storm to 17.5% high at the same gage for the 1997 storm. The shape of the observed hydrographs and the calibrated simulated hydrographs are in general agreement for all of the storms and can be seen in the ***Calibration*** spreadsheets for each gage.

Table 17: 1997 Comparison of Runoff, Peak Discharge, and Time to Peak

Date	Gaged Stream and Location	Observed Runoff (inches)	Simulated Runoff (inches)	% Difference from Observed Runoff	Observed Peak Discharge (cfs)	Simulated Peak Discharge (cfs)	% Difference from Observed Peak	Observed Time to Peak	Simulated Time to Peak
Raw 97	02146600 McAlpine Creek	3.71	5.68	53.1%	6,170	10,791	74.9%	23Jul97,13:45	23Jul97,11:08
Cal 97		3.71	4.33	16.7%	6,170	7,252	17.5%	23Jul97,13:45	23Jul97,13:11
Raw 97	02146750 McAlpine Creek	4.58	5.00	9.2%	9,310	11,749	26.2%	24Jul97,05:30	23Jul97,08:58
Cal 97		4.58	3.96	-13.5%	9,310	9,703	4.2%	23Jul97,05:30	23Jul97,08:57

Gage Analysis Comparison: As mentioned earlier and seen in Table 11, there are three gages in this study area that have a long enough gage record to perform a statistical recurrence interval analysis. Table 18 displays how simulated calibrated flows compare to the gage analyses. We are slightly lower than the gage analysis in the McMullen Creek watershed but our chief hydrologist stated that the McMullen Creek data is trending upward so much that he does not put a great amount of confidence in that projected flow. But the two projected flows on McAlpine, he is confident in, and we are just over-estimating those flows. This analysis, along with the historical storm analysis of each of these watersheds provides confidence in the calibration procedure and the final flows in Appendix B.

Table 18: Comparison of AECOM Gage Analysis Results

Gage ID	Gaged Stream and Location	Dates Analyzed	DA (miles^2)	Q100 (cfs)	Calibrated Q (cfs)	% Diff
02146700	McMullen Creek at Sharon View near Charlotte, NC	1975-2009	7.0	5,140	4,443	-13.6%

02146600	McAlpine Creek at Sardis Rd near Charlotte, NC	1975-2009	39.6	8,910	10,001	12.4%
02146750	McAlpine Creek below McMullen Cr near Pineville, NC	1975-2009	92.4	11,600	12,931	11.5%

High Water Mark Comparison: As an additional level of quality assurance, the final simulated calibrated flows from the August 2008 storm were input into updated HEC-RAS models. These models are not final calibrated models but do contain the most up to date stream geometry and structure information. McMullen and McAlpine Creeks had the majority of the marks with over 20 each, while Swan Run, Campbell, and Four Mile were also represented with at least one high water mark.

This ***initial*** high water mark comparison reveals that the calibrated hydrology appears reasonable. On McMullen Creek, there are a few areas where the proposed water surface elevation is up to 3.54 feet lower than the measured high water mark. The most significant of these, 3.54 and 2.86 are measured near structures and may be caused by a specific computational difference or a more likely scenario could be that the structure was partially blocked during the actual storm event, thus causing a higher elevation of the high water mark versus the modeled elevation. The 1.2 foot difference at the downstream end of the model may be caused by backwater from McAlpine Creek. On McAlpine Creek we see consistently higher water surface elevations in our simulated model from station 90,000 to the downstream extent of the model. We believe that we can decrease our simulated water surface elevations by slightly decreasing n-values and by applying justified ineffective areas. There are two high water marks near the upstream end of the model that we measure lower than. They are both near structures and we believe this to be the reason for the lower simulated flows. Additional calibration will be required on the hydraulic models.

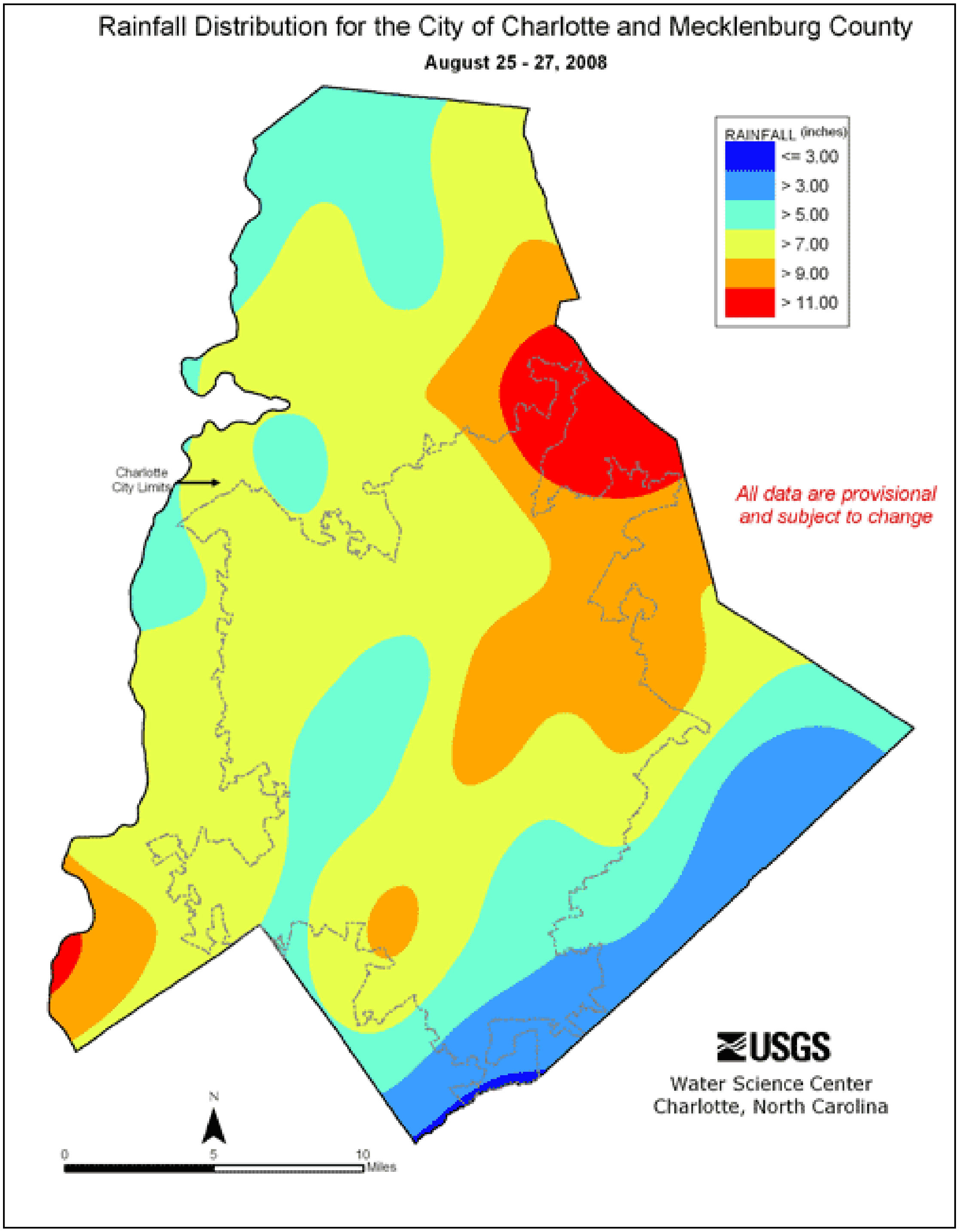
Overall, we have 51 high water marks for the August 2008 storm event. Our raw HEC-RAS models, using the flows provided by the simulated, calibrated HEC-HMS models calculate 9 water surface elevations that are below the corresponding measured high water mark. We feel in most cases, the impact of a hydraulic structure is the reason for the difference, and feel that this can be addressed during the calibration of the HEC-RAS models.

4.5 Calibration in Watersheds without Historical Stream Flow Data

A review of the un-calibrated peak flows as compared to the observed gage peak flows for each storm in all of the watersheds reveals that original simulated estimates are high in peak flow and volume for every watershed except for McMullen Creek, which is a special case as referenced continuously in the report above. When the McMullen model was not accounted for, our un-calibrated peak flows ranged from 20.8% – 140.7% higher than observed flows and the volumes were 33.0% - 80.9% higher than observed volumes. Therefore, some significant calibration was required to try and mimic the observed flow at all of the gages, sans McMullen. With that in mind, we wanted to apply some calibration to the un-gaged sub-watersheds. First, we wanted to make sure we did not have any other un-gaged watersheds that were similar to McMullen Creek. Only Four Mile, Clems Branch, and the smaller tributaries to McAlpine Creek did not have a stream gages on them and would require some sort of estimated calibration. All of the flow comparisons can be seen in Appendix B. After a review of the curve numbers, land use, basin shape, and built out conditions, it was assumed that none of these sub-watersheds were similar to McMullen Creek and they would require some sort of calibration. To that end, Four Mile Creek was scrutinized most intently because it was the largest and had the most impact on flows into McAlpine Creek. It was decided, after some consideration of curve numbers, land use, basin shape, and general watershed location that Four Mile Creek appeared to be similar to Irvins Creek in the referenced parameters. Therefore, the Irvins Creek calibration factors of 0.92*Raw CN, Initial Abstraction of 0.2375*S, and lag factor of 1.5 were applied to the Four Mile Creek sub-basins. The smaller tributaries to McAlpine Creek on the other hand appeared to not have a great impact on the overall flows at the gages on the main channel, but we did want to apply some calibration to these tributaries. It was determined then, that the most conservative factor of a similar gaged basin would be referenced. Therefore, Rea, Sardis, and

Swan Run adopted the 0.875*raw CN that was calculated from the McAlpine US model and the initial abstraction of 0.275*S was taken from the Campbell Creek model because it was the most conservative. Clems Branch received a 0.92*raw CN factor, an initial abstraction factor of 0.2375*S and a lag factor of 1.5.

Appendix A


Rainfall Distribution for the City of Charlotte and Mecklenburg County
August 25 - 27, 2008
RAINFALL (inches)
<= 3.00
> 3.00
> 5.00
> 7.00
> 9.00
> 11.00
Charlotte City Limits
All data are provisional and subject to change
N
0
5
10
Miles
USGS
Water Science Center
Charlotte, North Carolina

Appendix B

	Eff DA (mi2)	Calc DA (mi2)	Calc Basin ID	Eff 1% (cfs)	Raw Sim 1% (cfs)	Reg 1% (cfs)	Ass % Imp	Cal 1% (cfs)	% Change Eff	% Change Reg	Initial Avg CN	Cal Avg CN	1% Gage Analysis Estimate
Campbell Creek											78.54	66.76	
at confluence w/ McAlpine	7.45	7.45	BAS1080	2571	4054	3884	25	2385	-7%	-38%			
2100ft D/S of Exec Center Dr	4.5	4.54	BAS852	2424	2954	2644	25	1888	-22%	-28%			
50ft D/S of Exec Center dr	3.11	3.15	BAS847	1900	2439	2322	25	1429	-25%	-38%			
400ft D/S of Barcliff Park	1.21	1.22	BAS792	1505	1362	1299	25	924	-39%	-30%			
Four Mile Creek											77.25	71.07	
at confluence w/ McAlpine	18.95	19.11	BAS932	4750	7107	6810	25	5026	6%	-26%			
2200ft U/S of Providence Rd	10.1	10.1	BAS904	4807	6613	4664	25	4507	-6%	-3%			
5500ft U/S of Providence Rd	8.24	7.59	BAS888	4510	5809	3939	25	3751	-17%	-5%			
6100ft D/S of Trade St	6.12	6.04	BAS871	4301	5332	3439	25	3464	-19%	0%			
2800ft D/S of Trade St	5.08	5.05	BAS862	3696	4483	3091	25	3076	-17%	0%			
2200ft U/S of Trade St	3.18	3.14	BAS846	2755	2904	2154	20	1801	-35%	-16%			
5600ft U/S of Trade St	1.13	1.12	BAS787	1048	952	1168	20	575	-45%	-51%			
Rocky Branch											77.89	71.66	
at confluence w/ Four Mile	2.11	2.12	BAS831	1858	1850	1708	25	1337	-28%	-22%			
1500ft U/S of Four Mile Rd	1.44	1.46	BAS806	1237	1667	1369	25	1164	-6%	-15%			
900ft U/S of Providence Rd	0.96	0.92	BAS764	911	1018	1043	25	686	-25%	-34%			
Irvins Creek											73.57	67.68	
at confluence w/ McAlpine	14.82	14.76	BAS918	3780	7641	5394	20	5671	50%	5%			
200ft D/S of Independence	14.27	14.44	BAS916	3752	7600	5326	20	5675	50%	6%			
3000ft U/S of Independence	9.57	9.98	BAS903	3053	5370	3857	15	4054	33%	5%			
100ft U/S of Lebanon Rd	5.33	5.53	BAS867	2770	4062	2720	15	2976	7%	9%			
U/S of Beaverdam Ln	3.97	4.13	BAS857	2253	3156	2286	15	2359	5%	3%			
U/S of Beaverdam Ln	2.00	1.98	BAS825	1006	2070	1479	15	1425	41%	-4%			
700ft U/S of Apple Creek Dr	1.26	1.19	BAS1098	852	1046	1091	15	738	-14%	-33%			
400ft U/S of Lawyers Rd	0.79	0.87	BAS762	824	790	906	15	546	-34%	-40%			
Irvins Creek Trib 1											79.17	72.84	
at confluence	4.31	4.28	BAS858	1940	2588	2995	30	1852	-5%	-38%			
2300ft D/S of Sam Newell	3.55	3.76	BAS1014	1749	2638	2772	30	1786	-2%	-36%			
1500ft D/S of Independence	2.28	2.39	BAS838	1717	2494	2118	30	1707	-1%	-19%			
800ft U/S of Windsor Park	1.31	1.56	BAS810	1500	1826	1644	30	1245	-17%	-24%			
Irvins Creek Trib 2											70.08	64.48	
at confluence	1.98	1.98	BAS1077	1344	1343	1281	10	970	-28%	-24%			
400ft U/S of Lawyers Rd	1.58	1.73	BAS1076	1559	1317	1182	10	915	-41%	-23%			
2300ft U/S of Lawyers Rd	0.98	1.00	BAS781	971	947	856	10	589	-39%	-31%			
McAlpine Creek											79.91	70.32	
5600ft D/S of Lancaster Hwy	93.2	93.47	BAS1020	11641	19986	17454	30	12907	11%	-26%			11600

	Eff DA (mi2)	Calc DA (mi2)	Calc Basin ID	Eff 1% (cfs)	Raw Sim 1% (cfs)	Reg 1% (cfs)	Ass % Imp	Cal 1% (cfs)	% Change Eff	% Change Reg	Initial Avg CN	Cal Avg CN	1% Gage Analysis Estimate
McAlpine Creek											79.91	70.32	
U/S of RR Bridge nr Monroe	32.27	32.35	BAS938	9039	16128	9302	25	10430	15%	12%			
U/S of Independence	16.29	16.37	BAS923	5683	9087	6204	25	5172	-9%	-16%			
U/S of Idlewild - Gage	7.33	7.32	BAS2001	3294	5075	3560	20	2576	-22%	-27%	75.9	66.41	
400ft U/S of Lawyers Rd	4.94	5.11	BAS864	3099	4322	2875	20	1987	-36%	-31%			
500ft D/S of Marlwood Circle	2.81	2.88	BAS841	1885	2311	2047	20	1070	-43%	-48%			
700ft U/S of Marlwood Circle	1.74	1.79	BAS820	1380	1583	1544	20	877	-36%	-42%			
McAlpine Creek Trib 1											81.37	71.6	
at confluence	3.26	3.33	BAS848	2211	3531	2580	30	2679	21%	4%			
3800ft U/S of US521	1.46	1.46	BAS807	1502	1845	1584	30	1198	-20%	-24%			
McAlpine Creek Trib 1A											81.61	71.82	
at confluence w/ Trib 1	1.22	1.26	BAS796	944	1430	1450	30	1049	11%	-28%			
300ft U/S of Ballantyne Cmns	0.96	0.98	BAS1034	977	1061	1249	30	837	-14%	-33%			
McAlpine Creek Trib 3											81.56	71.78	
at confluence w/ McAlpine	1.77	1.79	BAS819	1519	2109	1783	30	1466	-4%	-17%			
750 D/S of Rea Rd	1.34	1.29	BAS797	1327	1701	1466	30	1072	-19%	-26%			
McAlpine Creek Trib 6											75.8	66.32	
at confluence w/ McAlpine	2.06	2.06	BAS829	1258	2537	1978	20	1276	1%	-34%			
2300ft U/S of confuence	1.64	1.65	BAS814	1251	2180	1473	20	1075	-14%	-27%			
5000ft U/S of confuence	1.36	1.51	BAS1081	1299	2209	1395	20	1065	-16%	-24%			
McMullen Creek											81.28	81.28	
at confluence w/ McAlpine	15.19	15.27	BAS1021	5902	5498	6362	30	5340	-10%	-16%			
4200ft U/S of Johnston Rd	12.09	12.28	BAS910	5264	5352	5591	30	5074	-4%	-9%			
5200ft D/S of Quail Hollow Rd	9.91	9.86	BAS1025	4566	4901	4910	30	4685	3%	-5%			
U/S of Mountainbrook Rd	7.5	7.52	BAS887	4561	4680	4180	30	4519	-1%	8%			5140
1000ft D/S of Arborway Rd	5.39	5.38	BAS865	4476	4352	3429	30	4235	-5%	23%			
1300ft U/S of Arborway Rd	4.74	4.75	BAS861	4358	4081	3184	30	3964	-9%	24%			
300ft D/S of Lincrest Pl	2.02	2	BAS828	2506	2122	1906	30	2051	-18%	6%			
2200ft U/S of Lincrest Pl	1.56	1.49	BAS1032	2179	2058	1602	30	1981	-9%	23%			
McMullen Creek Trib											81.28	81.28	
at confluence w/ McMullen	1.36	1.42	BAS803	1923	1928	1556	30	1870	-3%	19%			
1200ft D/S of S Sharon Amity	1.11	1.17	BAS790	1747	1693	1386	30	1646	-6%	17%			
U/S of S Sharon Amity	0.84	0.84	BAS775	1473	1454	1142	30	1412	-4%	22%			

	Eff DA (mi2)	Calc DA (mi2)	Calc Basin ID	Eff 1% (cfs)	Raw Sim 1% (cfs)	Reg 1% (cfs)	Ass % Imp	Cal 1% (cfs)	% Change Eff	% Change Reg	Initial Avg CN	Cal Avg CN	1% Gage Analysis Estimate
Rea Branch											82.56	72.65	
at confluence w/ McAlpine	1.77	1.81	BAS821	2512	2174	1797	30	1168	-54%	-35%			
U/S of Parkview Dr	1.64	1.36	BAS800	2458	1746	1520	30	959	-61%	-36%			
U/S of Sequoia Red Ln	1.14	1.15	BAS1038	2167	1768	1376	30	990	-53%	-27%			
Sardis Branch											75.22	66.2	
at confluence w/ McAlpine	2.37	2.39	BAS845	2121	2600	1833	20	1593	-25%	-13%			
200ft U/S of N Sardis Rd	2.2	1.85	BAS824	2272	1931	1573	20	1187	-48%	-24%			
1100ft D/S of Sardis Rd	1.51	1.71	BAS1100	1840	1907	1505	20	1136	-38%	-23%			
Swan Run Branch											77.05	67.8	
at confluence w/ McAlpine	2.11	2.12	BAS1042	2067	2696	1846	25	1559	-24%	-15			
5300ft U/S of Sharon View Rd	1.18	1.28	BAS1070	1687	2211	1369	25	1121	-34%	-19%			
Six Mile Creek											75.03	67.53	
@ county line	22.63	22.44	BAS937	6596	6374	6236	15	5164	-22%	-17%			
4400ft D/S of Tom Short	9.33	9.44	BAS899	3629	3348	3730	15	2630	-26%	-30%			
4100ft U/S of Tom Short	6.52	6.52	BAS877	3242	2860	2996	15	2254	-30%	-23%			
200ft U/S of Providence Rd	4.48	4.29	BAS859	2564	2110	2340	15	1772	-31%	-24%			
3100ft U/S of Providence Rd	3.47	3.45	BAS849	2133	1884	2054	15	1582	-26%	-23%			
3000ft D/S of Tilley Morris	2.38	2.35	BAS836	1783	1324	1636	15	1135	-36%	-30%			
Flat Branch											80.72	72.65	
at confluence w/ Six Mile	4.15	3.94	BAS853	2863	2374	2671	25	1964	-31%	-26%			
1400ft U/S of Threat Vail Ln	2.5	2.75	BAS839	2358	2231	2152	25	1846	-21%	-14%			
2500ft D/S of Tom Short	2.06	2.2	BAS833	1995	2020	1890	25	1675	-15%	-12%			
2000ft D/S of Tom Short	1.71	1.84	BAS808	1756	1851	1702	25	1536	-13%	-10%			
1400ft D/S of Tom Short	1.23	1.24	BAS795	1283	1438	1345	25	1154	-8%	-13%			
3500ft U/S of Tom Short	0.89	0.96	BAS773	1138	1220	1158	25	981	-14%	-15%			
Clems Branch											76.28	68.66	
3100ft DS of Lancaster Hwy	2.34	2.34	BAS834	2030	2679	1809	20	1947	-4%	8%			
2700ft DS of Lancaster Hwy	1.56	1.56	BAS809	1388	1815	1421	20	1173	-15%	-17%			
40ft DS of Lancaster Hwy	0.8	0.8	BAS751	772	1192	958	20	571	-26%	-40%			