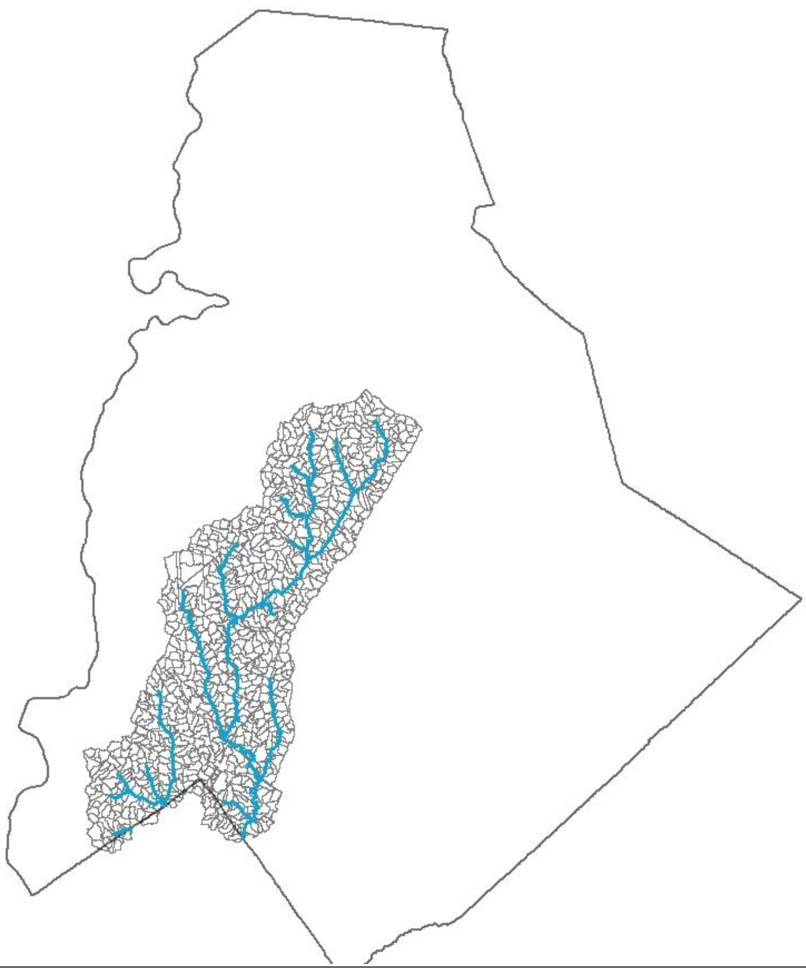
PRELIMINARY

MECKLENBURG COUNTY FLOODPLAIN MAPPING 2008

Sugar/Irwin Sub-Watershed Hydraulics Report



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AECOM
6201 Fairview Road, Suite 400, Charlotte, North Carolina 28210
T 704.553.6150 F 704.553.6151 www.aecom.com

Introduction

The purpose of this study is to quantify the magnitude and extent of flooding during storms of selected recurrence intervals within the Sugar/Irwin watershed of Mecklenburg County, North Carolina. Accomplishing this task required the development / capture of detailed hydrologic and hydraulic data. This report will outline the various parameters and procedures used to perform the detailed *hydraulic* modeling in the Sugar/Irwin watershed, with the detailed hydrologic modeling being described and outlined separately in the "*Mecklenburg County Floodplain Mapping 2008: Sugar/Irwin Sub-Basin Hydrology Report"*.

Scope of Study

The intent of the Charlotte-Mecklenburg Storm Water Services Floodplain Mapping Project is to provide accurate and up-to-date floodplain maps for the entirety of Mecklenburg County. This involves the restudying and remapping of all streams in the county that have been studied in previous FEMA flood studies. The initiative, which began most recently in 2007, is being carried out through a strategy that sub-divides the county into major watersheds, with each watershed being studied individually (though consistency between the various studies is ensured through adherence to the county's "Floodplain Analysis and Mapping Standards Guidance Document"). Since then, the Charlotte-Mecklenburg Storm Water Services (CMSWS) has conducted restudy efforts in a number of watersheds in conjunction with various study contractors, with AECOM being one of them.

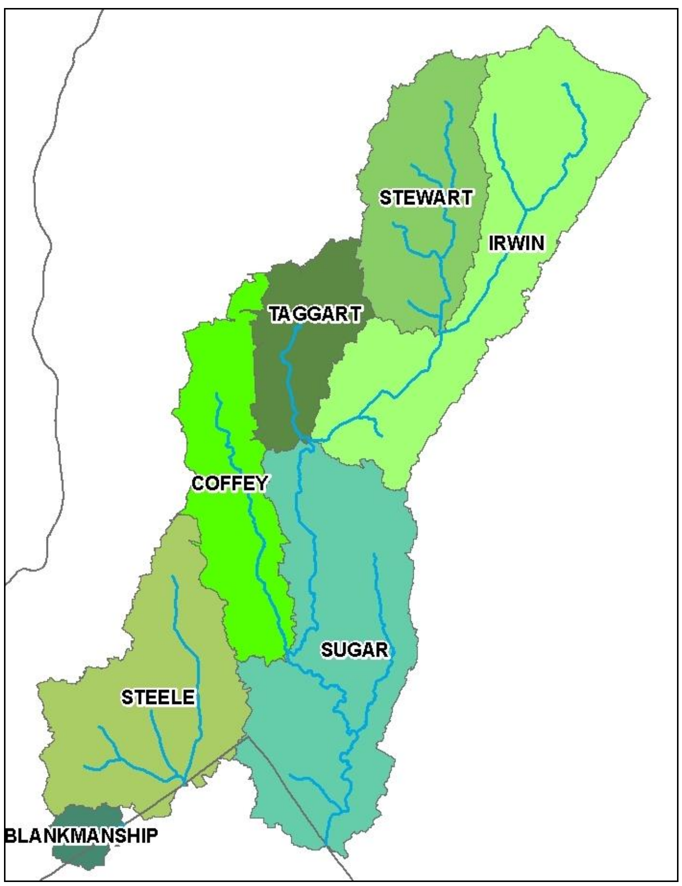

Figure 1 – Sugar/Irwin Sub-Watersheds

The Sugar/Irwin watershed consists of approximately 60.1 miles of detailed riverine mapping. A list of the study limits for streams studied by detailed methods can be found in table 1 below:

Table 1. Stream Reaches Studied by Detailed Methods in the Sugar/Irwin Watershed

Stream Name	Downstream Limit	Upstream Limit	Length (mi.)
Blankmanship Branch	Mecklenburg County Line	Approx. 100 feet upstream of Steele Creek Road	0.7
Coffey Creek	Confluence with Sugar Creek	Approx. 0.7 miles upstream of West Boulevard	6.3
Irwin Creek	Confluence with Sugar Creek	Approx. 0.9 miles upstream of Nevin Road	10.7
Irwin Creek Tributary 1	Confluence with Irwin Creek	Approx. 840 feet upstream of an Access Road	0.8
Kennedy Branch	Confluence with Irwin Creek	Approx. 220 feet upstream of Slater Road	2.1
Kings Branch	Confluence with Sugar Creek	Approx. 370 feet upstream of I-77	4.4
McCullough Branch	Confluence with Sugar Creek	Approx. 415 feet upstream of Nations Ford Road	1.4
Polk Ditch	Confluence with Walker Branch	Approx. 295 feet upstream of South Tryon Street	1.4
Steele Creek	York County, South Carolina State Line	Approx. 170 feet upstream of Brown-Grier Road	4.5
Stewart Creek	Confluence with Irwin Creek	Approx. 765 feet upstream of Capps Hill Mine Road	5.3
Stewart Creek Tributary 1	Confluence with Stewart Creek	Approx. 485 feet upstream of Access Road 02	1.2
Stewart Creek Tributary 2	Confluence with Stewart Creek	Approx. 275 feet upstream of I - 85	1.6
Stewart Creek Tributary 3	Confluence with Stewart Creek	Approx. 2,065 feet upstream of Hoskins Road	1.1
Sugar Creek	York County, South Carolina State Line	Approx. 1,220 feet upstream of Billy Graham Parkway	12.1
Taggart Creek	Confluence with Sugar Creek	Approx. 445 feet upstream of Denver Avenue	3.5
Walker Branch	Confluence with Steele Creek	Approx. 1625 feet upstream of South Tryon Street	2.2
Walker Branch Tributary	Confluence with Walker Branch	Approx. 370 feet upstream of Steele Creek Road	0.8

Hydraulic Approach

Water-surface elevations of floods of the selected annual chance of exceedance discharges were computed through use of the Army Corps of Engineers' HEC-RAS step-backwater computer program version 4.1. These computer models were calibrated using stream gage data and historic high water data collected during field investigations.

A countywide LiDAR dataset flown in 2007 was used for terrain data. Hydraulic cross section geometries were obtained from a combination of terrain data and field survey. All bridges, dams, and culverts were field surveyed to obtain elevation data and structural geometry. The infrastructure inventory was mostly obtained from archives of the effective study. The effective survey file was reviewed and QC'd by the County in the field and each structure 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. There were six new structures added to the effective file. Cross sections were field surveyed at approximately 1500ft increments along the streams to determine channel geometries between bridges and culverts. Most of the overbank cross-section data for the backwater analyses were obtained from the LiDAR dataset.

Initial Manning's n-value assumptions were made based on values published in "Open-Channel Hydraulics" [Chow, 1959]. N-value change locations along each cross-section were set to coincide with the approved landuse polygons developed for the calculation of curve numbers in the hydrologic analysis. Refinements were made to these initial assumptions through a combination of field investigation and examination of Mecklenburg County 2009 color orthophotos for both channel and overbank areas, with additional adjustments made to account for the presence of buildings (as outlined in the county's *Floodplain Analysis and Mapping Standards Guidance Document*). A tabulation of the landuse descriptions and their associated range of assumed n-values can be found in Table 2 below:

Table 2. Assumed N-value Variation with Respect to Landuse

Landuse Code	Landuse Description	Minimum n-value	Maximum n-value
1	WOODS/BRUSH	0.125	0.160
2	OPEN SPACE, GREATER THAN 2 ACRES RESIDENTIAL	0.055	0.095
3	GREATER THAN 0.5 TO 2 ACRES RESIDENTIAL	0.075	0.115
4	0.25 TO 0.5 ACRE RESIDENTIAL	0.075	0.125
5	LESS THAN 0.25 ACRE RESIDENTIAL/APTS./MULTIFAM	0.095	0.135
6	INSTITUTIONAL; SCHOOLS, HOSPITALS, ETC.	0.070	0.070
7	INDUSTRIAL - LIGHT (WAREHOUSES, ETC.)	0.075	0.075
8	INDUSTRIAL - HEAVY	0.080	0.080
9	COMMERCIAL - LIGHT (OFFICE PARKS, HOTELS)	0.080	0.100
10	COMMERCIAL - HEAVY (CAR PARKS, MALLS)	0.055	0.075
11	WATER BODIES/PONDS	0.040	0.040
12	TRANSPORTATION, MULTILANE ROADS, INTERSTATES	0.060	0.060

Channel n-values varied from 0.036 to 0.051. Overbank reach lengths were calculated along the approximate centerline of the anticipated flowpath of the overbank flow during the 1-percent-annual-chance event. Overbank flow centerline locations were estimated from the topography, and refined once initial 1-percent-annual-chance runs were made.

Starting conditions for the hydraulic models were set to normal depth using starting slopes calculated from channel invert values taken from the terrain and survey data. State line tie-ins were considered for Sugar and Steele Creeks as well as Blankmanship Branch. The Sugar and Steele Creek models were calibrated slightly to be within 0.5' at the border just using the normal depth boundary condition. Blankmanship Branch on the other hand, required more attention. It was noted that the York County, SC elevation, was about 3 feet higher than our preliminary simulated elevation. Upon further investigation it was noted that the York County model has a structure about 300 feet downstream of the state line. This structure causes backwater of about 6 feet and this backwater extends across the state line into Mecklenburg County. Therefore, the boundary condition for the Blankmanship Branch HEC-RAS model was set to a known water surface elevation for the 10%, 1%, and 0.2% annual chance storm events. The 2% annual chance storm event was not provided in the York County FIS. The normal depth slopes for the remaining events were then adjusted to match the slopes of the known water surface elevations. That is the reason for the odd conglomeration of starting conditions for the Blankmanship Branch model.

Hydraulic Modeling Results

In comparison with the effective base flood elevations, the newly calculated 1-percent-annual-chance water surface elevations have increased in some locations but decreased in others along the studied streams. This is to be expected, given that – in conjunction with other factors – the discharges yielded by the accompanying updated hydrologic analysis have done the same.

The County noted that they were specifically concerned with water surface elevations in the Stewart Creek watershed. There was some significant flooding recently that occurred outside the effective flood zone. The current initial analysis indicates that water surface elevations along Stewart Creek have increased from 0.3 – 4.4 feet when compared to effective elevations at the same location.

A comparison between the effective base flood elevations and the newly calculated 1-percent-annual-chance water surface elevations at select locations can be found in table 3 below:

Table 3. Effective vs Updated 1-Percent-Annual-Chance Water Surface Elevations

	Eff Q (cfs)	Sim Q (cfs)	Eff 1% WSEL	Sim 1% WSEL	Diff (ft)	Cal Sim 1% WSEL	Diff (ft)
Blankmanship Branch							
At County Line	930	954	614.4	611.0	-3.4	616.0	1.6
Approx 3,100 ft. U/S of County Line	870	712	635.7	627.1	-8.6	635.5	-0.2
Coffey Creek							
Confluence with Sugar Creek	3,452	4,155	565.6	565.8	0.2	566.9	1.3
Approx 6,700 feet U/S of Shopton Road	3,359	3,785	611.2	610.9	-0.3	610.9	-0.3
Approx 600 feet D/S of Piney Top Drive	3,024	3,524	649.5	648.7	-0.8	646.5	-3.0
Approx 700 feet U/S of West Boulevard	2,931	2,740	654.1	652.6	-1.5	652.7	-1.4
Approx 1,900 feet U/S of West Blvd	2,774	2,458	656.7	656.0	-0.7	656.2	-0.5
Irwin Creek							
Confluence with Taggart Creek	12,300	12,341	606.7	603.1	-3.6	608.6	1.9
Approx 200 feet U/S of Remount Road	9,000	11,670	630.8	629.9	-0.9	628.5	-2.3
Approx 400 feet U/S of I-277	6,400	6,974	641.4	643.2	1.8	641.8	0.4
Approx 2,400 feet D/S of I-85 Svce Rd	3,230	4,080	669.1	672.0	2.9	673.3	4.2
Approx 1,200 feet D/S of Starita Road	2,870	3,246	696.7	696.1	-0.6	695.5	-1.2
Approx 700 feet U/S of Dalecrest Drive	2,220	2,570	706.7	706.8	0.1	706.1	-0.6
Approx 1,200 feet U/S of Nevins Road	1,580	1,534	725.7	725.5	-0.3	724.4	-1.3
Approx 4,700 feet U/S of Nevins Road	1,260	1,378	739.3	740.3	1.0	739.8	0.5
Irwin Creek Tributary 1							
At confluence with Irwin Creek	2,570	2,456	615.1	605.9	-9.2	614.3	-0.8
Kennedy Branch							
At confluence with Irwin Creek	3,001	3,004	669.0	663.4	-5.6	664.2	-4.8
Approx 2,600 ft. U/S of mouth	1,774	1,633	674.0	674.0	0.0	673.7	-0.3
Approx 200 ft. D/S of Cindy Lane	1,349	1,568	728.1	725.3	-2.8	725.4	-2.7
Approx 300 ft. D/S of Slater Road	948	1,554	730.3	729.0	-1.3	729.0	-1.3

	Eff Q (cfs)	Sim Q (cfs)	Eff 1% WSEL	Sim 1% WSEL	Diff (ft)	Cal Sim 1% WSEL	Diff (ft)
Kings Branch							
At Confluence with Sugar Creek	1,488	2,941	537.9	535.0	-2.9	534.6	-3.3
Approx 1,000 ft. D/S of Kings Branch Ct	1,240	3,166	606.0	606.1	0.1	606.3	0.3
Approx 200 ft. U/S of Archdale Drive	1,060	2,241	617.6	621.2	3.6	621.2	3.6
Approx 200 ft. D/S of I-77	610	1,148	628.9	631.3	2.4	631.3	2.4
McCullough Branch							
At Confluence with Sugar Creek	1,253	1,379	529.0	530.9	1.9	530.9	1.9
Approx 500 ft. D/S of Nations Ford Rd	1,248	1,220	558.6	557.2	-1.5	557.0	-1.6
Polk Ditch							
At Confluence with Walker Branch	951	1,383	564.5	565.0	0.5	565.0	0.5
Approx 4,900 ft. U/S of Confluence	732	748	581.4	582.1	0.7	582.1	0.7
Steele Creek							
At County Line	5,384	7,970	569.1	567.4	-1.7	570.0	0.9
Approx 210 ft. U/S of County Line	2,797	3,690	569.1	568.1	-1.0	570.3	1.2
Approx 800 ft. U/S of John Price Road	1,914	2,376	589.4	590.1	0.7	591.5	2.1
Approx 100 ft. U/S of Arrowwood Appt Rd	1,589	2,032	590.9	591.5	0.6	592.1	1.2
Approx 1,800 ft. D/S of Red Hickory Lane	1,156	1,264	601.3	602.4	1.1	602.4	1.1
Approx 600 ft. U/S of Red Hickory Lane	901	1,124	612.8	610.1	-2.7	610.4	-2.4
Approx 1,200 ft. D/S of Brown Grier Rd	463	523	619.9	619.2	-0.7	619.2	-0.7
Stewart Creek							
At Confluence with Irwin Creek	3,513	6,184	629.8	631.2	1.4	630.9	1.1
Approx 400 ft. D/S of Rozzelles Ferry Rd	2,636	5,802	651.2	652.5	1.3	652.4	1.2
Approx 300 ft. U/S of Southwest Blvd	1,536	4,929	675.2	677.5	2.3	677.7	2.5
Approx 2,400 ft. U/S of Hoskins Road	802	1,936	699.1	703.5	4.4	703.5	4.4
Stewart Creek Tributary 1							
At confluence with Stewart Creek	2,544	2,774	637.0	633.5	-3.5	633.5	-3.5
Stewart Creek Tributary 2							
At Confluence with Stewart Creek	2,617	3,472	647.8	651.1	3.3	649.6	1.8
Approx 200 ft. U/S of Barlowe Road	1,068	1,336	701.2	700.3	-0.9	702.6	1.4
Stewart Creek Tributary 3							
At Confluence with Stewart Creek	1,814	1,626	673.0	673.9	0.9	673.9	0.9
Approx 100 ft. D/S of Hoskins Road	1,198	1,518	723.4	714.2	-9.2	714.1	-9.3
Sugar Creek							
At County Line	13,469	16,994	538.1	536.2	-1.9	538.1	0.0
Approx 2,800 ft. U/S of I-77	11,686	14,367	575.3	578.3	3.0	578.0	2.7
Taggart Creek							
At Confluence with Sugar Creek	2,346	5,120	601.6	599.9	-1.7	599.9	-1.7

	Eff Q (cfs)	Sim Q (cfs)	Eff 1% WSEL	Sim 1% WSEL	Diff (ft)	Cal Sim 1% WSEL	Diff (ft)
Taggart Creek (continued)							
Approx 900 ft. D/S of West Boulevard	1,979	4,532	611.8	613.4	1.6	613.3	1.5
Approx 1,600 ft. D/S of Morris Field Dr	1,856	3,449	626.1	628.5	2.4	628.6	2.4
Approx 100 ft. U/S of Winston Cont Rd	1,682	1,646	637.8	640.0	2.2	639.9	2.1
Approx 200 ft. D/S of Mulberry Ch Rd	1,168	1,304	680.3	686.8	6.5	679.8	-0.5
Approx 100 ft. U/S of Mulberry Ch Rd	909	746	680.7	686.9	6.2	680.3	-0.4
Walker Branch							
At Confluence with Steele Creek	2,713	4,551	564.8	564.1	-0.7	564.1	-0.7
Approx 750 ft. U/S of Confluence	2,169	3,163	565.6	566.3	0.7	566.3	0.7
Approx 2,500 ft. D/S of Hwy 49	1,589	1,760	582.5	582.4	-0.1	582.4	-0.1
Walker Branch Tributary							
At Confluence with Walker Branch	903	1,549	583.1	583.2	0.1	583.1	0.0

The differences displayed in red text represent simulated BFE's that have decreased by more than 1 foot from the effective BFE's at the same location. The differences in blue text represent simulated BFE's that have increased by more than 1 foot from the effective BFE's, and black text represent a BFE change of less than 1 foot relative to the effective. The over 9 foot difference identified on Stewart Creek Tributary 3 is attributed to the effective RAS model having a blocked obstruction at the downstream end of the RR culvert. A definitive explanation for the blocked obstruction could not immediately be found. However, investigation into the effective modeling has revealed that the addition of blocked obstructions downstream of culverts was a practice used in the past to eliminate crossing profiles. While it cannot be verified with absolute certainty that the obstructions were added to fix crossing profiles, removing the obstructions from the effective model at the downstream face of Hoskins Road induces crossing profiles at this location.

The 3 foot decrease from the effective BFE that occurs on Coffey Creek approximately 600 feet downstream of Piney Top Drive (at simulated station 28455) is an anomaly. At all other cross sections between Piney Top Dr and Bynum Dr, simulated BFEs are consistently 1' to 1.5' lower than the effective, which is likely attributable to the significantly higher n-values used in the effective RAS model (0.062 in the channel, 0.18 to 2.0 in the overbanks). However, at effective station 27855, the BFE falls to critical depth, which appears to artificially push up the effective BFE at the next US XS, where the simulated BFE is 3' lower. The simulated RAS model does not default to critical depth at effective station 27855 (simulated station 28100), and as a result, is not artificially elevated at the location in question (simulated station 28455) as the effective BFE is.

The 4.2 feet increase relative to the effective BFE identified on Irwin Creek approximately 2400 feet downstream of I-85 Service Road (called "Access Rd 04" in the simulation) appears to be due to the fact that the simulated 1% discharge is approximately 800 cfs greater than the effective at this location. The effective model uses a discharge of 3230 cfs from approximately 2400' downstream of I-85 to immediately downstream of I-277, where it then increases to 6400 cfs. The simulated 1% discharge increases from 3663, to 4080, to 6401, to 6781 cfs over the same expanse. The cumulative effect of these discharge differences (which are significant at the downstream end of the reach described above in the vicinity of Oaklawn Ave, where the effective discharge is 3230 and the simulated discharge is 6781 cfs) is a 4.2' difference in BFE. In the reach between I-85 and the I-85 service road (Access Road 04 in the simulation) where the discharges are again similar, the effective and simulated BFEs converge again.

Kennedy Branch at the confluence with Irwin Creek is 4.2 feet less than effective due to the differences in cross section geometry, invert slope, and general invert elevation that is revealed by the better topographic data used in the simulation. The effective RAS model used a normal depth slope boundary condition value of 0.002, with invert elevations starting at 654.3' at the downstream end of Kennedy Branch, and increasing to 657.3' at the downstream face of the footbridge at effective station 1803'. The simulated RAS model's normal depth slope boundary condition is a somewhat steeper 0.0054, but with channel depths and invert elevations that are consistently 3' deeper than those in the effective model in the downstream reach (starting at 651.5' at the downstream end of Kennedy Branch). This difference in channel depth and normal depth slope results in a significant difference in BFE at the confluence with Irwin Creek. However, this BFE difference is negated by the fact that the simulated RAS model shows that the downstream reach of Kennedy Branch is inundated by backwater from Irwin Creek until immediately downstream of the footbridge at simulated station 1795' (effective station 1803').

Kings Branch at the confluence with Sugar Creek is 4.6 feet less than effective due to the differences in cross section geometry, and invert slope that is revealed by the better topographic data used in the simulation. The effective RAS model used a normal depth slope boundary condition value of 0.001, with invert elevations starting at 527.19' at the downstream end of Kings Branch, and increasing to 530.54' at the downstream face of I-485 culverts at effective station 127.3'. The simulated RAS model's normal depth slope boundary condition is a significantly steeper 0.012, and also begins immediately at the downstream face of I-485 culverts, rather than a lead-up buffer cross-section. This difference in normal depth slope with cross-section and structure placement in the model results in a significant difference in BFE at the confluence with Sugar Creek. However, this BFE difference is negated by the fact that the simulated RAS model shows that the downstream reach of Kings Branch is inundated by backwater from Sugar Creek until immediately downstream of the Westinghouse Blvd. culverts at simulated station 2165' (effective station 2167.5'). Kings Branch, 200 feet upstream of Archdale is 3.8 feet greater than effective due to the fact that the simulated 1% discharge is approximately 1690 cfs greater than the effective at this location. The effective model uses a discharge of 1060 cfs from approximately 200' upstream of Archdale Rd. to approximately 1,000' downstream of Kings Branch Ct., where it then increases to 1240 cfs. The simulated 1% discharge increases from 2749, to 2788, to 3171, then decreases slightly to 3166 cfs over the same expanse.

Stewart Creek, 2,400 feet upstream of Hoskins Rd is 4.2 feet greater than effective mainly due to the increased discharges calculated in the updated hydrology modeling, see the Preliminary Sugar/Irwin Hydrology Report. Preliminary discharges upstream of Southwest Boulevard have increased by between 140 – 220%.

Hydraulic Modeling Calibration

As specified in the county's *Floodplain Analysis and Mapping Standards Guidance Document*, calibration of the hydraulic models was conducted in order to ensure that the models accurately reflect the conditions as they exist on the ground. This was accomplished through comparison of observed water surface elevations from a known storm event (in this case, the August 2008 and August 2011 storms) with those yielded by the hydraulic models when using similar discharges. The simulated discharges that were used for this comparison were initially calculated using the recorded precipitation data from the event of interest in the hydrologic models that were developed in conjunction with this analysis (more detailed information about the development of these discharges can be found in the "*Mecklenburg County Floodplain Mapping 2008: Sugar/Irwin Sub-Basin Hydrology Report"*). If the simulated discharges from the HMS models were not completely calibrated to match the observed data then the observed USGS peak discharge data was input into the RAS models so an independent hydraulic calibration could be performed. The gage data was interpolated upstream and downstream of the gage sites using the ratio of the gage data to the HMS data at the gage location. If two gages were available on the stream, the average ratio was applied to the simulated discharges in between the gages. For the August 2011 storm, gage 0212430293 on Irwin Creek recorded a peak discharge of 10600 cfs while gage 02146300 recorded 4055 cfs. The calibrated HMS model calculated peak discharges of 11878 and 4231 cfs, respectively. The average observed/simulated ratio = 0.9254 was used to produce the observed discharges in between the two gages. The individual ratio at gage 02146300 was 0.9584 and that ratio was applied to each simulated discharge as you progressed upstream. If a stream had no gage, no discharge adjustment was made. Various parameters of the hydraulic models were then revised as needed in an attempt to match the observed elevation values within +/- 0.5 feet.

Table 4: Stream Gages used for Sugar/Irwin River sub-basin Model Parameter Calibration

Gage Station ID	Gaged Stream and Location	Drainage Area (sq mi)
02146211	Irwin Creek at Statesville Ave at Charlotte, NC	6.0
0214627970	Stewart Creek at State St at Charlotte, NC	9.3
02146285	Stewart Creek at West Morehead St at Charlotte, NC	11.1
02146300	Irwin Creek near Charlotte, NC	30.7
02146315	Taggart Creek at State St at Charlotte, NC	5.7
02146348	Coffey Creek near Charlotte, NC	9.1
02146381	Sugar Creek at NC 51 near Pineville, NC	65.3
0214678175	Steele Creek at SR 1441 near Pineville, NC	6.7

The available observed water surface elevation data for the August 2008 and 2011 storms were derived from several USGS gages located along the creeks seen in Table 4, as well as from surveys of high water marks (HWMs) on Coffey Creek, Irwin Creek, Kennedy Branch, Kings Branch, McCullough Branch, Steele Creek, Stewart Creek and its Tributaries, Sugar Creek, and Taggart Creek that were conducted in the days subsequent to the storm events.

Calibration to Stream Gage Data

In accordance with the county's *Floodplain Analysis and Mapping Standards Guidance Document*, primary consideration during the hydraulic calibration phase was given to the observed WSELs recorded at the stream gaging stations. Discharge and stage data were available from the USGS in 15-minute increments at each station, and peak flow values (and the corresponding stages) were used as the calibration values. A comparison of the simulated and observed water surface elevations for the August 2011 event at each USGS gage location can be found in table 5.

Table 5: Stream Gage versus Simulated Water Surface Elevations for the 2011 Storm

Stream	Gage ID	Model XS Station	Gage Elevation (ft)	Pre-Cal XS Elevation (ft)	pre-cal diff (ft)	Post-Cal XS Elevation (ft)	post-cal diff (ft)
Coffey Creek	2146348	6125	576.09	573.20	-2.89	573.82	-2.27
Irwin Creek	2146300	4069	610.70	611.6	0.91	611.4	0.7
Irwin Creek	2146211	38173	669.41	672.92	3.51	672.61	3.2
Stewart Creek	2146285	1368	636.05	634.58	-1.47	634.71	-1.34
Stewart Creek	214627970	6439	645.85	648.5	2.65	646.79	0.94
Sugar Creek	2146381	6588	534.25	540.34	6.09	535.72	1.47
Taggart Creek	2146315	6272	615.34	618.16	2.82	617.23	1.89

Generally speaking, simulated elevations are high at 5 of the 7 gage stations for the 2011 storm event. The 214627970 gage on Stewart Creek could be in backwater from Irwin Creek and that could have an impact on the calculations. The Coffey Creek gage elevation is 2.3' higher than the simulated elevation and there appears no apparent explanation other than an obstructed culvert opening at Tryon Street during the storm.

There also appears to be some discrepancy in the reporting of the stage elevations versus the HWM measurements at the USGS gages; such as the Taggart Creek gage, where the observable elevation apparently exceeded the maximum measurable level of the gage. Therefore, this comparison cannot be accurately assessed.

Calibration to High Water Marks

A total of 40 HWM surveys were conducted in the Sugar/Irwin watershed in the days following the August 2008 event. There were 44 surveys performed after the August 2011 storm event. These surveyed HWMs were used in the calibration process as secondary targets due to their more variable nature relative to the gage measurements. In light of this, somewhat less rigorous efforts were made to bring the hydraulic models into agreement with the HWMs, with agreement being achieved with varying degrees of success. A tabulation of calibration results at each HWM can be found in the Sugar/Irwin High Water Mark Spreadsheet.

Most of the HWMs were calibrated by re-drawing downstream cross sections, adjusting the n-values, or by adjusting how a downstream bridge or culvert was modeled. The adjustments were successful most of the time as there were about half of the HWMs that simulations could not get within +/- 1.0 feet. Of those marks, several can be explained by mitigating circumstances. Please see the HWM spreadsheet for more detail. There were several instances of the DS face of a structure having a high water measurement that was higher than the US face and even an instance where we believe the HWM measurement was due storm water system issues and not riverine flooding issues. There were also several instances of the HWM being in backwater from a major system which can not be represented in the RAS model.

There is an area that presents consistent differences between the simulated model elevations and the measured high water marks: on Irwin Creek from approximately station 12500 upstream thru the end of the model. The simulated elevations are consistently high as compared to the HWMs. They range from 1 foot to 5 feet higher. The simulated discharges were adjusted to match the observed discharges at the gages and interpolated upstream and downstream, which made the simulated elevations match better but were still somewhat higher than observed. There are several large structures in this area including I-77 and several of its ramps that could have a significant impact on elevations. Investigation into these structures revealed that all were input into the model correctly and specific model parameters lie within industry standards. Therefore, it appears that in order to calibrate the hydraulic model to match observed elevations, one would have to adjust parameters outside of standard ranges and we are hesitant to do that.

Calibration along streams with no Historic Flood Data

Streams with no stream gage data were not specifically calibrated at this time.

Floodways and Community Encroachment Boundaries

The floodway represents the portion of the channel or other watercourse and the adjacent land area that should be reserved/maintained to carry the base flood without increasing flood elevations by more than a specified maximum tolerance. As specified in the county's *Floodplain Analysis and Mapping Standards Guidance Document*, two floodways were created for the Mecklenburg County FIRMs; the FEMA Floodway and the Community Encroachment Boundary.

FEMA Floodway

Encroachments for the FEMA floodway were initially set using method 4 in the encroachment routine in HEC-RAS version 4.1.0 using 0.5 feet as a target surcharge. The 1% annual chance existing conditions discharges were used in this process. Calculated surcharge values from the FEMA floodway analysis ranged from -0.04 to 0.54 feet and surcharges were optimized to be reasonably close to 0.5 ft. Floodway widths were also optimized to represent gradual changes from cross section to cross section in order to conform to the County's Guidance Document.

Community Encroachment Area

The community encroachment area (CEA) was determined using a 0.1 foot maximum surcharge using the ***modified*** 100-year existing conditions base flood discharges. The 100-year existing conditions discharge was modified to account for the future loss of storage due to the filling of the floodplain fringe to the FEMA floodway. The ***modified*** 100-year discharges were then used to re-compute the CEA Boundary Line optimizing the surcharge to be as close to 0.1 as possible. Calculated surcharge values from the Community Encroachment Area analysis ranged from -0.04 to 0.14 and all surcharges were optimized to be reasonably close to 0.1 ft. The widths were again optimized similarly to the FEMA floodways and in accordance with the Guidance Document.