Ellicott City Flood Study
And
Concept Mitigation Report

McCormick Taylor Project No. 5493-01
April 3, 2014

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EXECUTIVE SUMMARY

This study was conducted at the request of Howard County Bureau of Environmental Services for the purposes of creating a detailed hydraulic model of the flood flows encountered along Main Street in Ellicott City, Howard County, Maryland, and using that model to examine the effect of proposed conceptual improvements on flooding conditions. Several hydrologic models of the Hudson Branch watershed were created to calibrate a baseline hydrologic model which included the effects of existing stormwater quantity management within the watershed. The hydrology of the flooding event of September 7, 2011 (remnants of Tropical Storm Lee) was also recreated to calibrate the hydraulic model against observed flooding conditions during the event.

Creation of the hydraulic floodplain model along Main Street (from Rogers Avenue to the confluence with the Tiber River) included the use of one-dimensional (HEC-RAS v.4.1) and two-dimensional (TUFLOW) models to most accurately represent the complex flow through the terrain and structures present within the floodplain along Main Street. Once developed and calibrated, this model served as a baseline for the comparison of various flood mitigation concepts, including additional stormwater management in the uplands watershed and additional storm drain and culvert conveyance systems to supplement the existing channel.

Additional stormwater management concepts, designed to reduce the volume of water reaching Main Street during an intense rain event, were limited by available publically-owned rights-of-way because the majority of the watershed is currently built out with commercial and residential properties. These locations focused on unused areas within the US 40 / US 29 interchange. The additional storm drain conveyance options were limited by physical constraints such as elevation to tie into the existing systems, adequate cover under roadways, and available space adjacent to and between existing structures.

The proposed modeling analyzed the opportunities to provide additional quantity management and improved conveyance within the study area and provided a comparison to existing conditions.

1.0 INTRODUCTION

1.1 BACKGROUND

Hudson Branch, a tributary of the Tiber River (a.k.a Tiber Branch), itself a tributary to the Patapsco River, winds along Main Street in Ellicott City, Howard County, Maryland. Runoff from the 1.6 square mile watershed to this stream, the upland boundaries of which extend north and west of the US 40 / US 29 interchange, flows through a confined channel and occasional storm culverts along both the north and south sides of Main Street before meeting its confluence with the Tiber in a parking lot south of Main Street (Parking Lot 'D'). The Tiber River continues eastward from Parking Lot 'D' in a confined channel to its ultimate confluence with the Patapsco.

The confined nature of the channel, due in part to the steep topography surrounding Main Street, as well as the historic buildings which line or straddle its immediate banks, contributes to the dramatic flooding experienced in the Main Street corridor during certain intense rainfall events. The development within the watershed, some of which is managed for quantity control to varying degrees, also plays a role. The severe flooding experienced on Main Street during the remnants of Tropical Storm Lee on September 7, 2011 was a prime example of an intense flooding event in this area; the storm flooded a sizeable stretch of Main Street and its surrounding homes and businesses with runoff anywhere from a few inches to several feet deep.

Based on the observations of flow during that storm, it was clear that the stream, which meanders back and forth across Main Street around several buildings and across the road in numerous places during higher flows, could not be fully modeled using traditional one-dimensional modeling software such as HEC-RAS. This analysis, performed using TUFLOW two-dimensional hydraulic modeling software along with detailed topographic survey, attempts to create a more accurate representation of typical Main Street flooding by considering the two-dimensional flow vectors resulting from floodwaters over this highly varied landscape. The establishment of this baseline flooding condition allows for a more accurate representation of the effect on flood elevations resulting from the various conceptual improvements examined within this study.

1.2 PROJECT LOCATION

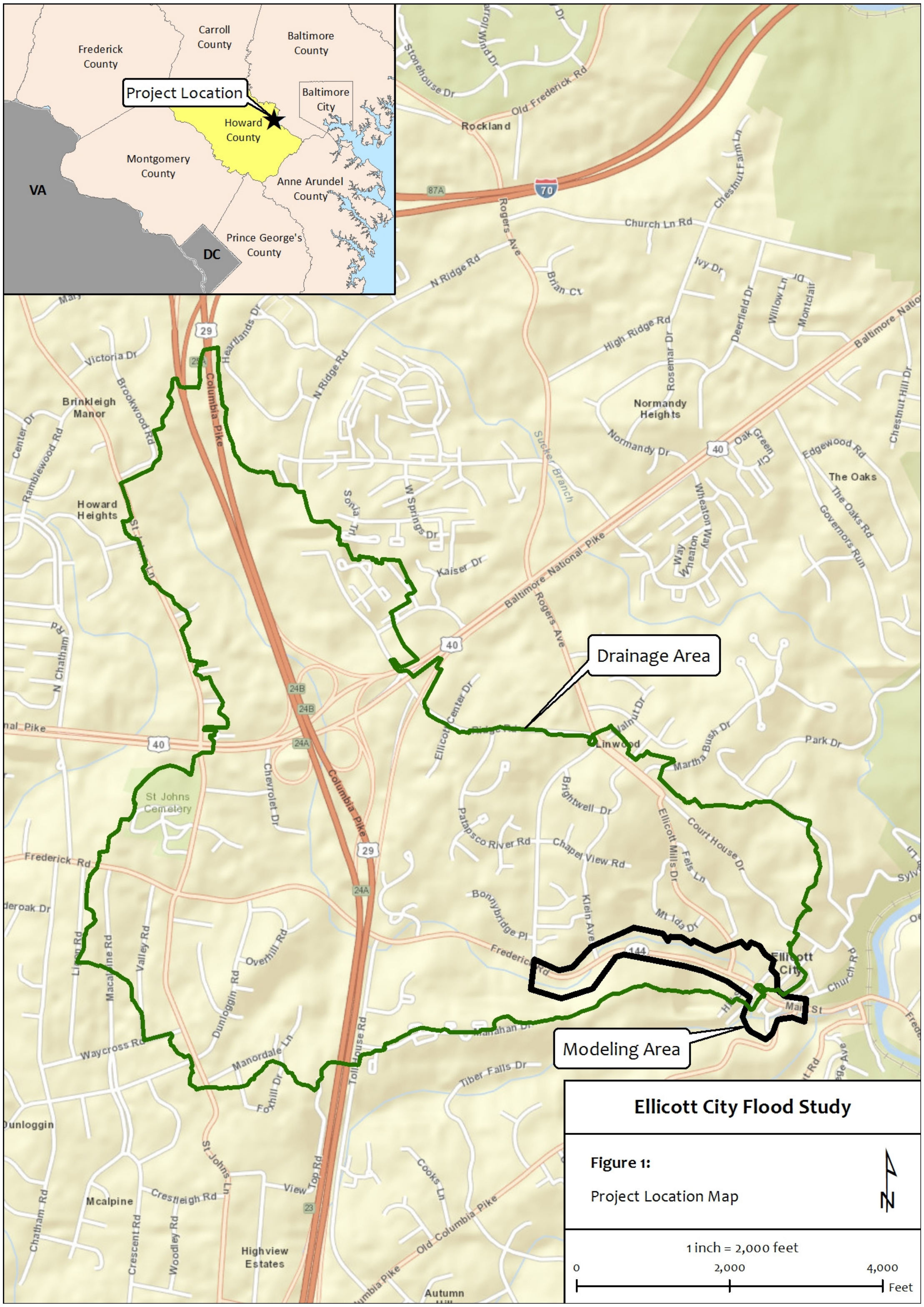
This study focused on the historic section of Ellicott City, Maryland. The extent of the detailed hydraulic floodplain analysis extends from the vicinity of the intersection of Main Street (a.k.a Frederick Road) and Rogers Ave. east through Parking Lot 'D'. The portion of stream channel from Rogers Ave to just east of 8600 Main Street (West End Services) was analyzed using HEC-RAS v.4.1; the remainder of the channel downstream from that point was analyzed using TUFLOW to establish a two-dimensional floodplain surface. The location of the project and the subject watershed can be seen in *Figure 1.1*.

1.3 PROJECT GOALS

The goals of this study include the following:

- Develop hydrology for the Hudson Branch watershed that may be used in subsequent hydraulic analyses of the area, and considers the effect of existing stormwater quantity management as a baseline for future analysis.
- Develop a two-dimensional hydraulic floodplain model through the area affected by the Main Street flooding during the Tropical Storm Lee event and calibrate the model based on observed conditions that day.
- Develop potential improvements to the hydrology of the Hudson Branch (additional management of stormwater quantity) and the hydraulics of the conveyance network through the town (improvements to channels, floodplains and storm drain systems to increase conveyance through this area, and define limitations of the existing network).
- Quantify the potential positive impacts to flood elevation and frequency as a result of the conceptual improvements, using the baseline hydrologic and hydraulic models developed for existing conditions as a means of comparison.

Figure 1.1: Vicinity Map of the Ellicott City Flood Study Area



2.0 HYDROLOGIC ANALYSIS

In order to determine the proper hydrologic flow quantities for use in the study, several steps were employed. The drainage area (DA) was analyzed using TR-20 for a single drainage area, and also with seven (7) sub drainage areas, then compared to regional regression data for Piedmont Urban areas. This TR-20 model was subsequently subdivided further to show the effect of existing and proposed stormwater management within the hydrologic model.

Once the architecture of the TR-20 model was set, rain gage data from the Tropical Storm Lee event of September 7, 2011 was used to create a rain table for use in the TR-20 model that would mimic the precipitation from that event. The flow data generated through the TR-20 hydrologic model, was then used as input for hydraulic models, which were calibrated using anecdotal information (witness accounts, video) about local water surface elevations during the Tropical Storm Lee flood event. The hydrologic details of this sequential analysis are described below.

2.1 INITIAL TR-20 ANALYSIS

Hydrologic modeling was used to generate recurrence interval discharges for the study site based on existing land use and soil conditions. USDA Soil Conservation Service (SCS) TR-55 and TR-20 computer programs were used to determine runoff from the watershed area. The downstream study point used to determine the drainage area for the study was located at the upstream end of the Hudson Branch channel into Parking Lot ‘D’, just upstream of the confluence with Tiber Run.

The initial analysis did not subdivide the 1.56 square mile watershed. The second analysis subdivided the drainage area into seven (7) subwatersheds based on their configuration within the watershed and/or significant changes in the predominant land use type. For the subdivided analysis, reach routing section tables used in the TR-20 model were developed in the GIS Hydro Program environment or from GIS contour data and Flowmaster analysis.

The overall drainage area to the study point consists of a mix of residential (low, medium and higher density) and commercial/urban areas, the interchange of US 29 and US 40, and some undeveloped open/wooded space in the northern portion of the watershed and the hillier terrain along the southern and eastern perimeter. Soil types include B, C and D Hydrologic Soil Groups, the percentages are as noted below.

Table 2.1 - Hydrologic Soils Distribution

Hydrologic Soil Group	*% of Drainage Area*
A	0%
B	63%
C	8%
D	29%

Table 2.2 - Land Use Information

Land Use	*Percent of Watershed*
Brush / woods	18
Pasture / open space	9
Impervious (roads, parking not incl. below)	6
Residential - 1 ac.	16
Residential – 1/4 to 1/8 ac.	28
Urban Commercial	15
Urban Industrial	8

Land use was derived from County GIS data and aerial photography, the breakdown is noted above in Table 2.2. Soils information for the project was obtained from the Web Soil Survey developed by NRCS (http://websoilsurvey.nrcs.usda.gov/). This data together was used to determine curve number values for each study point using TR-55 methodology. See *Appendix A* for curve number computations. See *Appendix A* for Hydrologic Soils Maps as well as Land Use and Drainage Area Maps.

TR-55 methodology was also used for time of concentration calculations. An analysis of the overall drainage area indicated a total time of concentration of 1.136 hours, or 68.2 minutes, to the downstream study point for the single drainage area analysis. See *Appendix A* for time of concentration computations.

The rainfall depths for the 24 hour duration storm were obtained data from the Precipitation Frequency Data Server, maintained by the Hydrometeorological Design Studies Center (HDSC) of NOAA's National Weather Service (http://www.nws.noaa.gov/ohd/hdsc/) shown below.

Table 2.3 - Atlas 14 Rainfall Data

Return Period (years)	*Rainfall Depth w/ area reduction (inches)*
2	3.21
5	4.13
10	4.94
50	7.28
100	8.53

Discharges were calculated for the 2-, 5-, 10-, 50- and 100-year recurrence intervals. The 24-hour Type II event was used for all analyses except where shown in *Section 2.3.3* below, as this is the standard for stormwater management analysis in Maryland. The results of the TR-20 analysis for the watershed analyzed as a Single DA and analyzed as Seven Subareas are shown in the section below in *Table 2.5*.

2.2 REGIONAL REGRESSION CALIBRATION

In order to calibrate the TR-20 results for the study area, the Thomas Fixed Region regression equations were used in this analysis. The equations for this physiographic region were developed based on a generalized least-squares regression of the peak flow and basin characteristic data from 16 urban (impervious area > 10%) watershed stations in the Piedmont Region. For the urban equations, drainage area (DA) ranges from 0.49 to 102.05 square miles and impervious area (IA) from 10.9 to 42.8 percent. The standard errors range from 26.0 percent (0.111 log units) for Q_{25} to 41.7 percent (0.174 log units) for$Q_{1.25}$. As the drainage area size is within the range noted above, the following equations were used (the FR data summary can be found in *Appendix A*).

Table 2.4 - Fixed Region Regression Equations

Piedmont (Urban) Fixed Region Regression Equation	Standard error (percent)	Equivalent years of record
$Q_{1.25} = 17.85\ DA^{0.652}\ (IA+1)^{0.635}$	41.7	3.3
$Q_{1.50} = 24.66\ DA^{0.648}\ (IA+1)^{0.631}$	36.9	3.8
$Q_{2} = 37.01\ DA^{0.635}\ (IA+1)^{0.588}$	35.1	4.5
$Q_{5} = 94.76\ DA^{0.624}\ (IA+1)^{0.499}$	28.5	13
$Q_{10} = 169.2\ DA^{0.622}\ (IA+1)^{0.435}$	26.2	24
$Q_{25} = 341.0\ DA^{0.619}\ (IA+1)^{0.349}$	26.0	38
$Q_{50} = 562.4\ DA^{0.619}\ (IA+1)^{0.284}$	27.7	44
$Q_{100} = 898.3\ DA^{0.619}\ (IA+1)^{0.222}$	30.7	45
$Q_{200} = 1413\ DA^{0.621}\ (IA+1)^{0.160}$	34.8	44
$Q_{500} = 2529\ DA^{0.623}\ (IA+1)^{0.079}$	41.2	40

For the equations above, the 'IA' represents the % impervious area. GIS Hydro was used to calculate this value (41.7%) based on existing land use data layers. The Panel Report also recommends using the single standard error prediction interval to obtain an acceptable range of discharges. The results of the analysis, including the acceptable range based on the Q +/- 1 Std. Error, are shown below.

Table 2.5 - Initial Hydrology Calibration Results

Return Period (years)	*Single DA (cfs)*	*7 Subareas (cfs)*	*FR Eq. (cfs)*	*FR Eq. +1 SE (cfs)*
2	636	648	453	612
5	1006	1026	827	1063
10	1347	1403	1160	1464
50	2384	2473	2180	2784
100	2940	3084	2770	3620

Based on these results, the TR-20 values fall within the desired calibration range, indicating that the hydrologic model is acceptable for further analysis. This model will be used as the starting point for the additional modeling noted below.

2.3 Analysis With Existing Stormwater Management

The drainage area features many communities and commercial sites with existing stormwater management, which varies from just water quality and/or 2-

and 10-year management to full 100-year management. There are County records of 26 SWM quantity management facilities within the watershed, some of which have detailed design computations and records and others where the as built data is sporadic. Also, some of the ponds are small enough relative to the watershed size that their impact on overall watershed hydrology is questionable. In order to consider both of these factors, and come up with a reasonable approach to approximating the management effects of small facilities, a small sample was examined to determine the effect of approximation for smaller facilities before applying this approach to the entire watershed, as detailed below.

2.3.1 DEVELOPMENT OF METHODOLOGY

The eight largest facilities, all with drainage areas of approximately 6 acres or greater, were represented as structures in the model and routed accordingly to model their effects on management. From as-built drawings and computations, storage-discharge tables were developed to model the effects of each of these eight storage structures. Runoff from upstream was routed through the structures, then added (ADDHYD) to other runoff areas within the model. Please refer to the drainage area map located in *Appendix A* that details the subareas and SWM described below.

The eighteen smaller SWM facilities each have drainage areas less than 6 acres. To approximate the effect of their management, these facilities were incorporated into the TR-20 model by reducing the curve number (CN) of the drainage area to reflect runoff conditions under a “woods in good condition” land use. The CN of each SWM facility drainage area was added to the CN of the surrounding subarea to create a weighted average CN that reflects reduction in runoff resulting from small SWM facilities. The methodology behind using this method is described below:

SWM facilities 5 and 14 (Subarea 3) were evaluated to investigate the best way to represent small SWM facilities. First, SWM 5 and SWM 14 (3.7 and 4.3 acres, respectively) were modeled using the same method used for large SWM facilities, which incorporates routing with stage-storage-discharge tables from the as built computations (Table 2.6, Method 1). Method 1 should be the most accurate at describing the management effects because it incorporates the unique characteristics of each facility. Method 1 could not be applied to all small SWM facilities because not enough detailed design/as built information exists to accurately recreate the required storage-discharge tables for all facilities. Method 1 (pond routing) was then used as a baseline comparison for Method 2.

Method 2 estimates the management effects of SWM facilities 5 and 14 by using the weighted average CN method described above.

The “No management” scenario depicted in Table 2.6 shows the results when the effects of SWM facilities 5 and 14 are not included in the model in any way. In the table below Outlet SA 3-3 is the outlet of subarea 3-3 (containing SWM facility 14), Outlet SA 3-4 is the outlet of subarea 3-4 (containing SWM facility 5), and DA 3 outlet is the confluence of discharges from subareas 3-1 through 3-8.

Table 2.6 - Peak discharge and timing for three outlet locations.

10-year Storm						
	Method 1		***Method 2***		***No Management***	
Location	Peak Flow (cfs)	Timing (hr)	Peak Flow (cfs)	Timing (hr)	Peak Flow (cfs)	Timing (hr)
Outlet SA 3-3	92.8	12.09	93.8	12.09	108.1	12.09
Outlet SA 3-4	163.9	12.05	164.8	12.05	173	12.05
Outlet DA 3	583.2	12.16	584.3	12.16	604.5	12.15
50-year Storm						
	Method 1		***Method 2***		***No Management***	
Location	Peak Flow (cfs)	Timing (hr)	Peak Flow (cfs)	Timing (hr)	Peak Flow (cfs)	Timing (hr)
Outlet SA 3-3	175.7	12.09	174.2	12.08	192.8	12.08
Outlet SA 3-4	263.0	12.05	270.1	12.05	278.6	12.04
Outlet DA 3	1067.2	12.16	1067.4	12.15	1089.7	12.15
100-year Storm						
	Method 1		***Method 2***		***No Management***	
Location	Peak Flow (cfs)	Timing (hr)	Peak Flow (cfs)	Timing (hr)	Peak Flow (cfs)	Timing (hr)
Outlet SA 3-3	226.8	12.09	221.2	12.08	238.1	12.08
Outlet SA 3-4	318.4	12.04	327.4	12.04	333.6	12.04
Outlet DA 3	1336.1	12.16	1335.9	12.16	1355.3	12.16

Based on the data above, the peak discharges and timing generated through Method 2 correspond reasonably well with those generated with Method 1. Method 2 slightly underestimates peak discharge at outlet SA 3-3 and slightly overestimates SA 3-4 though these variations are well within the expected relative error associated with hydrologic modeling of this nature. More notably, peak discharge and timing at the primary downstream outlet (Outlet DA 3) are almost identical to results for both methods (1 and 2). This indicates that any variations in the flow values between these two methodologies are essentially negated by the time the routing reaches the outlet of DA 3 (and therefore, the downstream study point) demonstrating this to be a reasonable approximation for the smaller areas.

Methods 1 and 2 were also compared for SWM facilities 18 and 19, in Subarea 6. SWM facilities 18 and 19 have drainage areas of approximately 29 and 7 acres, respectively. When applied to DA 6, the weighted average CN method (Method 2) overestimated peak discharge at the immediate drainage outlet by 8% for the 10-yr storm to 19% for the 100 yr-storm (Table 2.7). When applied only to SWM facility 19 (7 acres), the peak discharge overestimated by approximately 5% for all storms. This indicated the approximation Method 2 approach may be less suitable for ponds with larger drainage areas in this watershed.

Table 2.7 - Peak discharge for DA 6, SWM Facilities 18 and 19.

Storm Event	Method 1	Method 2	No Management
	Peak Flow (cfs)	Peak Flow (cfs)	Peak Flow (cfs)
10-yr	197.3	213.3	305.7
50-yr	357.4	414.6	524.6
100-yr	443.9	529.5	642.0

On this basis, we have determined for the purposes of establishing design flows and modeling the impacts of existing SWM on these flows at the overall downstream study points, the curve number reduction method (Method 2 above) would be applied to managed drainage areas less than approximately 6 acres, with resulting accuracy within expected tolerances. The remaining areas, where full stage-storage data is available without additional detailed survey, will be modeled using Method 1. A list of the SWM facilities and how they were modeled can be found in *Appendix A*.

2.3.2 RESULTS WITH EXISTING SWM

Using the methods described above, the hydrologic results using the 24-hour Type II storm event are noted in Table 2.8 below. For the additional subdivision of drainage areas, in order to separate managed areas from non-managed areas, additional reach routing was performed for routing lengths greater than 500', however in most instances the routing was not significant enough to alter the results. Subareas were otherwise added (ADDHYD) together within each of the 7 drainage areas. Note that the additional subdivision of the DA to 35 subareas results in flow increases over the single and 7 DA models, however this was necessary to include all individually managed areas. Including effects of existing management brings the values back in line with the calibration window discussed above. Since the regression equations are based on stream flow data (which includes the effects of existing management in urban areas where gage data is taken for derivation of the curves) this is consistent with expectations.

Table 2.8 - Subdivided Hydrology and Existing Management Results

Return Period (years)	Peak Flow, TR-20 subdivided (35 DAs) but no SWM (cfs)	Peak Flow, TR-subdivided (35 DAs) large SWM only (cfs)	Peak Flow, TR-20 subdivided (35 DAs) all quantity SWM facilities (cfs)
10	1639	1437	**1356**
50	2998	2738	**2647**
100	3948	3647	**3549**

Table 2.8 shows the peak flow at the study point for the TR-20 model under 3 scenarios. Ultimately, the third scenario incorporating all quantity SWM facilities was used as the most representative scenario for the watershed and was used as the basis for a comparison baseline of future concept improvements.

2.3.3 TROPICAL STORM LEE HYDROLOGY

In order to calibrate the hydraulic model to the conditions that were observed and recorded during the Tropical Storm Lee event, it was necessary to use rainfall data from that actual event, rather than a standard 24- hour Type II storm and rainfall table, as the precipitation that day fell predominantly in a much shorter timeframe (the majority in 2-3 hours) and did not necessarily mimic the curve of the standard hyetograph. To accomplish this, a custom rainfall table was created and used within TR-20 to mimic the precipitation and runoff from that storm.

The precipitation that day was 'banded' in tight, intense bands that rode longitudinally along their major (predominantly north-south) axis, which led to a high variation in intensity and total rainfall just a few miles east or west of the bands. Therefore, official precipitation data from BWI was not favorable for use in this study despite its ~10-mile proximity to the watershed. There were two sources of automated rainfall data available within, or just outside of, the watershed: The County offices located in the northeast portion of the watershed (raw data was provided by Howard County for this study) and an established weather station in Mount Hebron (station #KMDELLIC3, operated by the weather forecasting and reporting website "Weather Underground") just outside the NW boundary of the watershed. The station has operated since 2005 with consistent data throughout that time period.

http://www.wunderground.com/weatherstation/WXDailyHistory.asp?ID=KMDELLIC3&month=9&day=7&year=2011

The County data indicated 3.57 inches of rainfall on that day. The weather station KMDELLIC3 indicated a daily total of 8.11 inches, with 4.89 inches falling in just over 2 hours and peak rainfall rates of 3.3 inches/hour. The KMDELLIC3 data was chosen to create the rainfall table, as it was more consistent with the radar estimated precipitation peak values that were graphically shown by NOAA/NWS to be within the study watershed that day. Both 6-hr and 24-hr storm durations were considered, but ultimately the 24-hour storm resulted in the more representative flooding conditions. As discussed below, these values calibrated favorably with the conditions observed and recorded by residents that day.

Table 2.9 - TS Lee Simulated Hydrology Results

Storm Event	*Peak Flow, TR-20 subdivided (35 DAs) but no SWM (cfs)*	*Peak Flow, TR-subdivided (35 DAs) large SWM only (cfs)*	*Peak Flow, TR-20 subdivided (35 DAs) all quantity SWM facilities (cfs)*
Tropical Storm Lee (24 hrs)	2340	2192	**2122**
Tropical Storm Lee (6 hrs)	1949	1724	**1643**

2.3.4 TIBER BRANCH HYDROLOGY

In order to consider the tailwater effects on the Hudson Branch at the confluence with the Tiber Branch in Parking Lot 'D' to form the Tiber River, a hydrologic analysis of the Tiber Branch watershed to the confluence was performed based on a single drainage area, and using the TR-55/TR-20 methodologies described above. Existing SWM within this watershed was not independently considered as this flow occurs outside of the focus area for the majority of this study. The details of this analysis may be found in *Appendix A*. The resulting flows used in the hydraulic modeling of the tailwater and the channel through Parking Lot 'D' are noted below:

Table 2.10 - Tiber Branch Hydrology Results

Return Period (years)	*Peak Flow, TR-20 subdivided (1 DA) and no SWM (cfs)*
5	295
10	423
50	828
100	1058
Tropical Storm Lee (24hr)	653

3.0 HYDRAULIC MODELING

The Main Street Ellicott City Flood study utilized a combination of one-dimensional and two-dimensional modeling tools to develop the floodplain analysis through the study area. As the two-dimensional modeling is quite data intensive, its use was limited to the sections of the study where the worst flooding was observed, and where it was clear that the effect of the terrain on the direction of flow would be significantly different using this approach. The details of this approach are presented below.

3.1 HEC-RAS MODELING OF THE UPSTREAM REACH

The 1-Dimensional hydraulic model was utilized from Rogers Avenue to 8515 Frederick Road (Main St.). The hydraulic analysis was performed using the Army Corp of Engineers HEC-RAS (Hydrologic Engineering Center River Analysis System) computer program, Version 4.1.0. HEC-RAS is designed to compute one-dimensional flow profiles in natural and constructed stream channels by applying the energy equation between cross sections.

Models were generated for both existing and proposed conditions. Data used to develop the models included cross sections, Manning's n values, loss coefficients and boundary conditions. Both models were run in the mixed flow regime. The downstream boundary condition used normal depth with the existing downstream slope and upstream boundary condition used critical depth.

3.1.1 Cross Section Data

Cross section information was provided by cross section survey and supplemented by digital topography provided by Howard County. The existing condition and proposed conditions models consist of forty (40) cross sections. The cross sections begin downstream of Rogers Ave (River Station 40) and extend to 8515 Main Street (River Station 1) for a study reach length of approximately 2200 linear feet. The sections are coded from left to right looking downstream.

3.1.2 Starting Water Surface Elevations

Boundary conditions are required for the HEC-RAS models to compute the flow profiles. For the subcritical flow regime, a starting water surface elevation needed to be specified at the downstream bounding cross section (RS 1). The normal depth method was used as the downstream boundary condition. The downstream channel slope (0.0154 ft/ft) was used to approximate the energy slope.

3.1.3 Manning's "n" Values

The Manning's roughness coefficient, 'n', is an estimate of the resistance to flow in a given channel. Factors which may affect the roughness include bed material, vegetation, channel irregularities, obstructions and channel alignment. The Manning's 'n' values were assigned based on field investigations and tables provided in Chow's "Open Channel Hydraulics" Manual. The 'n' values used in this study range from 0.035 to 0.05 in the channel and 0.02 to 0.10 in the overbanks. The roughness was raised to 0.05 at station 28, 32, 33, and 34 to demonstrate the increased roughness due to turbulence within the channel in the 1-dimensional model.

3.1.4 Expansion and Contraction Coefficients

HEC-RAS computes energy losses due to the changes in flow area by multiplying the changes in the velocity head with the corresponding expansion and contraction coefficient. The typical gradual transition values of 0.1 and 0.3 for the expansion and contraction coefficients, respectively, were used throughout the model. Higher values of 0.3 and 0.5 were used at sections surrounding the bridges where more abrupt transitions within the cross sections occur.

3.1.5 Ineffective Flow Areas

HEC-RAS is a one-dimensional energy-balancing model that does not account for abrupt changes in flow conveyance and secondary eddies. The program assumes that all the area in the cross section is effective unless changes are made to the flow conveyance by blocking an area or increasing the roughness "n" values.

Ineffective flow areas are used to define areas of a cross section where water is not actively being conveyed. Areas where secondary eddies are expected are considered dead storage. Typical examples of such areas are the "flow shadows" behind embankments while flows go through contracted bridge openings. The ineffective flow areas were established using the generally accepted ration of flow direction to cross section direction of 1:1 for contraction and 3:1 for expansion.

Houses were modeled as blocked obstructions and the corresponding areas where the flows are contracting and expanding around these buildings are shown as ineffective areas.

3.1.6 STRUCTURAL DETAILS

The modeled reach of Hudson Branch includes three waterway structures, included in the existing and proposed conditions model. They are described individually below:

Nearly 550 feet downstream of the upstream model limits, Hudson Branch passes underneath a private residence spanning the channel. The structure is supported by a stone wall and spans the channel by 33 feet and is 18 feet wide. The structure is located between sections 31 and 32 in the hydraulic model.

The next structure is located 1050 feet downstream of the upstream modeling limits. Hudson Branch crosses under Frederick Road through a 108" (9') CMP culvert. The culvert is 566 feet long and crosses under the roadway and private residences until it outfalls to the north of Frederick Road. The structure is located between sections 14 and 24 in the hydraulic model.

The third structure is located 50 feet upstream of the downstream modeling limits. A 96" CMP culvert 95 feet in length conveys Hudson Branch between a historic building and 8526 Main Street. The culvert provides access to the historic property to the north. The structure is located between sections 2 and 3 in the hydraulic model.

3.1.7 EXISTING CONDITIONS HEC-RAS

The HEC-RAS analysis for the existing condition was conducted for the mixed flow regime. The upstream reach limits begin immediately downstream of the Frederick Rd./Rogers Ave. intersection. The model begins at River Station 40. Hudson Branch runs parallel to Main Street to the south. The stream is fixed between the valley wall to the south and a segment of residential houses to the north along Main Street. The stream is conveyed under Main Street via a 108" CMP Culvert for 566 feet until it discharges to the north side of Main Street. The stream flows between the north valley wall and the row of homes along the north side of Main Street. There is a 96" CMP that conveys flow past a historic building in Ellicott City that is 95 feet in length.

Due to the length and orientation of the 108” culvert under Frederick Road, a separate model was used to determine the behavior of flow over the 108” culvert between sections 24 and 14. Modeling a culvert in HEC-RAS does not allow for analysis of varying topography over a structure. To simulate the conveyance of flow in the culvert, the maximum culvert flow capacity, as determined through HY-8 analysis, was removed from the upstream cross section (24) and reintroduced at the downstream end of the culvert (cross section 14). This method provides an idea of how much flow overtops the culvert and the water surface elevations of the 10- and 100-yr storm events for cross sections 23-15. A low flow model was created to analyze the culvert with traditional methods for the 2-year and 5-year storm events. The 2-year high flow model was compared to the 2-year low flow model to validate that adjusting the discharges in this location would provide a similar representation of the culvert behavior through sections 24 to 14. The water surface elevations upstream and downstream of the culvert had a nominal difference of 0.2’.

The 1-Dimensional Model shows backwatering behind the culvert under Frederick Road starting at the 2-year storm. Above the 2-year storm, the channel begins to flow out of bank onto Frederick Road. The 5-year event yields incipient roadway flooding, with flow staying in bank for most of the reach except for cross sections 27-25. Flows from the 5-yr storm do not overtop the 108” culvert. The 10-year event shows out of bank flow throughout the reach with the exception of the section between stations 36 to 31. All higher events show considerable roadway flooding for the 50 and 100 year events upstream of the Frederick Road Culvert. The roadway would continue to convey flow down the roadway to toward available inlets, but the 1-Dimensional model cannot provide an accurate representation of this flow pattern. At this location the analysis is more appropriately modeled by the TUFLOW 2-dimensional model which is described in section 3.2.

Floodplain maps and input data for the existing HEC-RAS model are attached in *Appendix D*.

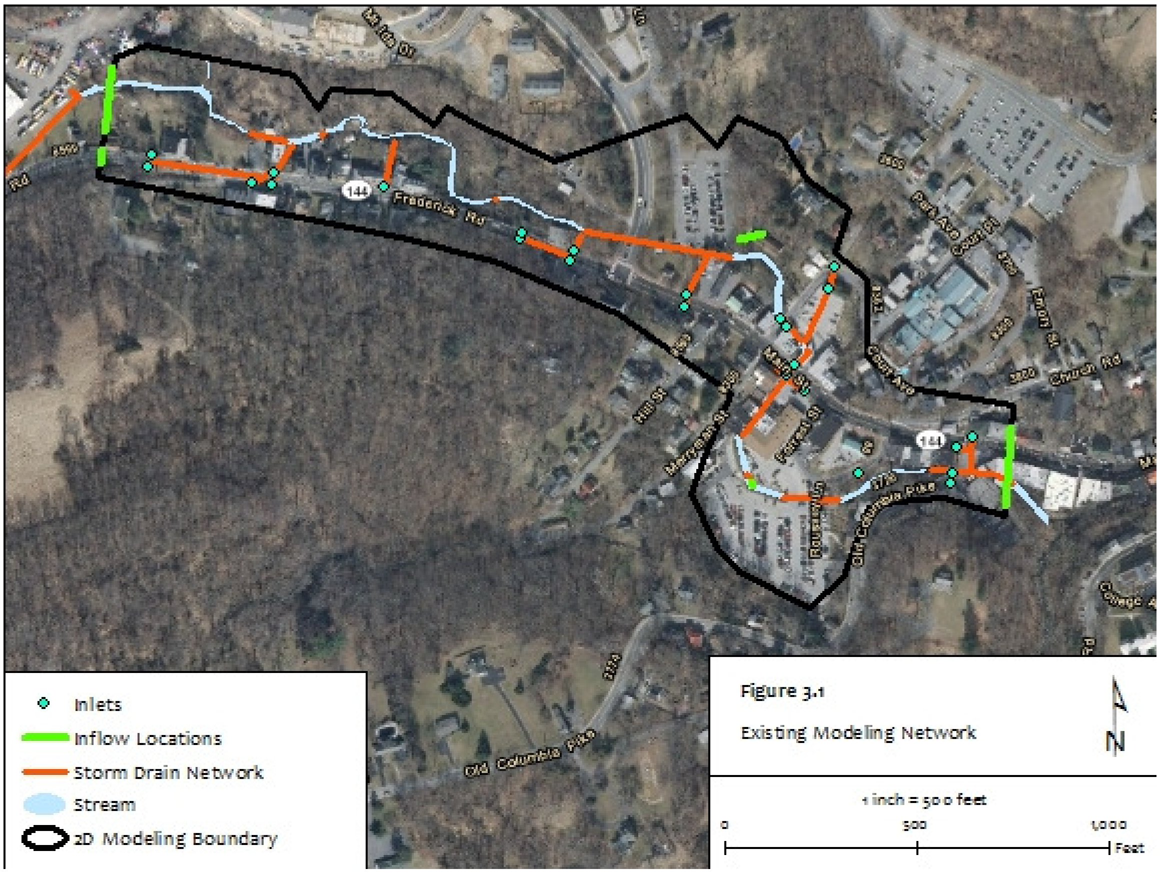
3.2 TUFLOW 2-DIMENSIONAL MODELING

2-Dimensional modeling using the TUFLOW flood simulation software was conducted in the southeast corner of the Hudson Branch drainage area, from approximately 8578 Frederick Rd. downstream to the intersection of Frederick Rd. and Old Columbia Pike. The TUFLOW simulation software provides computations for flood analysis using both 1-dimensional and 2-dimensional solutions. The complexity of the drainage network and topography of the downtown area necessitated the use of a 1D/2D simulation program, such as TUFLOW, to best represent flood conditions.

The TUFLOW simulation program requires several key inputs to drive the simulation computations (See Figure 3.1). Inputs into the TUFLOW model were generated using ArcGIS software to create spatially oriented data layers. These data layers were then “read” into text document command prompts, which initiated and provided the necessary data to drive the computation engine that is TUFLOW.

To represent the flow of water into the modeled region, it was necessary to define four different inflow hydrographs for each model scenario. Inflow hydrographs were generated using the TR-20 hydrologic model of the drainage area. The hydrographs at four different inflow locations were defined by specific cross sections in the TR-20 model.

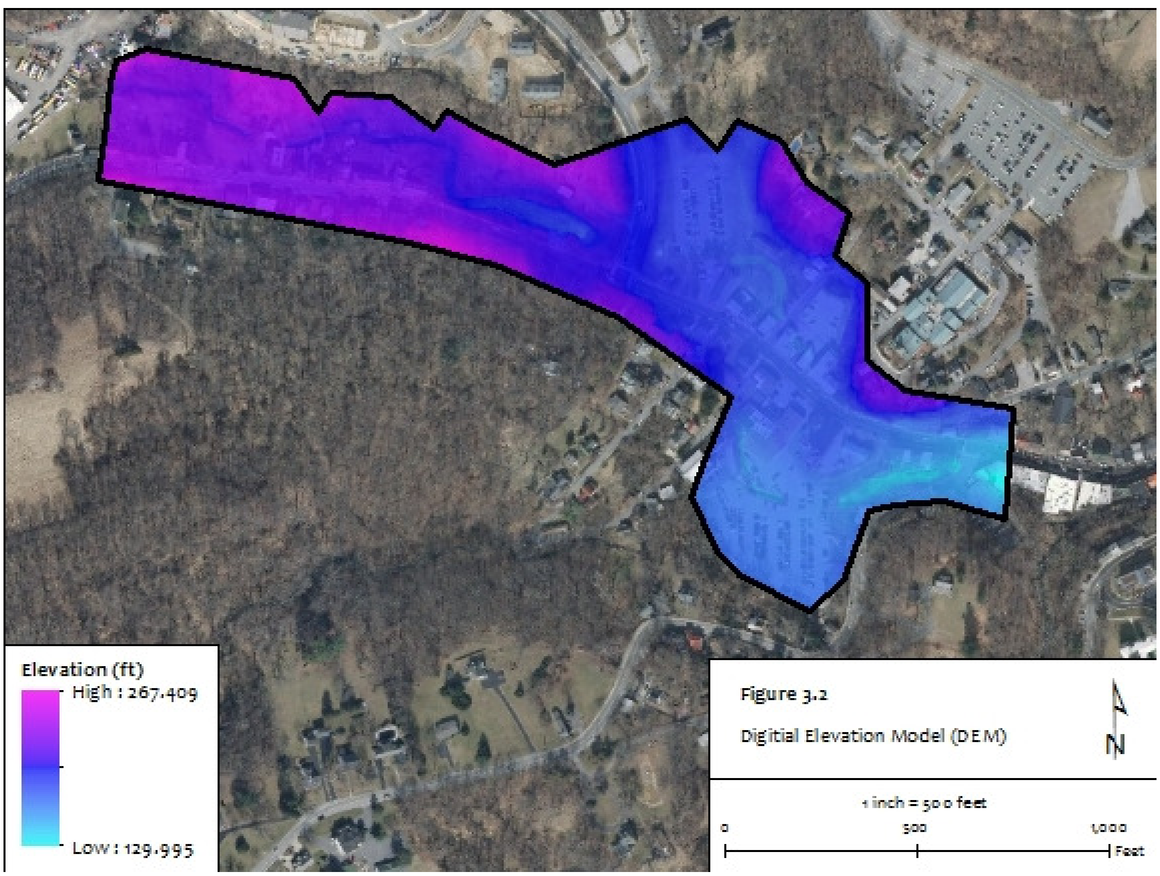
Figure 3.1: Schematic showing the key elements used to define the hydrologic and hydraulic characteristics of the TUFLOW simulations.



To reduce the simulation time for the TUFLOW models, the inflow hydrographs were abbreviated in duration to represent the majority of flood volume, while neglecting low flows at the beginning and end of the storm. The inflow hydrographs for the Tropical Storm Lee 2-D models begin at time equal to 9.9 hours and have a duration of 6.18 hours, replicating the flows from approximately 10am to 4pm on September 7, 2011. The standard storm events (5-, 10-, 50-, and 100-yr) were modeled with inflows beginning at time equal to 10.02 hours. Because these standard events experience the majority of flow in a slightly more prolonged hydrograph than the Tropical Storm Lee event, the models representing these storm events were run with an 8 hour inflow duration. See *Appendix E* for inflow hydrographs.

Another basic input requirement of TUFLOW models is topographic data. Topographic data for the 2-D modeling area was acquired through surveys which produced digital terrain models (DTM) and cross-section information. DTM data was provided for much of the area, however only surveyed cross-sections were available for Parking Lot 'D'. The surveyed cross-sections in Parking Lot 'D' were combined with GIS contour information to create a DTM that was merged with the surveyed DTM to provide a comprehensive topographic representation of the area. The merged DTM was then converted to DEM format to satisfy compatibility requirements of the TUFLOW program. The TUFLOW model then generated a 5 foot grid and, using the DEM data, assigned elevations to each grid. A grid size of 5 feet was chosen based on the size of the modeling region, the size of the stream channel, and the desired level modeling detail. The smaller the grid, the more detailed the topographic data; however, a smaller grid also presents issues such as long simulation times and greater flow instabilities. The 5 foot grid size yielded a reasonable simulation time of roughly 2.25 hours, while providing enough detail to sufficiently represent regional topography.

Figure 3.2: Digital elevation model (DEM) used to define topography of the TUFLOW simulations.



The TUFLOW simulations also required information detailing the inlet, culvert and bridge network inside the 2-D modeling region. A conveyance structure network describing these structures was embedded as a 1-D network inside the

2-D modeling region. Boundary conditions connecting the 1-D and 2-D areas completed the addition of these structures into the model.

Other various elements were added to the model to further describe the 2-D simulation region. As discussed in Section 3.3, several of these parameters were adjusted throughout the modeling process to better represent the anecdotal evidence of the flooding conditions resulting from Tropical Storm Lee. Once these parameters were finalized for each storm event, parameters were not changed, ensuring consistent comparisons between existing and proposed modeling scenarios.

The total flood simulation was run for a duration of 8 hours for the model representing the Tropical Storm Lee event, and 9 hours for models representing the 5-, 10-, 50-, and 100-year storm events. The length of each simulation was enough to calculate flood outputs for all significant flooding from each storm event. The real-world time required to run each model simulation was approximately 2.25 hours.

The outputs generated by the TUFLOW model require post-processing using command prompts to produce results that can be visually reviewed using ArcGIS. A variety of output results can be generated to view variables such as flow, velocity, and water level at various times and locations throughout the modeled region. *Appendix E* contains maps that show maximum flood depth and velocity vectors indicating flow direction and magnitude near the peak of storm flooding.

3.3 CALIBRATION OF MODEL VS. TS LEE EVENT

In order to assure that the model was depicting the depth and direction of flow through the terrain surrounding Main Street, anecdotal data was used as a point of comparison to the hydraulic model, which was run using a simulation of the Tropical Storm Lee event. The water surface elevations from that model were compared to measurements and visual indicators, and the model was adjusted as necessary in an attempt to recreate those conditions as closely as possible. This will assure, to the greatest extent possible given the available information and the resolution of the data, that the model will represent other typical storm events in a manner that would represent the actual flooding conditions during such a storm.

3.3.1 AVAILABLE DATA

Residents of Ellicott City have uploaded several personal videos to YouTube, many with time stamps that can be used to visually correlate the depth of water relative to existing structures within the study area such as buildings, curbs, channel crossings and the like. Among the videos used for this purpose:

- Approximate Address: 8672 Frederick Rd. (Looking South across Frederick Rd, east of Rogers Ave)
    - Evaluated depth of water along Frederick Rd, flow escaping channel

  - http://www.youtube.com/watch?v=WG5FTrTOVSI

- Approximate Address: 8390 Frederick Rd. (Looking Southeast across Frederick Rd.)
  - Evaluated floodplain extent, flow depth and velocity along Frederick Rd.
  - http://www.youtube.com/watch?feature=endscreen&NR=1&v=A5YdAoBQvws
- Approximate Address: 8342 Frederick Rd. (Looking North across Frederick Rd, at Merryman St.)
  - Evaluated location where flow entered Frederick Rd. and flow depth and velocity
  - http://www.youtube.com/watch?feature=endscreen&v=XdaaYIom30I&NR=1
- Approximate Address: 8309 Frederick Rd. (Looking South across Frederick Rd, near Ellicott Mills Brewing Co.)
  - Evaluated depth of water along Frederick Rd
  - http://www.youtube.com/watch?NR=1&v=fUih7aI8ZIY&feature=endscreen
- Approximate Address: 3731 Hamilton St. (Looking East across Parking Lot ‘D’)
  - Evaluated flood plain and depth of flow in Parking Lot ‘D’
  - http://www.youtube.com/watch?feature=endscreen&v=nPGCLV3m6FA&NR=1
- Approximate Address: 8203 Frederick Rd. (Looking South across Frederick Rd, across from intersection at Old Columbia Pike)
  - Evaluated depth and velocity of flow down Frederick Rd. (Main St)
  - http://www.youtube.com/watch?feature=endscreen&NR=1&v=GAx7_ADHUKc

A report prepared for Howard County, “Case Study: Valley Mede-Ellicott City Tropical Storm Lee Flood Event” was provided for this use as well, as it contains records of over seventy interviews with residents recounting their recollection of the event, including the depth and direction of flood waters on their property. This report was particularly useful in confirming where the flow came out of the channel onto roads and properties, and where the flow was redirected around structures.

The rainfall data discussed in Section 2.3.3 above was used to create a synthetic, dimensionless rainfall table in TR-20, which generated the hydrographs used by the hydraulic model to recreate this flooding event. The synthetic rainfall table generated for the Tropical Storm Lee event had a cumulative rain depth of 8.11 inches and a duration of 24 hours.

3.3.2 CORRELATION WITH MODELS

The TUFLOW simulation model was compared to anecdotal evidence from the Tropical Storm Lee event, using generated outputs showing the extent of the floodplain, maximum depth of flooding, and velocity (direction, magnitude) of flow. The timing of the flooding was also examined. Generally speaking, the results of the calibration models correlated well with the anecdotal data, within the expected tolerances for this type of work.

A perfect match between simulated outputs and anecdotal evidence provided in the case study or found in online videos was not anticipated due to the precision of both the model resolution and anecdotal evidence, but the simulations were expected to yield results that generally represented the behavior of the flooding. Because topography within the models was represented with an interpolated 5 foot grid, locations with steep banks or severe topographic changes were not expected to simulate flood depths that matched precisely with anecdotal evidence. Model tolerance related to depth of flooding was also high because of potential conflict/error associated with personal accounts and non-scientific evidence of flood depths. Instead, model performance based on flooding behavior was largely evaluated by comparing simulated and real-world evidence through the overall extent of flooding and direction of flow paths.

The overall maximum floodplain was evaluated first to determine if modeled flooding occurred in the same locations shown in anecdotal evidence. Next, flow depths, directions, and velocities were compared. For initial modeling iterations, model characteristics augmented to calibrate the model included material roughness, 1-D culvert form loss coefficients, model topography, and 1D/2D boundary conditions.

The upstream portion of the TUFLOW modeling area (approximately 8578 Frederick Road to 8500 Frederick Road) contained a significant amount of flooding along the stream and in the roadway. Initial model runs injected the upstream flow hydrograph across the stream at the culvert exit north of the dwelling at 8578 Frederick Road. This resulted in minimal flows reaching the roadway, which was not representative of anecdotal evidence, which suggested significant flooding along the road between 1’ and 3’ in depth. To better model flooding along the roadway, a portion of the inflow hydrograph was injected along the top of the model region along Frederick Rd. for high flow events. The flow along the roadway was approximated to be 1/3 of the total flow overtopping the culvert, while the other 2/3 of flow overtopping the culvert were assumed to enter the 2-D domain in the grass areas along the south side of the stream. Proportioning the inflow hydrographs resulted in better simulation of flooding behavior along the roadway. The upstream portion of the model was also augmented through increasing stream roughness along the stream bends to better simulate backwatering effects seen north of the building at 8560 Frederick Rd. Many buildings in this area experienced significant flooding. The depth of basement flooding in these buildings was slightly variable relative to the flood depths simulated by the model; however, the relative magnitude of the simulated flood depths along various houses correlates fairly well with magnitudes of flooding described by homeowners cited in the 2011 Valley Mede-Ellicott City case study. The simulation model also shows higher flow velocities through the buildings that experienced the most significant basement flooding. Buildings on the eastern end of this area (approximately 8500 Frederick Rd), experienced flood waters escaping the north side of the roadway, flowing back into the stream. The model also simulates flows leaving the roadway and reentering the stream in this location.

The middle portion of the 2-D modeling area did not experience significant roadway flooding during the Tropical Storm Lee storm event; the simulation model was concurrent with this anecdotal evidence. Instead, this area experienced very high flow velocities and flow rates within the stream channel.

The downstream portion of the 2-D modeling area (approx. 8400 Frederick Rd. through Parking Lot ‘D’) experienced significant roadway and residential flooding. Anecdotal evidence from the case study and from online videos suggests that flow reentered Frederick Rd. at approximately 8398 Frederick Rd. (immediately west of Court Ave.); reentry of flow onto Frederick Rd. was also simulated at this location with the TUFLOW model. Anecdotal evidence of significant flooding of the dwellings from 8390 to 8398 Frederick Rd. was simulated in the model through substantial flood depths and flow vectors indicating an eddy that pushed flows into these dwellings.

In initial model runs, simulated flow vectors in this area ran through the building north of Parking Lot ‘D’. Anecdotal results did not suggest extreme flooding in this building, thus the model topography was adjusted to increase the surface elevation for the building footprint, resulting in more accurate flow patterns in which flows were directed down Frederick Rd. and Hamilton St.

To better simulate flow depths along the roadway and in parking lots in this area, structure form losses and stream roughness were increased in several locations. The simulated extent of flooding in Parking Lot ‘D’ appeared consistent with flooded areas shown in videos online.

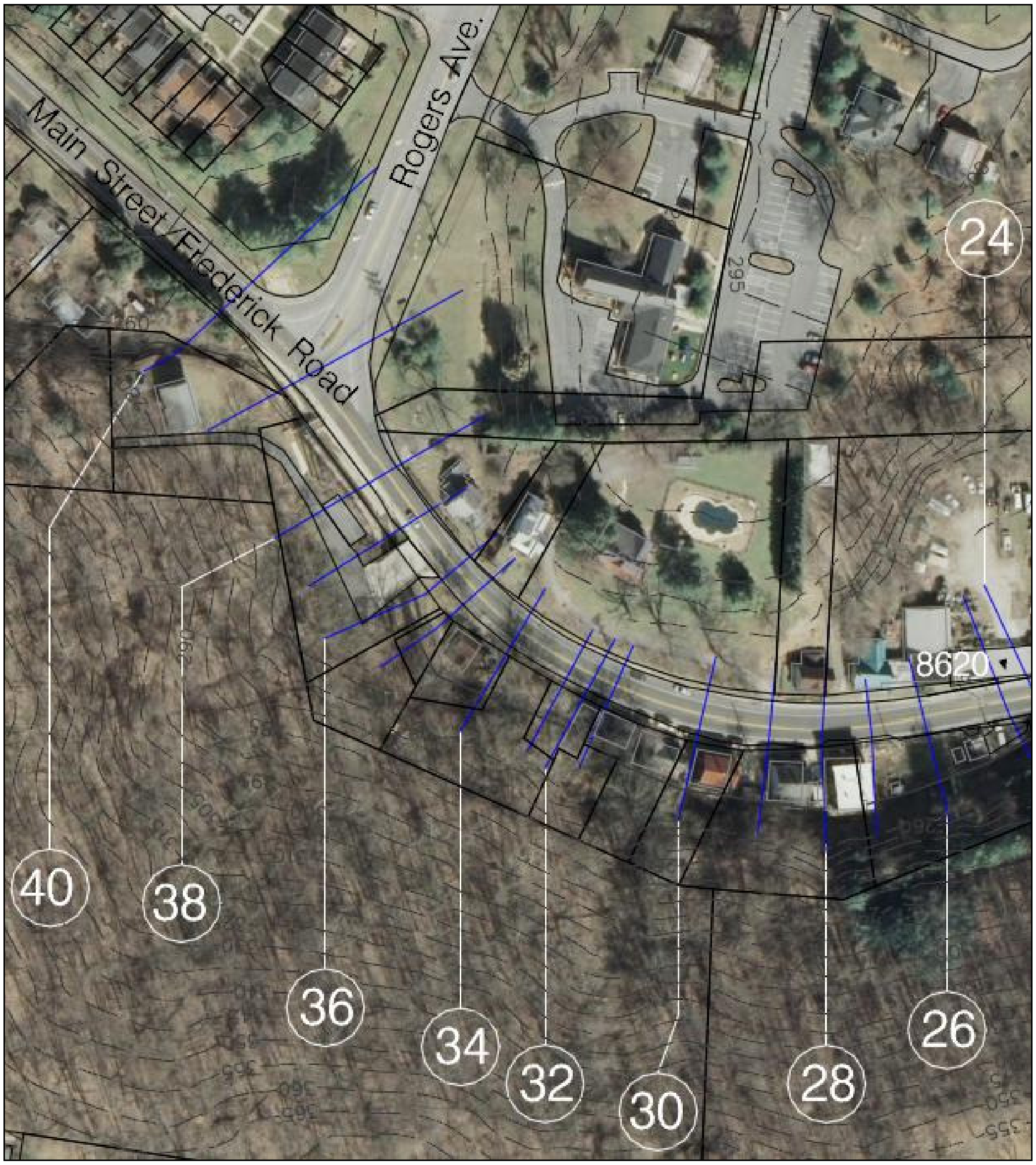
Another indicator of model performance in this area was flow velocity down Frederick Rd. Simulated flow down Frederick Rd. was approximately 20 ft/s in most areas. When compared to debris seen floating down the roadway in online videos, the actual flow velocity experienced during the storm appears to be very close to the simulated value.

3.4 EXISTING CONDITIONS RESULTS

The results of the existing model simulations were evaluated through extent of flooding, flow depth, and flow velocity (magnitude and direction for 2-D model). The top portion of the hydraulic modeling region (from Rogers Ave./Frederick Rd. intersection to 8578 Frederick Rd.) was modeled with HEC-RAS for the 2-, 5-, 10-, and 100-year storm events, while the lower region (from 8578 Frederick Rd. to the Frederick Rd./Old Columbia Pike intersection) was modeled with the 2-dimensional TUFLOW simulation program. A portion of the modeled region was modeled both with 1-D and 2-D models. 40 cross sections provide reference points for evaluating water surface elevations of the HEC-RAS output, while the TUFLOW results can be evaluated by examining maps showing extent of flooding, flow depth, and flow velocity. Also, six representative Cross Sections were placed perpendicular to flow within the 2-D modeling region to explicitly examine flow depths between different model scenarios. For discussion purposes, the behavior of flooding under the various modeling scenarios is broken out into 8 different areas. Results of the 1-dimensional and 2-dimensional models are in *Appendices D* and *E*, respectively.

Area 1 – 1-D Model (Frederick Rd./Rogers Ave. intersection to Culvert at approx. 8620 Frederick Rd.)

Figure 3.3: Location and Cross Section Map of Area 1.



The HEC-RAS model showed the 2-yr storm event in this area was contained within the channel and the 5-yr storm event showed incipient roadway flooding. The 10-yr and 100-yr storms escaped the channel in several locations. The 5-yr storm stayed within the channel from cross section 40 down to cross section 27, where backwatering from the 108" culvert caused flow to escape onto the roadway, although flow did not overtop the culvert at cross section 24, which was immediately above the culvert.

The 10-yr storm stays within the stream banks for the majority of the reach but enters the roadway beginning at cross section 27. Flow escaping the channel for the 10-yr storm was likely the result of lower topography, reduced channel slope, and/or backwatering from the 108” culvert, which experienced overtopping to a depth of approximately 0.20 ft. The 10-yr flow was sufficiently low between cross sections 32 and 31 that the house spanning the stream at this location did not encourage flow to escape the channel. The 100-yr storm flowed over the channel banks and onto the roadway for the majority of this region. The 100-yr storm experienced backwatering effects from the dwelling spanning the roadway between cross sections 32 and 31 and from the downstream culvert. The most significant roadway flooding for the 100-yr event occurred at the downstream cross sections 28-24, where depth over the roadway reached a maximum height of approximately 5.02 ft.

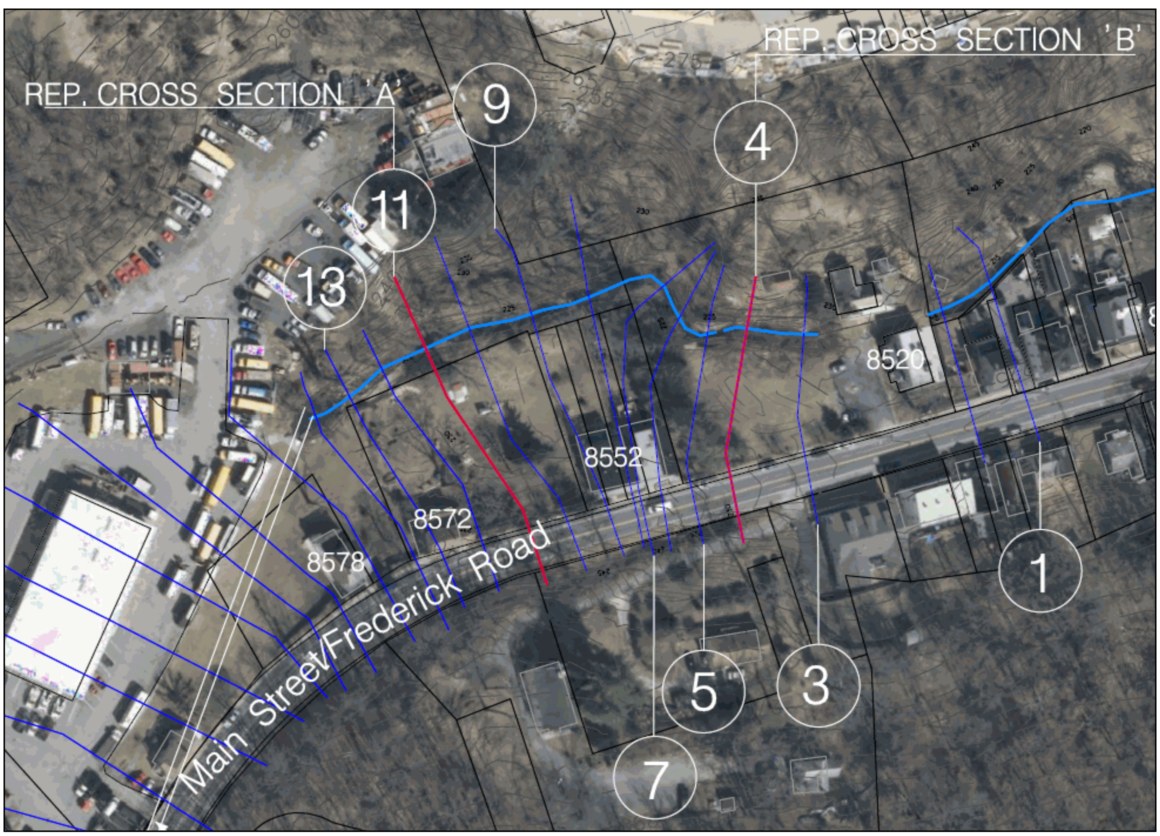
Area 2 – 1-D Model (Culvert at approx. 8620 Frederick Rd. to Rep. XS A)

Figure 3.4: Location and Cross Section Map of Area 2.

The second, "highflows" HEC-RAS model was used to simulate water surface elevations along the roadway and parking lot area above the 108" culvert. This model had slightly lower water surface elevations at the head of the culvert (cross section 24) than the "lowflows" HEC-RAS model because it does not take into account backwatering from the culvert. The 5-yr storm was modeled as having a minimal flow going down the roadway and resulted in ponding in low areas of the parking lot north of Frederick Rd. Existing storm drains in this parking lot would likely be able to remove much of this 5-yr flow and reduce ponding if they are functioning properly. The 10- and 100-yr storms experienced significantly greater flow overtopping the culvert and ponding in the parking lot, with the 100-yr water surface reaching the buildings at cross sections 24, 22, 20, and 19. Simulated flooding in the parking lot reached a maximum depth of approximately 6.36 ft for the 100-yr storm at cross section 22. Flow escaped the parking lot area and roadway by flowing through a depression northwest of the dwelling at 3548 Frederick Rd.

Area 3 – 1-D/2-D Model (Rep.XS A to Rep. XS B)

Figure 3.5: Location and Cross Section Map of Area 3.



Flooding from the 10-yr storm in this area was roughly 1 ft. deep for most of the floodplain around representative Cross Section 'A' (same location as HEC-RAS cross section 11), as simulated with both HEC-RAS and TUFLOW. The 10-yr water surface in HEC-RAS was simulated to be 230.16 ft. which was very similar to water surfaces simulated with TUFLOW. However, the TUFLOW model showed approximately 0.5 ft. of water flowing down the roadway after overtopping the 108” culvert upstream. TUFLOW velocity vectors showed flow escaping the roadway between the dwellings at 8578 Frederick Rd. and 8572 Frederick Rd. Significant flows escaping the roadway immediately east of the dwelling at 8552 Frederick Rd. ran into a grass berm protecting a parking lot to the east, causing some flow to continue down the roadway and some to reenter the channel at the approximate location of representative Cross Section ‘B’ (HEC-RAS cross section 4).

The simulated 100-yr storm inundated nearly this entire area; from the roadway across to the north overbank of the stream. HEC-RAS of the stream simulated flooding to an elevation of 232.67 ft. The water surface elevations simulated with TUFLOW were, in general, slightly higher than the HEC-RAS elevations, with an average of approximately 233 ft at representative Cross Section ‘A’. Flow along the roadway simulated with TUFLOW was approximately 1 ft for most of the area, but water depths in the floodplain surrounding the dwelling at 8552 Frederick Rd. approached 5 ft. Flow vectors from the 100-yr event suggested significant flow leaving the roadway between 8572 and 8552 Frederick Rd. and reentering the roadway between 8552 Frederick Rd. and the grass berm to the east.

The 50-yr storm exhibited similar flood behavior as the 100-yr storm in this area, with slightly lower water surface elevations. The 2-yr storm, as simulated with HEC-RAS, did not escape the stream channel and was sufficiently conveyed by the 96” culvert immediately downstream from representative Cross Section ‘B’. The results of the existing HEC-RAS simulations suggest that the 2-year storm event was generally contained within the channel and thus was not simulated with the TUFLOW modeling software.

Area 4 – 1-D/2-D Model (Rep. XS B to Rep. XS C)

Figure 3.6: Location and Cross Section Map of Area 4.

Backwatering from the 96" culvert running below the parking lot in front of 8520 Frederick Rd. pushed simulated flood waters onto Frederick Rd. for the 5-, 10-, 50-, and 100-year storms. This is evidenced by the water surface elevations generated with HEC-RAS and TUFLOW. HEC-RAS roadway water depths at representative Cross Section 'B' (HEC-RAS cross section 4) ranged from 0.77 ft for the 50-yr storm to 1.4 ft for the 100-yr storm. TUFLOW water depths along the roadway were lower (0.5-1ft) but increase slightly further to the east.

The 2- and 5-yr storm events were sufficiently conveyed by the 96" culvert and did not overtop the stream banks in the HEC-RAS simulation.

Between representative Cross Sections 'B' and 'C', flow left the roadway through driveways and reentered the stream channel, with flow persisting further down Frederick Rd. for larger storm events. Roadway flows from the 10-yr event reentered the channel behind 8490 Frederick Rd., while roadway flows for the 50-yr reentered the channel behind 8472 Frederick Rd. and flows from the 100-yr storm never entirely left the roadway.

From 8472 Frederick Rd. to 8490 Frederick Rd., flooding from the 50- and 100-year storms is likely due to backwatering from the downstream channel meanders as well as from floodwaters leaving the roadway.

Area 5 – 2-D Model (Rep. XS C to Rep. XS D)

Figure 3.7: Location and Cross Section Map of Area 5.

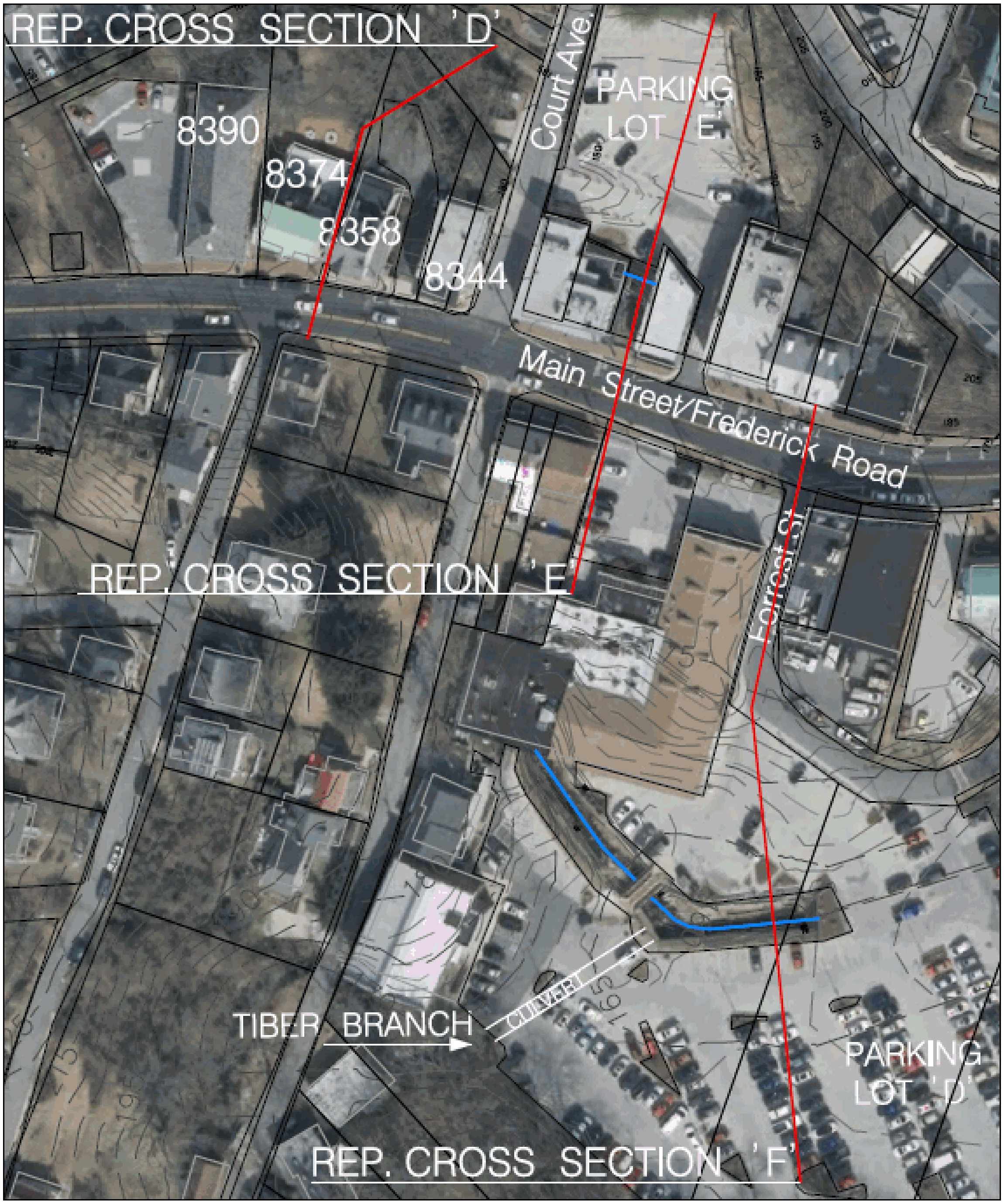
This area of the model experienced very little flooding for the 10-yr event, with the only flooding occurring at the downstream end near representative Cross Section 'D'. Flooding from the 50- and 100-year events was significant; both flood simulations suggested that the culvert running beneath Ellicott Mills Drive would backwater enough to force water onto Ellicott Mills Dr., resulting in shallow inundation of the parking lot and dwellings north of Frederick Rd., between Ellicott Mills Dr. and Court Ave.

The worst flooding in this region was simulated just upstream of representative Cross Section 'D', behind the dwellings from 8374 to 8390 Frederick Rd. High volume flows from the culvert running beneath Ellicott Mills Dr. meet with substantial flows from a ditch east of Parking Lot F; combined backwatering effects from the structure beneath Court Ave. and the low topography between the dwellings and the stream, resulted in a significant eddy, or area in which flow circulates in the opposite direction of the primary flow, that extends along the south side of the stream from 8390 Frederick Rd. to the driveway just West of Court Ave. in Area 6.

Water surface elevations across representative Cross Section 'D' were relatively consistent, resulting in maximum flood depths of 8.3 ft. in the floodplain and 1.3 ft on the roadway for the 100-yr event. The 10-yr event did not inundate the Frederick Rd. at the location of representative Cross Section 'D', but did enter the roadway further downstream. The maximum floodplain depth of the 10-yr event was approximately 6.1 ft. The 50-yr storm was between the 10- and 100-yr events in terms of flood depth and simulated flood velocities; flooding from this event did inundate the roadway at representative Cross Section 'D'.

Area 6 – 2-D Model (Rep. XS D to Frederick Rd./Old Columbia Pike Intersection)

Figure 3.8: Location and Cross Section Map of Area 6.



This area of the model experienced significant flooding of dwellings west of Court Ave. as well as flooding along Frederick Rd. and in local parking lots, for all three simulated storm events. The backwatering and eddy that was simulated west of Court Ave. entered Frederick Rd. along the driveway between 8358 and 8344

Frederick Rd. This overflow onto Frederick Rd. was simulated for all three storm events.

A lower flowrate of overflows entered Frederick Rd. from the driveway adjacent to Ellicott Mills Brewing Company. Flooding from this area originated at the open stream section at the south end of Parking Lot ‘E’.

Flood waters from these areas continued down Frederick Road to the end of the modeled region. The 50- and 100-yr storms simulated significant flowrates down the roadway. The depths of flooding along Frederick Rd. was greatest between representative Cross Sections ‘E’ and ‘F’, and decreased as velocity down the roadway increased towards the intersection of Frederick Rd. and Old Columbia Pike. 100-yr roadway depth along representative Cross Section ‘E’ was approximately 4.1 ft and velocities between representative Cross Section ‘F’ and the intersection with Old Columbia Pike approached 35 ft/s; these flows were significantly less for the 10-yr storm, with a respective average roadway depth of 1.6 ft and velocities approaching 20 ft/s.

Significant flooding of Parking Lot ‘D’ was simulated for all three storm events. Flood waters in the parking lot had multiple origins depending on the storm event. For the 10-yr storm, flood waters originated almost entirely from the open stream section running through the parking lot, with some minimal flows coming down Forrest St. from Frederick Rd. The 50- and 100-yr events simulated flood waters entering Parking Lot ‘D’ from the open stream section, from Forrest St, and from overtop the culvert that confluences the Tiber Branch with Hudson Branch near the footbridge. Flooding from this open stream section is likely the result of backwatering from the footbridge and downstream culvert, as well as from the low channel depth (high bedrock depth) relative to the parking lot.

The extent of flooding in Parking Lot ‘D’ for the 50- and 100-yr events threatens the building at the northwest corner of the lot with a turbulent back eddy, while low velocity but high water surface elevations threaten several buildings at the east end of the lot. Flood depths along representative Cross Section ‘F’ vary greatly because of varying topography, significant elevation differences and differing flow paths. The most stable area for depth of flooding was in the overbank north of the open stream section, downstream from the footbridge. Flood depth in this location was 1.9 feet for the 10-yr model and 3.5 ft for the 100-yr model.

4.0 CONCEPTUAL IMPROVEMENTS

The study focused on two main types of conceptual improvements, stormwater quantity management to reduce the quantity of flow into the Main Street corridor, and conveyance improvements that would upgrade or supplement the storm drains and channels through the flooded area to carry more water at a lower elevation for a given event. Though there are a number of smaller stormwater improvements that could be implemented, the scope of this study was limited to the largest feasible sites that could have the most significant impact on the quantity of flow, as well as sites within public rights-of-way. The structure of the model created for this study allows for any variation on, or combination of, improvements to be run through the model at a later date, however for the sake of keeping the large amount of data manageable, the focus of this study will include 3 improvement iterations: SWM Only, Conveyance Only, and All Improvements

4.1 DEVELOPMENT OF SWM SITES

The challenges in locating new sites to provide significant quantity management were numerous. Much of the watershed is built out with residential and commercial development, with the exception of some wooded areas on the periphery of the watershed. These areas are not suitable as they are in steep terrain, would involve significant tree loss, and most importantly do not receive much if any runoff from developed areas due to their upland location.

The most promising locations for storing and managing a significant volume of runoff were the areas within the US 40 / US 29 interchange, which are owned by Maryland State Highway Administration (MSHA). These areas are not currently utilized by MSHA for stormwater management, presumably because the interchange was built prior to the SWM era. The grading of the proposed facilities is conceptual and does not account for potential geotechnical or regulatory constraints such as the presence of bedrock and limitations imposed by MSHA (the property owner) or other regulatory agencies. Three areas were examined for their potential improvement:

SWM Area 1 – This is the northeast loop of the interchange and is online with the main channel that carries DA 1 and a portion of DA 2 under US 40 to the south. As a result, any management applied in this location will attenuate the flow from nearly the whole northern portion of the watershed (North of US 40) making it the most effective of all the sites. The storage would be created by excavating most of the area inside the loop down to near the elevation of the existing channel. Though online ponds are typically not encouraged by Maryland permitting agencies, exceptions can be made for specific circumstances such as this, particularly in light of the fact that fish passage does not currently exist at this location due to a 3’ drop in a concrete structure at the entrance to the culvert under US 40. Because the pond storage created is in cut relative to surrounding areas, and outfalls into a storm drain system that does not daylight for over 900’ from the pond, it would most likely not require any additional seepage control (Code 378 exempt).

SWM Area 2 – This area is in the lower half of the southeast interchange loop and collects runoff within DA 2 from a portion of US 40 and its ramps, as well as an unmanaged commercial area just to the east. The outfall spillway pipe, currently a culvert under the loop ramp to the south, would require retrofitting for seepage control in compliance with Code 378, which could be achieved for the existing ramp embankment with a clay liner on the upstream face to supplement the pipe replacement. The stage-discharge table is based on maintaining groundwater baseflow and maximizing storage / attenuation while maintaining over 2’ of freeboard for the 100-year event.

SWM Area 3 – This area is in the over-widened median of US 29 in the southern portion of the interchange and receives runoff from the eastern portion of DA 3 including the currently managed areas in Ellicott Center, as well as portions of unmanaged commercial development and US 29 ramps. The outfall spillway pipe, currently a culvert under US 29 SB, would require retrofitting for seepage control in compliance with Code 378, which could be achieved for the existing roadway with a clay liner on the upstream face to supplement the pipe replacement. Alternately, a weir structure upstream of the existing US 29 culvert may allow for the culvert to remain as a non-378 spillway pipe in lieu of a pipe replacement under the roadway. Stage-discharge was developed under same principle as above.

An additional SWM area along US 40 WB, west of US 29 was initially investigated as a location to treat runoff from some of the western portion of DA 3, however it was discovered that this area is currently under development and not publically owned, therefore it was removed from further consideration

4.1.1 ANALYSIS OF THE EFFECTS OF PROPOSED CWP SWM IMPROVEMENTS

As part of the overall analysis, the County provided a map prepared by the Center for Watershed Protection of potential SWM LID retrofit site locations within the area and requested that the potential impact of these proposed facilities on flooding-related runoff be included. Without additional information regarding the specific design or drainage area of these BMPs two assumptions were made: Sites would treat the first 1” of runoff back to “woods in good condition” per Environmental Site Design (ESD) criteria. Drainage areas were based on the most likely location of the actual BMP relative to existing roads and structures in the vicinity of the point shown.

The initial consideration of these sites was to see if the impact on runoff was significant enough to include in the overall analysis relative to the precision and error inherent within the model. A Curve Number (CN) reduction to “Woods – Good” was made for the presumed drainage area to each site and that was factored into the overall weighted CN for each DA and compared to the original to determine the effect of overall peak flow quantities. If the site locations fell within an area where existing SWM existed and was being modeled by CN reduction as discussed in Section 2.3 above, then this reduction was not made, since it had already been considered in existing conditions. Since the study includes storm events above the 1” runoff event considered for ESD design, the MDE methodology for Relative Curve Number (RCN) adjustment for determining the effect of ESD on higher storm events was used. For the sites in question, the change in CN for the 2-year event becomes numerically insignificant (<1%) for 7 of the 10 sites analyzed, with the largest change of 2.3% for a facility in DA 7.

Table 4.1 – Changed Runoff Curve Numbers for Proposed CWP Facilities

Subarea	Drainage Area	Original CN	CN w/ CWP Facilities				% change**
			2-yr	10-yr	50-yr	100-yr	
1	2	80.559	80.558				-0.001%
	3	75.926	75.925				-0.001%
2	1	88.594	87.960				-0.716%
3	4	82.378	82.079	82.147	82.178	82.196	-0.363%
	7	86.132	85.339	85.433	85.485	85.549	-0.921%
4	3	79.166	78.689				-0.603%
6	2	80.006	78.695				-1.639%
	3	79.468	79.383				-0.107%
	5	66.708	65.497				-1.815%
7	4	72.091	70.444				-2.285%

**% Change between the original CN and CN w/CWP Facilities for the 2-yr storm.

Since the RCN adjustment decreases for the higher (>2-year) storm events considered in this study, and the impact for even the most significantly changed sub-areas was a matter of a few cfs for the 2-year event, it was determined that the impact of these conceptual proposed ESD sites was not significant enough to show a change in water surface elevations within the models, and was not pursued in greater detail within this study. It is noted that, despite the negligible impact on larger flooding events, these potential facilities still have value relative to their collective positive impact on water quality in the Patapsco watershed during more frequent storm events.

Table 4.2 – Peak Discharges with and without Proposed CWP Facilities

Return Period (years)	Peak Flow (cfs) Entire Drainage Area, no CWP Facilities	Peak Flow (cfs) Entire Drainage Area, w/CWP Facilites	Peak Flow (cfs) Subarea 3, no CWP Facilities	Peak Flow (cfs) Subarea 3, w/CWP Facilities
2	535	530	242	240
10	1356	--	568	567
50	2647	--	1074	1072
100	3549	--	1331	1329

4.2 DEVELOPMENT OF ADDITIONAL CONVEYANCE SITES

In addition to examining alternatives to reduce the quantity of water to the Main Street corridor, the possibility of providing increased runoff conveyance capacity, in the form of additional storm drains and channel widening where feasible, was examined. These alternatives, numbered 4-7 sequentially after the 3 SWM alternatives, and from upstream to downstream, are described below (See *Appendix C* for storm drain layout maps):

Alternate 4 Storm Drain – This alternate consists of a 48" concrete storm drain trunk line that intercepts the runoff from the Rogers Ave. storm drain (the northern, developed portion of DA 6) and conveys this flow eastward separate from the Hudson Branch flow (DAs 1-5) running roughly parallel to the channel and culvert system currently carrying Hudson Branch, and outfalling at the existing culvert outfall location at the east end of the West End property into an open channel behind the adjacent residential properties (8578, 8572 Frederick Rd). This option would also involve abandoning the existing cross culvert that connects the Rogers Ave flow to the channel in current conditions. A flow splitter was considered here to balance the flow between the two systems, but the tailwater from the culvert and channel made the new proposed system largely ineffective at its upstream point for higher flows, so the proposed model keeps the systems separate.

The sizing of the pipe is based on tying in to the existing Rogers Ave system invert with adequate pipe cover, as well as what is reasonably feasible for construction given issues like trench width and depth while maintaining traffic as well as likely utility conflicts. The intent of this alternate is to reduce the frequency at which overtopping of channel flow from the south side onto Main Street will occur just downstream of Rogers Ave.

Alternate 5 Storm Drain – The location of the upstream entrance to this system is based on supplementing conveyance where the open channel flow goes back into a closed pipe system again, in this case the culvert between the structures at 8520 Frederick Rd. The storm drain will capture a portion of this channel flow and divert it back to the roadway, running parallel with the road before outfalling back into the channel at the point where the channel curves south then east to be immediately adjacent to the road. This location was selected because it is the point where the existing condition roadway flow that escaped from the channel upstream enters back into the channel, and can be adequately conveyed by the existing channel. The concept pipe sizing is based on similar constraints as described in Alternate 4, above. There are some local storm drain tie in issues associated with this alternate as well that would be examined during the detail design phase if this alternate is pursued.

Alternate 6 Storm Drain – The location of the upstream end of this system was selected to provide additional conveyance just upstream of the constrictions associated with the flow under Court Ave, the Ellicott Mills Brewing Company and the downstream conveyance under La Palapa Restaurant. The storm drain will capture a portion of the channel flow upstream of Court Ave and carry it south, under the driveway between 8344 and 8358 Frederick Rd., briefly east along Frederick Rd., south again down Merryman St. then east just behind La Palapa where it will outfall into the existing channel, recombining with the flow from the existing system. The concept pipe sizing is based on similar constraints as described in Alternate 4, above.

Alternate 7 Channel/Structure Modifications – For the final alternate, the channel through Parking Lot 'D' which carries the flow downstream of the confluence with Tiber Branch, the dimensions of this channel were modified to include a layback of the currently vertical slopes at a 3:1 cross slope. Also the structure that carries the flow beneath the northeast portion of the lot was raised by 2 feet to accommodate more flow. There are many permutations of widening and structure modifications, with varying impacts to the parking lot, that could be examined here; the one chosen was a typical iteration intended to examine whether or not such modifications had a significant impact on the tailwater and water surface of the upstream channel and systems along Main Street.

4.3 MODELING OF IMPROVEMENTS

4.3.1 SWM IMPROVEMENTS

The SWM improvement alternates were modeled by developing a preliminary pond grading of each area, setting a weir elevation for flow above a base flow amount that would carry the 100-year storm with adequate (2'+) freeboard for overtopping at the lowest point, and calculating a stage-storage-discharge table to be inserted into the existing condition TR-20 model at the proper location. The proposed condition was modeled in TR-20 with all 3 alternates in place at once, and the resulting downstream hydrographs were used in the hydraulic model as a comparison against the baseline conditions.

4.3.2 CONVEYANCE IMPROVEMENTS

The conveyance improvements were modeled differently for the HEC-RAS and TUFLOW models. For the HEC-RAS model, Concept 4 was included by reducing the inflow at cross section 37 by 60 cfs and then adding 60 cfs back into the model at the exit of culvert 4 at cross section 14. This flowrate was removed as it was calculated that 60 cfs was the approximate maximum capacity of the Concept 4 pipe given the existing constraints. A similar approach was taken for Concept 5, which diverts flow from the river at cross section 2. The flowrate removed from cross section 2 was determined by cross-referencing the water surface elevations from the existing model with the total head listed in the storm drain hydraulic design table (*Appendix C*). Following this methodology, flowrates of 100, 120, and 150 cfs were removed from cross section 2 for the 2-, 10-, and 100-yr storm events, respectively.

For the TUFLOW conveyance model, new culverts were added to the 1-D culvert network to represent concepts 5 and 6. Concept 7 was represented by generating a new topographic layer to augment the grading of the stream bank to a 3:1 slope. The culvert through Parking Lot 'E' was raised 2 ft by changing the existing culvert characteristics to reflect the new culvert dimensions. The hydrographs from the existing conditions hydrologic models were run through the proposed conditions models as a comparison against the baseline conditions.

4.3.3 COMBINED IMPROVEMENTS

For this iteration, the proposed hydrology with the 3 SWM alternatives was run through the proposed conditions hydraulic model with the 4 conveyance improvements to determine the combined effect of all concept improvements on water surface elevations

4.4 MODELING RESULTS OF PROPOSED IMPROVEMENTS

Changes to water surface elevations between the 2-, 5-, 10-, and 100-yr storm events in the 1-D modeling region are displayed on cross sections in *Appendix D.* Floodplain depth/extent and velocity maps of the existing and proposed conditions are in *Appendix E*.

4.4.1 RESULTS OF SWM IMPROVEMENTS

The proposed SWM improvements significantly reduced peak flows into the modeled watershed region (Table 4.3).

Table 4.3 – TR-20 Simulated Peak Flowrate to Watershed Outlet for Existing Conditions and the Proposed Stormwater Management Concept

Storm Event	*Peak Flowrate (cfs)*		*Percent Change*
	Existing Conditions	*Proposed SWM Concept*	
2-yr	535	460	-14.0%
10-yr	1356	1099	-19.0%
Tropical Storm Lee	2122	1800	-15.2%
50-yr	2647	2167	-18.1%
100-yr	3549	2740	-22.8%

The reduced flowrates under the proposed scenario resulted in decreased water surface elevations, flow velocities and the extent of the floodplain; the magnitude of the changes to these variables is dependent on the unique topographic features at any specific cross section in the modeled area. *It is important to note that percent peak flowrate reductions do not necessarily represent equivalent reductions in water surface elevation, flow velocity, or flood extent*.

Another metric used to evaluate impact of the proposed improvements was the number of buildings within the floodplain (Table 4.4). All buildings within the 2-D modeling boundary (approximately 8578 Frederick Rd. to the intersection of Frederick Rd. and Old Columbia Pike) that were touched by the floodplain were quantified for existing conditions and the proposed stormwater management concept. This comparison was only conducted for storm events evaluated with the 2-dimensional model.

Table 4.4 – Number of Buildings within the Floodplain under Existing Conditions and the Proposed Stormwater Management Concept

Storm Event	*Number of Buildings in Floodplain*		*Change*
	Existing Conditions	*Proposed SWM Concept*	
10-yr	40	39	-1
Tropical Storm Lee	47	45	-2
50-yr	58	47	-11
100-yr	66	60	-6

The HEC-RAS models of the existing 2- and 5-yr storm events simulated minimal overbank flooding; the proposed SWM model reduced these simulated water surface elevations even further, providing greater freeboard for overbank flooding.

The HEC-RAS SWM concept model of the 10-yr storm simulated reduced water surface elevations and eliminated existing overbank flooding from the upstream cross sections 40, and 28. The model of the SWM improvements still experiences significant backwatering from the 108” culvert downstream, which results in the culvert overtopping and roadway flooding for cross sections 27-24 for the 10-yr event. 10-yr HEC-RAS water surface elevations between the existing and proposed SWM models dropped by 1.0 ft or less for the 1-D section below the 108” culvert. Flood depths and overall roadway flooding is reduced through all cross sections for the 100-yr event, and simulated roadway flooding was eliminated for 2 of the 27 existing cross sections that exhibited roadway flooding in the HEC-RAS model.

TUFLOW modeling of the proposed SWM concepts simulated reduced flooding from all storm events. The changes between the existing conditions and proposed SWM models are evident in the floodplain extent shown on the maximum flood depth maps.

The SWM concepts reduced the maximum extent of flooding more for the 5-yr event than for the 10-yr storm event. The concepts reduced roadway flooding and flooding around dwellings in Area 4 and Areas 5 and 6 for the 5- yr storm event, while the 10-yr event showed the greatest reductions in the parking lot of La Palapa and County owned Parking Lots ‘D’, ‘E’, and ‘F’. The SWM concept model reduced flood depths in the roadway at representative Cross Section ‘E’ by 0.66’ and by 0.78’ on the north overbank along representative Cross Section ‘F’.

The Tropical Storm Lee event is included in the iterations to allow for readers of this report to see a comparison of the expected improvements against a recent memorable event. The effects of the proposed SWM improvements for the Tropical Storm Lee event are evident throughout the modeled area. Reductions in flood plain extent were fairly comparable throughout the modeled area. For this storm event, the greatest impacts resulting from the SWM improvements are largely depth of flow reductions in areas 3 and 4. This can be evidenced by the change in inundation level in and around the dwellings in these areas. The effects of SWM improvements on the Tropical Storm Lee event most closely resembled the SWM effects for the 10-yr storm event.

The simulated floodplain extent of the 50-yr storm decreased under the SWM Concept model because flows did not overtop the culvert flowing below Ellicott Mills Dr. Without overtopping this culvert, the floodplain from the SWM model did not expand nearly as far into Parking Lot 'F' and did not escape onto Frederick Rd. until the driveway just west of Court Ave.

The SWM concepts had the greatest impact on flood depths of the 100-yr storm, however, this had a minimal effect on the overall extent of flooding because all culverts were still overtopped and road banks were flooded in the same locations. The depths, velocities, and overall extent of flooding from the 100-yr SWM Concept model closely match those simulated for the existing 50-yr model because their peak flowrates are very similar.

4.4.2 RESULTS OF CONVEYANCE IMPROVEMENTS

The proposed conveyance improvements had no impact on the total inflows to the model, thus all changes to the flow patterns were a direct result of the added storm drain structures. The HEC-RAS portion of the model was not greatly affected by inclusion of conveyance Concept 4; the water surface elevations of the 2- and 10-yr storms decreased by approximately 0.2 feet for the majority of the 1-D modeling region, while the 100-yr water surface only decreased by approximately 0.1 foot. For the cross sections immediately above the second large culvert (96") (cross sections 3 and 4), the water surface of the 2-yr event dropped approximately 1.3 ft under the storm drain concept model, while the 10-year water surface dropped 0.17 ft. and the 100-yr storm was negligibly impacted.

The TUFLOW model of conveyance concepts exhibited similar, negligible impacts on flooding for this upper section. The greatest effects of the storm drain concepts were simulated for the 10-yr event and are at representative Cross Section 'B', which is located immediately upstream of Concept 5. The addition of Concept 5 appears to reduce backwatering behind the 96" culvert, and reduces the water surface elevation in the channel by 0.6 ft, which was a greater reduction than was simulated for the SWM concept model. Floodplain water surfaces at representative Cross Section 'B' are negligibly impacted, indicating that the flooding relief of Concept 5 is localized and thus water is still escaping into the floodplain further upstream. In the heavily populated area where Concept 5 has diverted flow from the stream (8516 Frederick Rd. to 8450 Frederick Rd.), the overall extent of flooding appears slightly diminished for all storm events, as evidenced by the depth of flooding maps.

The results at representative Cross Section 'C' indicate that, for the 10-yr storm, Concept 5 had negligible impacts on water surface elevations downstream from where it reintroduces flow into Hudson Branch. For the 100-yr storm, Concept 5 redirected flow into the channel at representative Cross Section 'C', which eliminated the minimal flooding of the roadway and south overbank that had been simulated for the existing conditions model.

Concept 6, which diverted flow from west of Court Ave. to the open section in Parking Lot ‘E’, had conflicting effects on flooding of the downtown area between representative Cross Section ‘D’ and the intersection with Old Columbia Pike. The concept successfully diverted a portion of flow from the Frederick Rd. corridor, which reduced flood depths and velocities in the roadway and the flooding extent in parking lots along Frederick Rd. At representative Cross Section ‘E’, existing roadway flood depth was reduced by 0.5 ft by the 10-yr, storm drain model. Concept 6 also alleviated some flooding upstream of Court Ave. as evidenced at representative Cross Section ‘D’, where flood depth in the floodplain was decreased by 0.5 ft and 0.25 ft for the 10- and 100-yr storms, respectively.

Because Concept 6 diverted flow away from Frederick Rd. and into the stream channel in Parking Lot ‘E’, Parking Lot ‘E’ experienced increased flooding for all storm events. Concept 7 was designed to aid in the conveyance of flow through Parking Lot ‘E’, and it achieves this goal (see Concept Flow Comparisons, *Appendix C*), however, flood depth and flooding extent in Parking Lot ‘E’ still increases for the conveyance concept model. This is likely because the flow added to the stream from Concept 6 backwaters into the parking lot behind the footbridge.

Generally speaking, the reductions and effects of this concept for the Tropical Storm Lee event fall between the 10-year and 100-year events.

4.4.3 Results of Combined Improvements

The models showing the combined SWM and conveyance improvements simulated the greatest reductions in overbank flooding for all model areas except for Parking Lot ‘E’, where the SWM concept model simulated the least flooding.

The combined SWM and conveyance concepts HEC-RAS model simulated a cumulative effect on water surface elevations, however with only minimal reductions resulting from the conveyance improvements, the combined model water surface elevations were very similar to those of the SWM model. Compared to the existing model, the 100-yr water surface of the combined concepts model reached the roadway on 22 of 40 cross sections, which was four fewer than the existing condition model; three of the four cross sections where existing roadway flooding was eliminated were the same for both for the SWM and combined models.

Because the TUFLOW conveyance model did not greatly affect flood extents for the 50- and 100-yr storms, the TUFLOW combined model for these events is very similar to the SWM model. For the 5- and 10-yr storm events, the proportion of total flow manipulated through the storm drain concepts was substantial enough to alter overall flow patterns, thus the flooding extent of the combined model was most different from the SWM model for these storm events.

5- and 10-yr, existing water surface elevations were most substantially reduced with the combined TUFLOW model at representative Cross Sections 'D' and 'E'. At representative Cross Section 'D', the combined model reduced 10-yr, existing water surface elevations by nearly 2 ft in most areas. At representative Cross Section 'E', the 10-yr existing water surface elevations were reduced by 1.7 ft in the roadway and existing flooding of the parking lot at La Palapa was eliminated. In Parking Lot 'E', the combined model had slightly higher water surface elevations than the SWM model, however both models had similar flood extents within the Parking Lot; 10-yr existing roadway water surface elevations at representative Cross Section 'E' were 0.8 ft lower with the combined model than with the SWM model.

The greatest reductions in existing water surface elevations for the 100-yr event were simulated at representative Cross Sections 'A', 'B', and 'E'. In the south floodplain of representative Cross Section 'A' and in the channel of representative Cross Section 'B', existing water surface elevations dropped by 1.2 and 1.3 ft, respectively. At representative Cross Section 'E', existing flood elevation in Parking Lot 'E' decreased by 1.2 ft and by 1.1 ft in the roadway. Combined model flooding elevations in the channel and the immediate overbank along representative Cross Section 'F' were approximately the same as those simulated for the SWM model, while in the roadway, the combined model flood elevations were 0.2 ft lower than the SWM model (1.2 ft lower than the existing condition).

5.0 CONCLUSIONS

1-dimensional and 2-dimensional modeling of the downtown Ellicott City watershed has provided valuable insight into existing flood patterns of the region and allowed for assessment of the potential mitigation strategies to reduce future flooding from large storm events.

Models were calibrated with anecdotal evidence from the Tropical Storm Lee flooding event and used to simulate the existing flood conditions for large storm events (2-, 5-, 10-, 50-, and 100-yr recurrence intervals and the Tropical Storm Lee event). The results of the existing condition models were then used as baselines to evaluate three flood mitigation scenarios which included stormwater management improvements, conveyance improvements, and improvements combining stormwater management and conveyance concepts.

The results of the proposed concept modeling suggest the greatest reductions in flooding, as measured through flooding extent, flood depths, and flood velocities, would be achieved with the stormwater management pond concepts. The storm drain conveyance options offer only minor improvement in some areas relative to water surface elevations, and show increases in other areas downstream of the improvements, making the storm drain options less desirable. The proposed stormwater pond concepts will offer incremental, though not dramatic, reductions in flood elevations during a historical event like Tropical Storm Lee.

Also part of the study was an examination and assessment of the overall watershed effects of small-scale, SWM design concepts proposed by the Center for Watershed Protection (CWP). The proposed CWP facilities within the focus watershed were catalogued and applied to the existing condition TR-20 model. These facilities were found to have minimal impact on the discharge to the watershed outlet for the 2-yr storm, and thus were not considered as part of flood mitigation strategies for the large storm events targeted in this study.

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