2016
Ellicott City Hydrology/Hydraulic Study
and Concept Mitigation Analysis

McCormick Taylor Project No. 5519-93
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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

TABLE OF APPENDICES

EXECUTIVE SUMMARY

This study was expanded from the original 2014 Ellicott City Flood Study and Concept Mitigation Report at the request of Howard County Bureau of Environmental Services for the purposes of extending a detailed hydraulic model of the flood flows encountered along Frederick Rd./ Main St. in Ellicott City, Howard County, Maryland, and using that model to examine the effect of additional proposed conceptual improvements on flooding conditions. Several hydrologic models of the Hudson Branch, Tiber Branch and New Cut Branch subwatersheds of the Tiber-Hudson Branch 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 July 30, 2016 was also synthesized to calibrate the hydraulic model against observed flooding conditions during the event. National Weather Service (NWS) estimates were used as part of the hydrologic calibration of this storm synthesis.

Updates to the original hydraulic floodplain model along Main St. included expanding of the limits of 2-D (TUFLOW) models from the channel confluence with the Patapsco River upstream to the US 29 crossing, and to include small segments of the Tiber Branch and New Cut Branch in the vicinity of the Main St. corridor. Once developed and calibrated, this revised model served as a baseline for the comparison of a new set of flood mitigation concepts, including additional stormwater quantity management in the three tributary watersheds and additional storm drain and culvert conveyance systems through portions of Frederick Rd.

Flood mitigation approaches in the report focused on a goal of reducing the 100-year event flows as close as possible to the 10-year event flows, effectively reducing peak flow below the undeveloped “woods, in good condition” runoff scenario, which represents runoff potential assuming a woods land cover over the entire watershed, but does not change the existing channel or infrastructure along Main St. In the interest of achieving this reduction with as few discrete project sites as possible (i.e. cost-benefit efficiency) stormwater quantity management opportunities focused on larger facilities in-line with existing stream channels, particularly in the Tiber Branch and New Cut Branch subwatersheds. In the Hudson Branch subwatershed, where space was not available for sufficient in-line storage management with traditional ponds, alternatives also included underground management, and conveyance improvements to minimize roadway flooding.

The combined effects of the conceptual improvements noted above were run through the expanded 2-D hydraulic model to demonstrate the resulting reduction in flooding elevations relative to existing conditions. Proposed conditions analyses were run for the 10-, 25-, and 100-year events, as well as the synthesized July 30, 2016 event. These results are represented by color flood flow depth mapping, and described in detail within the report. The 100-year event also considered a subset of mitigation options to examine an incremental improvement condition below the full suite of recommended management options. The report notes that the improvements are independent of an event that creates backwater flooding of lower Main St. from the Patapsco River at its 100-year flood stage; backwater flooding from such an event (Tropical Storm Agnes, 1972) is not significantly impacted by improvements in the Tiber-Hudson watershed because the Patapsco River has a substantially (80X the Tiber-Hudson) larger watershed, which is responsive to a less-localized, general heavy rainfall across the majority of the watershed.

1.0 INTRODUCTION

1.1 BACKGROUND

Hudson Branch, a tributary of the Tiber-Hudson Branch, itself a tributary to the Patapsco River, winds along Main St. in Ellicott City, Howard County, Maryland. Runoff from the 1.55 square mile watershed of the Hudson Branch, 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 St. before meeting its confluence with the 0.54 square mile watershed of the Tiber Branch in a parking lot south of Main St. (Parking Lot 'D'). The Tiber-Hudson Branch continues eastward from Parking Lot 'D' in a confined channel where it meets its confluence with the 1.55 square mile watershed of the New Cut Branch. The combined flow of this total 3.7 square mile watershed (the remaining 0.06 sq. mi runs to the combined channel at the downstream end) continues through a confined channel under several historic buildings before meeting its ultimate confluence with the Patapsco River.

The confined nature of the channel, due in part to the steep topography surrounding Main St., as well as the historic buildings which line or straddle its immediate banks, contributes to the dramatic flooding experienced in the Main St. corridor during certain intense rainfall events. The development within the watershed, built over time beginning with Ellicott City's founding in 1772, some of which is managed for quantity control to varying degrees, also plays a role. The severe flooding experienced on Main St. and surrounding areas during the intense July 30, 2016 event, where over 6" of rain fell in about 2 hours, was an extreme example with a recurrence probability of 0.1% based on 3-hour National Oceanic and Atmospheric Administration (NOAA) Precipitation Data for the region. The storm caused widespread flooding of the Main St. community and its surrounding homes and businesses with flooding in excess of 6' feet deep in places. Several buildings along the channel experienced significant damage, and dozens of cars were washed downstream into the Patapsco River, resulting in two fatalities. This damage extended up Main St. from the historic district to the West End area just east of US 29.

Following up on the 2014 study performed for the Main St. commercial/residential district to analyze the effects of Tropical Storm Lee in 2011, the 2-D hydraulic model was extended from the Patapsco River downstream, to the US 29 crossing upstream for the Hudson Branch, with the confluence areas of the Tiber Branch and New Cut Branch also represented in the model. This analysis, performed using TUFLOW 2-D hydraulic modeling software along with detailed topographic survey, attempts to create a more accurate representation of typical Main St. flooding by considering the 2-D flow vectors resulting from floodwaters over this highly varied landscape. The further 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, which have been expanded well beyond the limitations of the previous study.

1.2 PROJECT LOCATION

This study focused on the historic section of Ellicott City, Maryland and areas to the west along Frederick Rd./Main St. (a.k.a Maryland 144) from the Patapsco River upstream to US 29. Short sections of the Tiber Branch upstream of Parking Lot 'D' and the New Cut Branch along New Cut Rd. were also included in the hydraulic model. The model was analyzed using TUFLOW to establish a 2-D floodplain surface. Proposed mitigation concepts, including stormwater quantity management and conveyance improvements, were identified for locations throughout the Tiber-Hudson Branch watershed. 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 Tiber Branch, New Cut Branch and Hudson Branch watersheds, combined with the previously developed and updated hydrologic model for the Hudson Branch that considers the effect of existing stormwater quantity management as a baseline for analysis. This hydrology includes a synthesis of the July 30, 2016 event.
- Develop a 2-D hydraulic** floodplain model through the area affected by the Main St. flooding during the July 30, 2016 event and calibrate the model based on observed conditions that day.
- Develop potential improvements to the hydrology of the Hudson Branch, Tiber Branch and New Cut Branch (additional management of stormwater quantity) and the hydraulics of the conveyance network through the town (improvements to channels, culverts 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 noted in the report, using the baseline hydrologic and hydraulic models developed for existing conditions as a means of comparison.

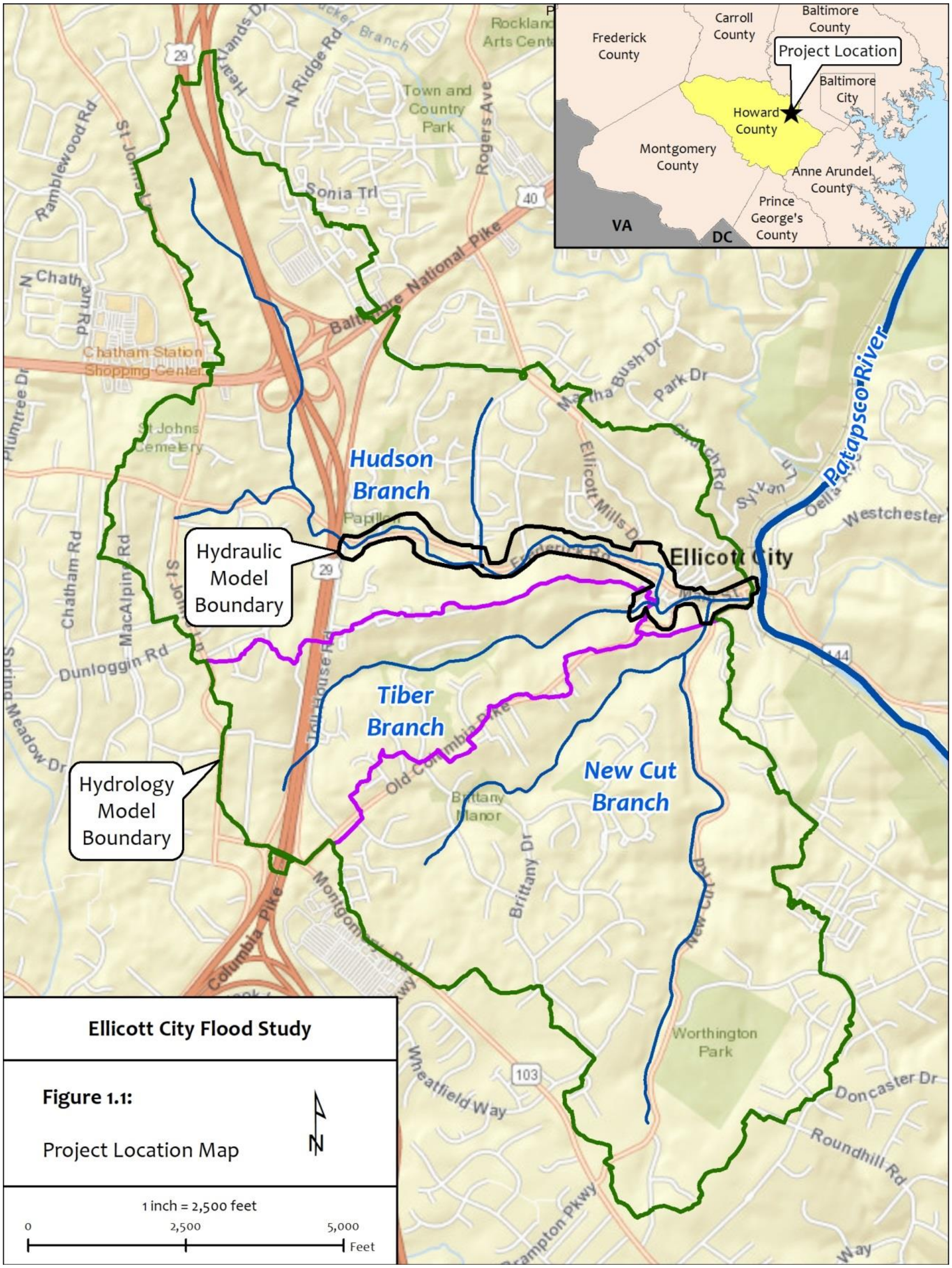
In addition to the goals defined above, this effort will generate a baseline model that can be used to examine various combinations of mitigation measures outside of the alternatives summarized in this report, such that the model can be a tool in the long term master planning effort for Ellicott City[t].

*Hydrology is the study of how much runoff will be generated within a watershed.

**Hydraulics is the study of how water will behave when flowing through and around topography or structures.

[t] May 31, 2017 Public Presentation can be found online at: https://www.howardcountymd.gov/Departments/Planning-and-Zoning/Community-Planning/Community-Plans/EC-Master-Plan

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. Two distinct TR-20 hydrologic models were developed; one representing the Hudson Branch watershed and one representing flows from the Tiber and New Cut Branches.

Subsequent TR-20 models were developed for the two large watersheds to represent different levels of subarea detail and the effects of stormwater management. The drainage area (DA) for the Hudson Branch was analyzed using TR-20 for a single drainage area (detail level 1), with seven (7) sub drainage areas (detail level 2), and with thirty-five (35) sub drainage areas (detail level 3). A combined drainage area for the Tiber- New Cut Branches was analyzed using TR-20 for a single drainage area (detail level 1), with eight (8) sub drainage areas (detail level 2) and with twenty-seven (27) sub drainage areas (detail level 3).

The level 2 TR-20 models were subdivided into the level 3 models in order to consider the effect of existing and proposed stormwater management within the hydrologic model, which required distinct subareas for each existing stormwater facility.

Once the architecture of the TR-20 model was set, rain gage data from the July 30, 2016 event 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 were compared to USGS discharge estimates from the event. The flow data (in the form of hydrographs) was then used as input for hydraulic models, which were calibrated using anecdotal information (witness account reports, video) about local water surface elevations during the July 2016 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. Though data from the initial 2014 study was used for a portion of the Hudson Branch watershed, the runoff curve number (CN) data was updated based on revised regulatory guidance regarding the soil classification (and resulting CN) associated with certain soil types present in the watershed. As a result, the runoff values are close but do not perfectly match those generated in the 2014 report. In addition to the existing conditions, current Howard County zoning data was utilized to examine ultimate conditions that reflect a full developed build out of the watershed, as a point of comparison to existing conditions. The existing conditions were quite close to the ultimate zoning results since few undeveloped sites remain in the watershed. In the interest of preserving the model for comparison with any future development conditions, and to be consistent with conditions on July 30, 2016, existing land use conditions were used for the analyses detailed below.

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 overall study was where the Tiber-Hudson Branch meets its confluence with the Patapsco River. This includes the subareas for the Hudson Branch, Tiber Branch (a.k.a. Cat Rock Run) and New Cut Branch. The New Cut Branch also includes the Autumn Hill tributary; for the purposes of this study, the Autumn Hill tributary is considered to be included in the discussions of the New Cut Branch.

The initial analysis (detail level 1) did not subdivide the Tiber-New Cut and Hudson Branch watersheds. The second analysis (detail level 2) subdivided the Hudson Branch drainage area into 7 subwatersheds; the Tiber-New Cut watershed was subdivided into 8 subwatersheds. The subwatershed boundaries were 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 for the Hudson Branch 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. For the Tiber, there are significant steep, wooded areas as well as residential land use. For the New Cut, the watershed is primarily developed residential areas with some steep and moderate woodland areas. Soil types include B, C and D Hydrologic Soil Groups, with the percentages are as noted below.

Table 2.1 – Hudson Branch Hydrologic Soils Distribution

Hydrologic Soil Group	*% of Drainage Area*
A	12%
B	27%
C	39%
D	22%

Table 2.2 – Tiber Branch/New Cut Hydrologic Soils Distribution

Hydrologic Soil Group	*% of Drainage Area*
A	9%
B	20%
C	60%
D	11%

Table 2.3 – Hudson Branch Land Use Information

Land Use	*Percent of Watershed*
Brush / woods	18.1 %
Pasture / open space / Agricultural	8.9 %
Impervious (roads, parking not incl. below)	5.8 %
Residential - 1 ac.	15.5 %
Residential – 1/4 to 1/8 ac.	27.8 %
Urban Commercial	15.7 %
Urban Industrial	8.2 %

Table 2.4 – Tiber-New Cut Branches Land Use Information

Land Use	*Percent of Watershed*
Brush / woods	27.6 %
Pasture / open space/ Agricultural	8.0 %
Impervious (roads, parking not incl. below)	0.6 %
Residential - 1 ac.	12.6 %
Residential – 1/2 ac.	2.7 %
Residential – 1/4 to 1/8 ac.	40.2 %
Urban Commercial	8.2 %
Urban Industrial	<0.1 %

Land use was derived from County GIS data and aerial photography, and the breakdown is noted above in *Tables 2.3 and 2.4*. Soils information for the project was obtained from the Web Soil Survey developed by NRCS (http://websoilsurvey.nrcs.usda.gov/). This data was used to determine curve number values for each study point using TR-55 methodology. See *Appendix A* for CN computations, Hydrologic Soils Maps as well as Land Use and Drainage Area Maps.

TR-55 methodology was also used for time of concentration calculations. The Hudson Branch model has a total time of concentration of 1.136 hours, or 68.2 minutes, and the Tiber/New Cut Branch watershed has a total time of concentration of 0.619 hours, or 37.2 minutes. An analysis of the overall drainage area indicated a total time of concentration of 1.18 hours, or 71 minutes, to the downstream study point at the confluence with the Patapsco River. See *Appendix A* for time of concentration computations.

The rainfall depths for the 24 hour duration storms were obtained from WinTR55 and represent NOAA Atlas 14 rainfall depths for Howard County.

Table 2.5 – NOAA Standard Rainfall Data

Return Period (years)	*Rainfall Depth w/ areal reduction (inches)*
2	3.19
10	4.91
25	6.14
50	7.23
100	8.47

Discharges were calculated for the 2-, 10-, 25-, 50- and 100-year recurrence intervals. The 24-hour NOAA_C rainfall distribution was used for all analyses except where shown in *Section 2.3.4* below, as this is the standard for stormwater management analysis in Maryland. The results of the TR-20 analysis for the two watershed models analyzed as single DAs (Level 1) and with large sub drainage areas (Level 2) shown in the section below in *Table 2.6*.

Table 2.6 – TR-20 Calculated Discharges for Standard Storm Events

	Return Period (yr)	*TR-20 Simulation (Level 1) (cfs)*	*TR-20 Simulation (Level 2) (cfs)*
Hudson Branch	*10-yr*	1208	1296
	25-yr	1682	1815
	50-yr	2121	2259
	100-yr	2618	2764
Tiber Branch	*10-yr*	--	453
	25-yr	--	664
	50-yr	--	855
	100-yr	--	1075
New Cut Branch	*10-yr*	--	1750
	25-yr	--	2450
	50-yr	--	3091
	100-yr	--	3771

2.2 USGS Estimates Calibration

As part of the July 30, 2016 post-storm analysis, representatives from USGS performed an estimate of flow in three separate channels, one in each of the three major subwatersheds (Hudson, Tiber and New Cut Branches), based on cross-section, estimated channel roughness and high water marks. This data was compared to the values estimated in the synthesized July 30, 2016 storm event TR-20 (Section 2.3.4 below) as a calibration. The Hudson Branch and New Cut Branch watershed discharge estimates were within the relative error window when compared to the USGS data (which was provided with a stated relative error of +/- 25%) and the Tiber Branch subwatershed was below the calibration window when compared to the USGS data. The hydrology parameters were examined for calibration based on the MD Hydrology Panel (2016) guidance and minor adjustments to time of concentration were made to calibrate flows for the Tiber Branch watershed. Given the small size (0.55 square mile) of the Tiber Branch watershed, potential error in the USGS post-storm measurement and the distributed variation of flow estimates in the overall Tiber-Hudson-New Cut Branches watershed, it was determined that further adjustment of hydrologic parameters of the Tiber Branch was not justified.

Table 2.7 – TR-20 Generated Hydrology for July 30, 2016 Event compared to USGS Estimates

	TR-20 Simulation (Level 3) (cfs)	*USGS Estimate (cfs)*	*USGS Estimate Range (±25%) (cfs)*
Hudson Branch*	3115	2750	2062 - 3438
Tiber Branch	1169	2100	1575 - 2625
New Cut Branch	3967	3320	2490 - 4150

*Estimate of flow at gauge location near Rogers Ave./Frederick Rd. intersection.

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 64 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 (<6 ac. Drainage Areas in Hudson Branch; <9 ac. in the other two subwatersheds), smaller SWM ponds were considered using a curve number reduction methodology in lieu of pond routing calculations, which were often not available for these smaller ponds. A detailed analysis of the validity of this approach can be found in the previous 2014 study (McCormick Taylor, 2014).

2.3.1 METHODOLOGY

The 18 largest, existing facilities in the watershed were represented as structures in the TR20 model and routed accordingly to model their effects on management. This included 8 facilities in the Hudson Branch watershed model and 10 facilities in the Tiber/New Cut Branches watershed model. From as-built drawings and computations, storage-discharge tables were developed to model the effects of each of these storage structures. Runoff from upstream was routed through the structures, then added (ADDHYD) to other runoff areas within the model. Refer to the drainage area map located in *Appendix A* that details the subareas and SWM described below.

The 46 smaller SWM facilities each have drainage areas less than 9 acres. To approximate the effect of their management, these facilities were incorporated into the TR-20 model by reducing the 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.

2.3.2 RESULTS WITH EXISTING SWM

Using the methods described above, the hydrologic results for the various recurrence intervals using the 24-hour NOAA_C storm event are noted in *Table 2.8* below. For the additional subdivision of drainage areas, in order to reflect timing differences in the hydrologic routing, reach routing was performed for drainage area routing lengths greater than 1000', however in most instances the routing was not significant enough to alter the results.

Table 2.8–Hudson Branch Subdivided Hydrology and Existing Management Results

	Return Period (yr)	TR-20 Simulation (Level 3), No Large SWM (cfs)	TR-20 Simulation (Level 3), With Large Existing SWM (cfs)
Hudson Branch	10-yr	1388	1203
	25-yr	2007	1768
	50-yr	2509	2313
	100-yr	3133	2907
Tiber Branch	10-yr	525	497
	25-yr	761	734
	50-yr	931	905
	100-yr	1057	1078
New Cut Branch	10-yr	1881	1640
	25-yr	2644	2330
	50-yr	3341	2988
	100-yr	3911	3581

Ultimately, the models incorporating all quantity SWM facilities (highlighted in *Table 2.8,* above) were used as the most representative scenario for the watershed; these models served as the comparison baseline for evaluation of future concept improvements.

2.3.3 WOODS IN GOOD CONDITION (UNDEVELOPED)

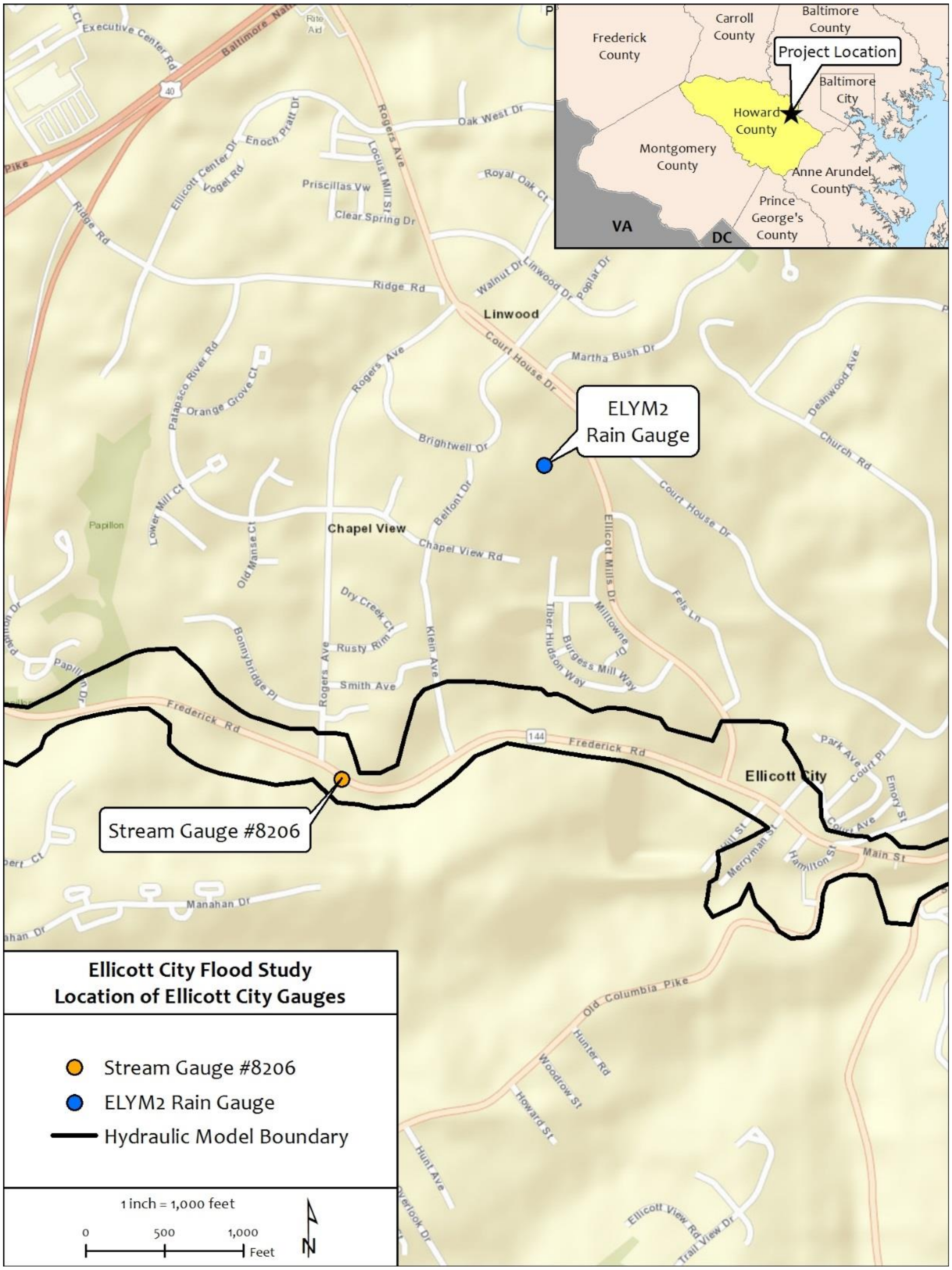
As a basic comparison of discharges if the entire watershed was managed to an "undeveloped" condition, a "Woods in Good Condition" TR-20 simulation was created. Under this scenario, the "woods in good condition" land use was assumed for the entire watershed, meaning the potential of the existing watershed to generate runoff was the same as if the area was entirely covered with woods. This scenario did not change the time of concentration for the watershed for several reasons: significant assumptions about the original channel geometry would be needed to replace the existing conveyance infrastructure; the existing infrastructure is unlikely to be completely removed and replaced with natural channel; and also, changing existing SWM infrastructure to natural channels would likely have negligible effect on the time of concentration, as the overall channel slope from top of the watershed to the outlet would be identical to the current conditions.

The results of the "woods in good condition" simulations are provided below and compared to the Level 3 discharges that include existing stormwater management. The undeveloped scenario represents significant reductions in the peak flows, however, as storm events become larger, the existing and undeveloped discharges become closer.

Table 2.9 – Undeveloped "Woods in Good Condition" Discharges compared to the Existing Conditions Discharges

	Return Period (yr)	*Existing Conditions Discharge (cfs)*	*Woods In Good Condition Discharge (cfs)*	*% Difference (cfs)*
Hudson Branch	*10-yr*	1203	629	-48%
	25-yr	1768	1064	-40%
	50-yr	2313	1507	-35%
	100-yr	2907	2075	-29%
Tiber Branch	*10-yr*	497	290	-42%
	25-yr	734	467	-36%
	50-yr	905	638	-30%
	100-yr	1078	842	-22%
New Cut Branch	*10-yr*	1640	1048	-36%
	25-yr	2330	1657	-29%
	50-yr	2988	2255	-25%
	100-yr	3581	2964	-17%

Figure 2.1: Location of Ellicott City Gauges used in the study



2.3.4 JULY 30, 2016 HYDROLOGY

In order to calibrate the hydraulic model to the conditions that were observed and recorded during the July 30, 2016 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 just over two 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 National Weather Service (NWS) provided precipitation data collected from a tipping-bucket style rain gauge (ELYM2) located along Court Ave. near the George Howard Building (*Figure 2.1, above*). This rain gauge data was normalized to provide a dimensionless rainfall distribution estimating rainfall at three minute intervals for the duration of the storm event. Cumulative rainfall for the storm was 6.6 inches and total duration of the event was 3.4 hours. Additional information on the details of the precipitation from this event can be found in a published study from NWS (NWS, 2016). The discharges simulated for the July 30, 2016 event are provided in *Table 2.7.*

3.0 HYDRAULIC MODELING

The study utilized 2-D modeling tools to develop the floodplain analysis through the study area. Detailed survey was collected for the surface of the entire area shown outlined in *Figure 3.1,* from the downstream side of US 29 to the confluence of the Tiber-Hudson Branches with the Patapsco River.

3.1 TUFLOW 2-D MODELING

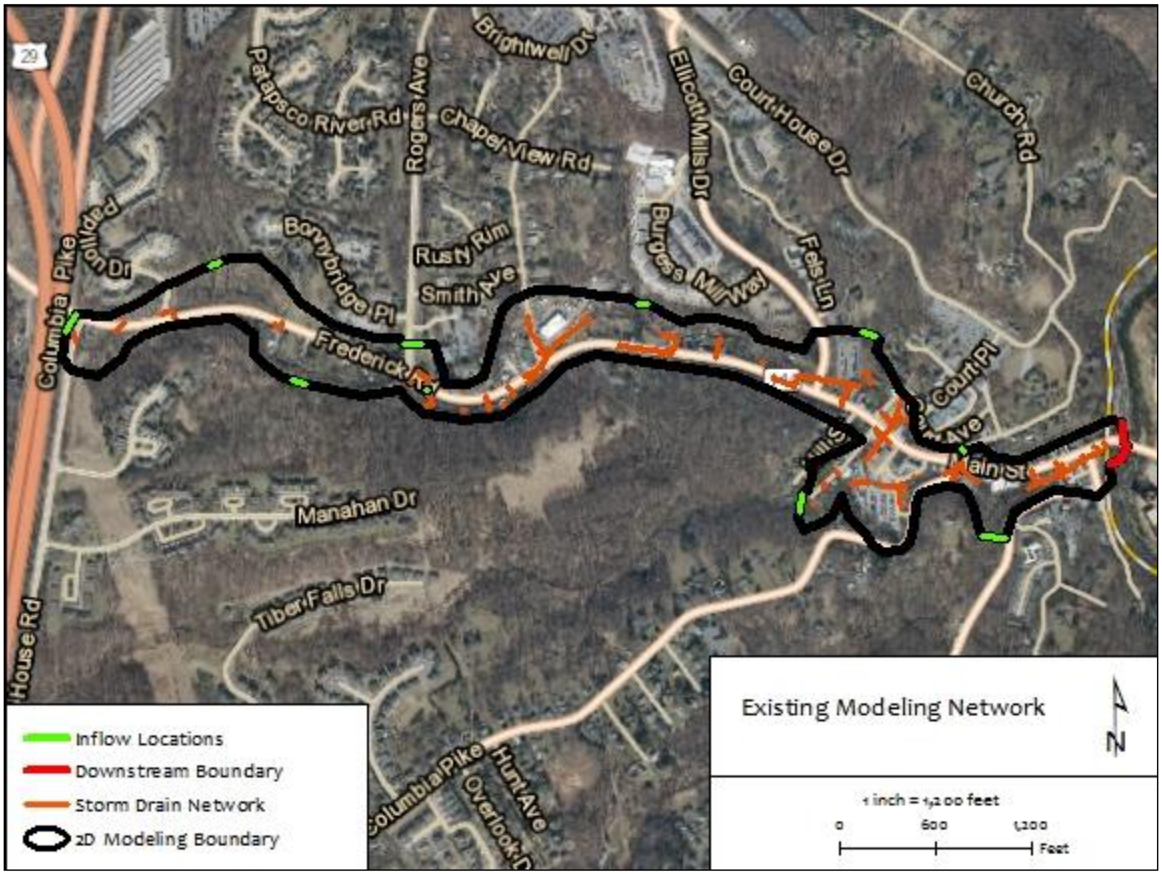
The TUFLOW simulation software provides computations for flood analysis using both 1-dimensional and 2-D solutions. The complexity of the drainage network and topography of the downtown area necessitated the use of a 1D/2D simulation program, such as TUFLOW, to best represent flood conditions.

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 the Surface-water Modeling System (SMS) (Aquaveo, 2016) software to create spatially oriented data layers and develop input files for the TUFLOW simulation program.

3.1.1 INFLOW HYDROGRAPHS

To represent the flow of water into the modeled region, it was necessary to define 10 different inflow hydrographs for each model scenario. Inflow hydrographs were generated using the TR-20 hydrologic models of the drainage area. The hydrographs at these inflow locations, including where the Tiber and New Cut branches entered the Tiber-Hudson main branch, were defined by specific cross sections within the TR-20 models. Runoff resulting from rainfall within the hydraulic model area was conservatively added to the closest upstream hydraulic model inflow. Hydrographs to seven inflow points were generated from the hydrologic model of the Hudson Branch and three inflow points were generated from the Tiber-New Cut Branches hydrologic model.

Figure 3.1: Schematic of Key Elements Used to Define the Hydrologic and Hydraulic Characteristics of the TUFLOW Simulations.

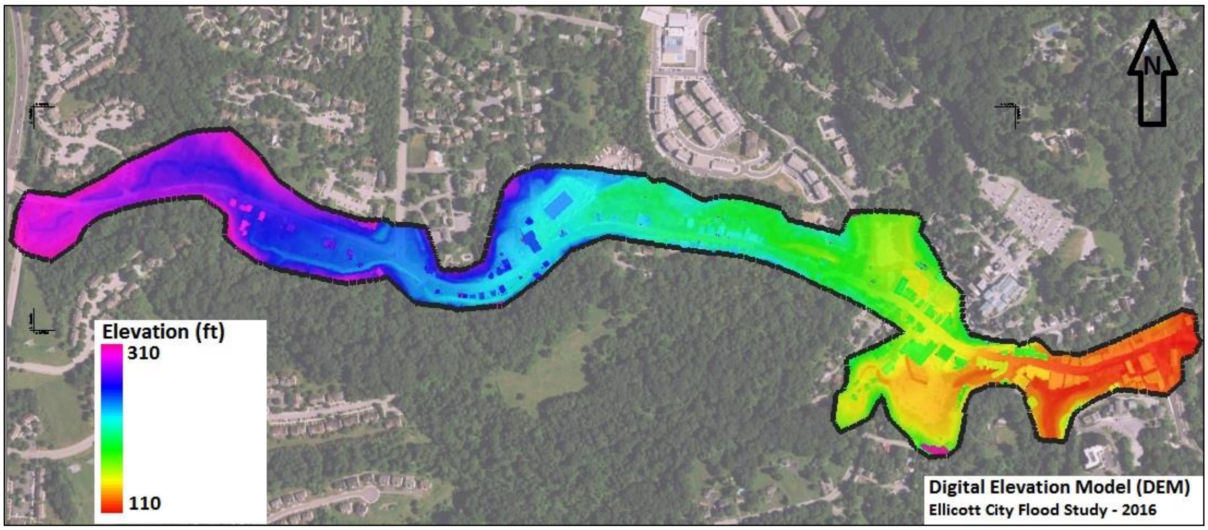


To reduce the simulation time for the TUFLOW models, the inflow hydrographs were abbreviated in duration to capture the peak discharges from each inflow location, while neglecting low flows at the beginning and end of the storm. The inflow hydrographs for the July 30, 2016 storm begin at time equal to 10 hour and have a duration of 5 hours, replicating the flows from approximately 6:10pm to 11:10pm on July 30, 2016. The standard storm events (10-, 25-, 50-, and 100-yr) were modeled with inflows beginning at time equal to 10.02 hours with a four hour inflow duration. The duration of each simulation was enough to calculate flood outputs for all significant flooding from each storm event. See *Appendix D* for inflow hydrographs.

3.1.2 TOPOGRAPHIC DATA

Another basic input requirement of TUFLOW models is topographic data to represent the ground surface within the model. Topographic data for the 2-D modeling area was acquired through aerial surveys supplemented with detailed field survey which produced digital terrain models (DTM) representing the surface of the ground. The DTM was interpolated to assign elevations to 5 foot square grid cells necessary for the TUFLOW simulation. 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 two 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.



3.1.3 MANNING'S ("N") ROUGHNESS VALUES

The Manning's roughness coefficient, 'n', is an estimate of the resistance to flow for a given area. Factors which may affect the roughness include bed material, vegetation, channel irregularities, and obstructions to flow. The Manning's roughness values were assigned based on field investigations, aerial imagery, and topographic survey data. Given the diverse landscape of the modeled area, a wide range of roughness values were defined, representing 19 different material types, with roughness values ranging from 0.02 for smooth pavement to 3.0 for solid buildings with minimal flooding.

3.1.4 EXISTING STRUCTURES

The TUFLOW simulations also required information detailing the inlet, storm drain, culvert and bridge network inside the 2-D modeling region. A conveyance structure network describing the inlets, storm drains and culverts 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. Bridge structures were represented as shapes within the 2-D modeling region. Most buildings in the model were represented with elevated topography and high roughness values.

3.1.5 Boundary Conditions and Time Step

Boundary conditions define how flows enter and exit the modeled area. All inflow boundary conditions were defined as flow versus time boundaries, allowing inflows to be represented by time dependent hydrographs. The downstream boundary condition was represented as a model computed water surface elevation versus flow boundary, based on the water surface slope. A starting water surface elevation at the downstream boundary of 113 feet was input for every model, to improve the computational stability of the simulations. Each model utilized a computational time step of 0.25 seconds.

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 the July 30, 2016 event. Once these parameters were finalized for each storm event, parameters were not changed, ensuring consistent comparisons between existing and proposed modeling scenarios.

Simulation outputs were generated at 5 minute intervals for each simulation, although further discussion focuses on the maximum outputs generated at each grid cell. The outputs generated by the TUFLOW model were post-processed using the SMS software to analyze outputs and generate graphics. A variety of output results can be generated to view variables such as flow, velocity, shear stress and water level at various times and locations throughout the modeled region. *Appendix D* contains maps that show maximum flood depths calculated during each simulation.

3.2 Model Calibration

In order to assure the model was accurately depicting depth and direction of flow through the terrain in the modeled area, anecdotal data was used as a point of comparison to the hydraulic model for the simulated July 30, 2016 storm event. The water surface elevations calculated with the July 30, 2016 event 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. Additionally, the simulation of large culverts in the TUFLOW model was compared to simulations of the culverts using alternative hydraulic modeling software published by the Federal Highway Administration, HY-8. These model calibration practices will assure, to the greatest extent possible given the available information and the resolution of the data, that the model will represent typical storm events in a manner that would represent the actual flooding conditions during such a storm.

3.2.1 Available Data from July 30, 2016 Event

A significant amount of anecdotal evidence from Ellicott City during the night of July 30, 2016 has been gathered through various sources on the web and through information or videos provided by the local citizens. Videos uploaded to YouTube and videos from security footage, in many cases provided 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. Below are some of the videos used for this purpose:

- Approximate Address: 8344 Main St. to Parking Lot 'D'; Post-storm along lower Main St.
    - Evaluated flow directions and depth at 8344, extent of flooding in Parking Lot 'D'
    - Evaluated post storm damage along lower Main St.
    - https://www.youtube.com/watch?v=ktHzzfPKlv8
- Approximate Address: 8125 Main St. to 8059
    - Evaluated flow directions, depth and velocities
    - https://www.youtube.com/watch?v=T4JMYuiieFc
- Approximate Address: 8059 Main St. to 8049
    - Evaluated flow directions, depth and velocities
    - https://www.youtube.com/watch?v=9-KmdQLEBKY
- Approximate Address: 8059 Main St. to 8049
    - Evaluated flow directions, depth and velocities
    - https://www.youtube.com/watch?v=k-shmNbxAqs
- Approximate Address: 8190 Main St.
    - Evaluated flow directions, depth, velocities and flow timing
    - Security camera footage;

      https://www.youtube.com/watch?v=nrGBtQhAvo8

A Howard County stage gauge (Gauge #8206) is located along the Hudson Branch in the concrete channel near the intersection of Frederick Rd. and Rogers Ave. The depth and time relationship recorded from this gauge was used as a measure of simulated model depth and flow timing to observed conditions.

A draft report prepared for Howard County, "Case Study- 2016 Ellicott City Flood Event" (Smith, 2017) was provided for this use as well, as it contains records of over 70 interviews with residents recounting their recollection of the event, including the depth and direction of flood waters on their property. Some of the anecdotal data referenced was still being vetted at the time of publishing of this report, however, this data was considered in conjunction with all other anecdotal evidence available as an additional data point for model evaluation.

3.2.2 Correlation With Models

The TUFLOW simulation model was compared to anecdotal evidence from the July 30, 2016 event, using generated outputs showing the extent of the floodplain, maximum depth of flooding, maximum shear stress, and velocity (direction, magnitude) of flow. The timing of the flooding was also examined. Generally speaking, the results of the calibration models correlated with the anecdotal data, within the expected tolerances for this type of analysis.

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 that were augmented to calibrate the model included material roughness, 1-D culvert form loss coefficients, model topography, 1D/2D boundary conditions, and inflow locations.

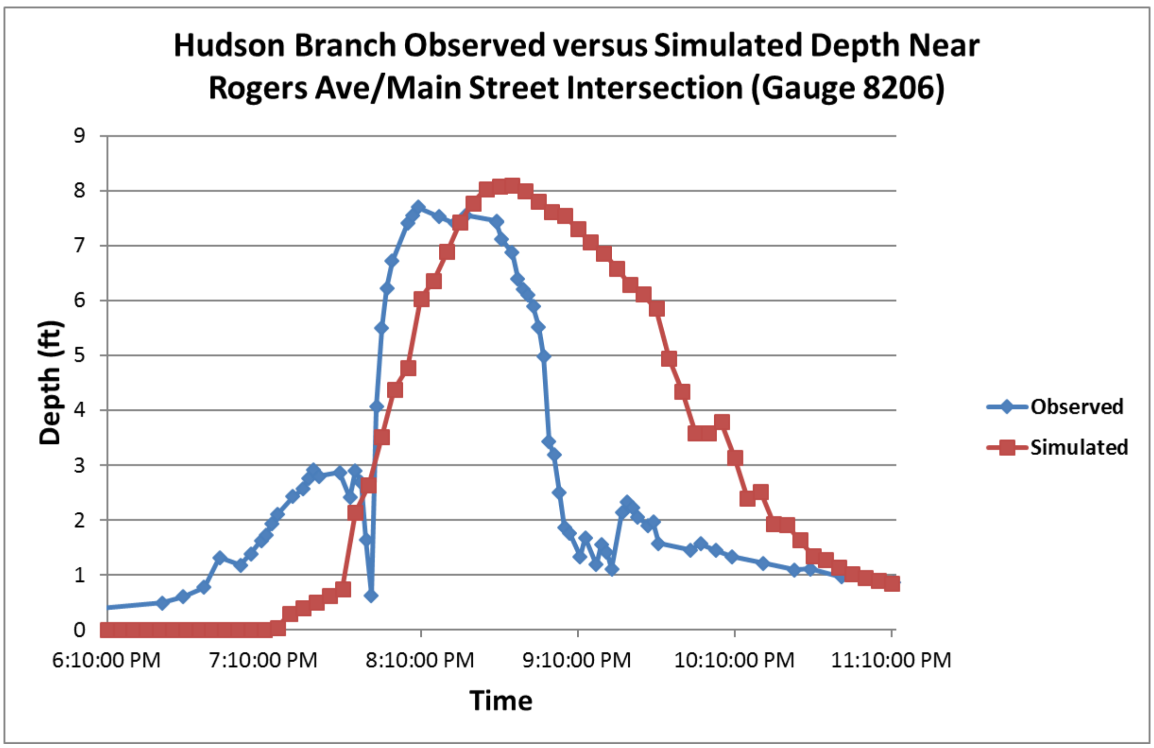
It is important to note, that the extent and depth of flooding in the model is intended to reflect flooding of the main branches of the Tiber-Hudson watershed and major inflows, and that localized, minor flooding resulting from smaller and less concentrated inflows is not shown. For example, anecdotal evidence indicates that properties along the north side of lower Main St. experienced flooding resulting from runoff coming down the steep, rocky hillside immediately to the north; this type of un-concentrated runoff was not the focus of this study. Additionally, the modeling assumes no change in model parameters during the simulation, which means it does not attempt to simulate variation in flows resulting from transient obstructions, like floating vehicles, and from events such as the embankment failure of a local sand filter.

The upstream portion of the TUFLOW modeling area (approximately 8879 Frederick Rd. to 8683 Frederick Rd.) contained a significant amount of flooding along the stream and some flow in the roadway. This area is less densely populated than the downtown area, and thus less anecdotal evidence was available. Reports from the Smith Planning Case Study report (Smith, 2017) indicated flow running down Frederick Rd. as a result of overtopping at each of the three main stream crossings in this area. Significant erosion was observed along the north side of Frederick Rd. just east of Papillon Dr.; the erosion in this area was simulated by the model as a location where flow from the roadway was reentering the channel and significant shear stresses were simulated along this stream bank.

The modeled area from Rogers Ave. east to Ellicott Mills Dr. was calibrated by adjusting the location of the inflows. Anecdotal evidence suggested the stormwater junction box on the northeast side of the Rogers Ave./Frederick Rd. intersection surcharged and that significant rates of flow were observed coming down Rogers Ave. onto Frederick Rd. The initial hydraulic model was adjusted to simulate flow surcharging the storm drain network and flowing down Rogers Ave. This split flow was simulated by having two inflow points; an inflow along the concrete channel was capped at a maximum discharge rate equal to the maximum capacity of the storm drain leading from the junction box, while flow rates greater than that capacity were injected onto Rogers Ave. Flow proportioning of this inflow provided an improved representation of the flows onto Frederick Rd. downstream of Rogers Ave.

The gauge data collected near the intersection of Rogers Ave. and Frederick Rd. was compared to model outputs to provide a measure of the model's ability to predict depth and storm timing.

Figure 3.3: Hydraulic Model depth output compared to recorded gauge depths from the July 30, 2016 storm event



Simulated depth and timing of the flows at the gauge correlated well with the gauge data and within the expected tolerances when considering the relative error associated with the hydrologic inputs and hydraulic variables at the site. Simulated flows were slightly delayed compared to observed data, but maximum flooding depth, and the relationship between depth and time indicated by the shape of the curve, are similar, indicating simulated depths for the Hudson Branch are fairly well represented by the model.

The hydraulic model parameters along the majority of the area between Ellicott Mills Dr. and the Tiber Branch confluence in Parking Lot ‘D’ remained the same as the parameters developed under the original 2014 flood study (McCormick Taylor, 2014). Buildings and the roadway in much of this area experienced severe flooding. The culvert from 8611 to 8580 Main St. was adjusted to reflect existing conditions, which includes the upstream 1/4 of the culvert length being 88” diameter and the downstream 3/4 of the culvert being 108” diameter. Model correlation with anecdotal observations was satisfactory for much of this reach, with only minor edits to stream topography for model calibration. The simulated extent of flooding in Parking Lot ‘D’ appeared consistent with flooded areas shown in videos online.

In lower Main St., significant flows were reported coming down Church Rd. The storm drain network from Church Rd. was assumed to be overwhelmed by the flows from large storm events, thus all flows originating in the drainage area uphill of Church Rd. were introduced into the street just north of the Church Rd. intersection with Main St. This inflow simulated runoff on Main St. that was observed during the earlier part of the storm event, before overtopping of the main stream channel. As previously noted, runoff identified coming from the north hillside behind the 8100 and 8000 Main St. blocks was not simulated, but some of those flows were included in the inflow on Church Rd.

Building footprints throughout most of the model areas were represented with high elevations and high Manning’s roughness values, as flood attenuation and conveyance through the buildings was generally negligible compared to other parts of the floodplain and anecdotal evidence specifying amount of flow through the buildings was variable; however, in the buildings from 8125 to 8077 Main St. along the south side of Main St., significant flow rates were observed through the buildings. These flows were supported by significant anecdotal and post-storm evidence; thus the initial model parameters were adjusted to allow simulation of flow through these specific buildings once it could no longer be contained within the stream. This adjustment results in significantly greater flows onto Main St. and more accurate flow behavior in the lower Main St. area.

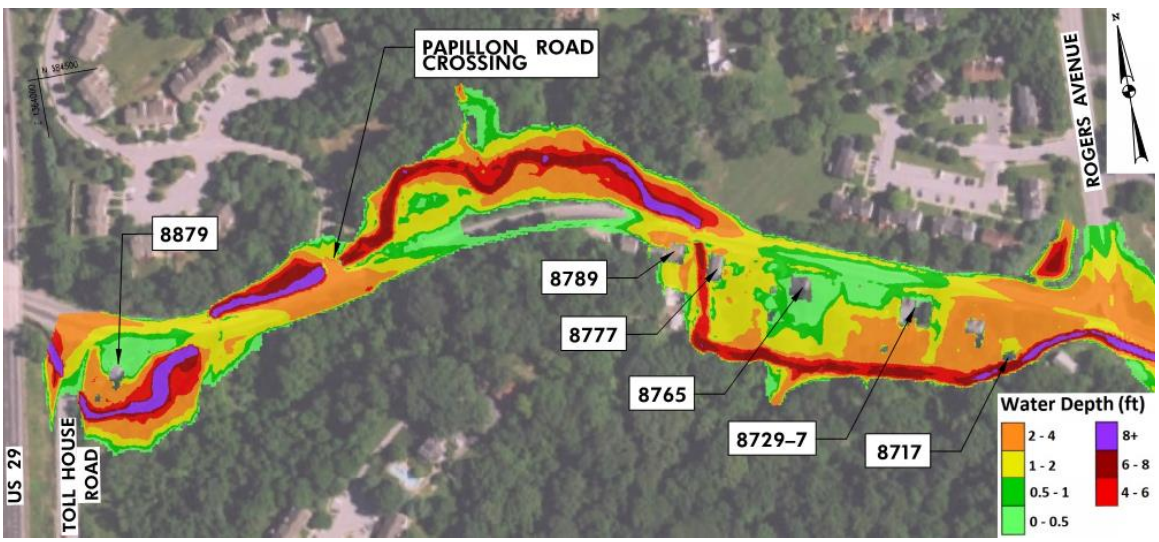
Other indicators of model performance in this downtown area were flow velocity and shear stress. Flow velocities between 10 and 15 ft/s were simulated in much of the area between 8250 and 8000 Main St.; these velocities are similar to velocities of floating debris observed in online videos of the event. Additionally, buildings and the roadway in this area experienced significant damage, which was predicted by the model simulation with shear stresses between 5 and 15 lb/sf along the roadway and within the buildings above the stream from 8125 to 8085 Main St.

3.3 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). For discussion purposes, the behavior of flooding under the various modeling scenarios is broken out into four different areas. The discussion below focuses on the benchmark 100-year, 24-hour storm event flood depths, a standard for floodplain determination and regulatory flood control permitting, which will be the point of comparison for improvement concepts. Note that the July 30, 2016 event flood elevations were on the order of 4”-6” higher in the problematic flooding locations, relative to the 100-year event. All house numbers noted below refer to Frederick Rd. (Main St.) unless otherwise noted. Results mapping of the 2-D models for various storm events including the 10- and 100-year, as well as the July 30, 2016 storm may be found in *Appendix D*.

3.3.1 AREA 1 – US 29 TO ROGERS AVE.

Figure 3.4: Location and Flood Depth Map of Area 1.



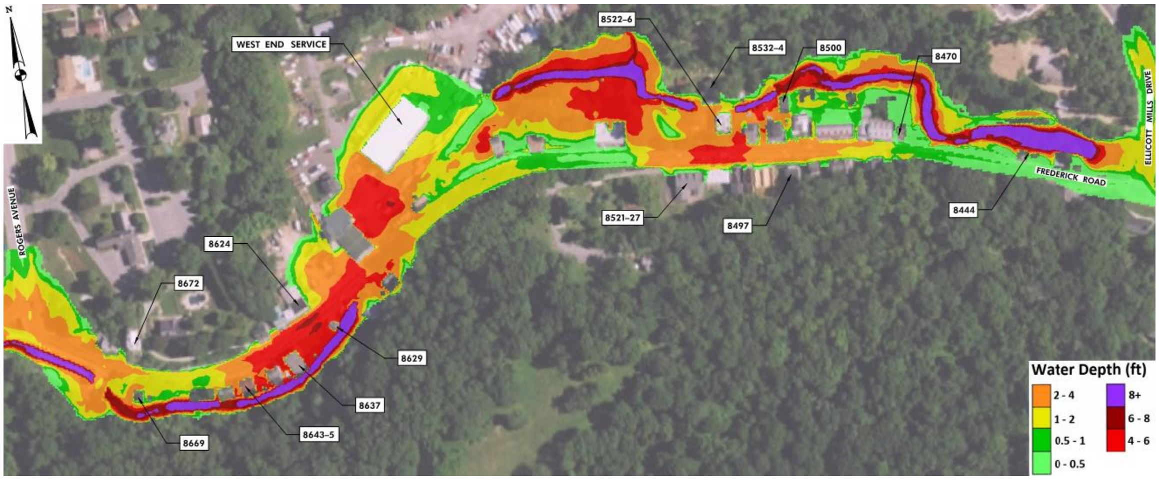
The model indicates the first instance of significant roadway flooding at the first point the stream crosses Frederick Rd. just east of Toll House Rd. in the 8800 Block where it shows as 1’+ deep, increasing to 2’+ deep at the second crossing of the stream under Papillion Dr. At its deepest point where a local sump in the roadway occurs near here, the roadway may flood up to 3’ deep. As the roadway grade ascends heading eastward the flooding depth decreases back to zero.

At the next stream crossing, southward under Frederick Rd. near 8789-77, there is 1’-2’ of roadway flooding due to insufficient culvert capacity. Flooding of the residential areas on the south side of the roadway occurs from 8777 east to the Rogers Ave. intersection, and is worst (2’+) from 8729 to 8717 where there is minimal floodplain availability for the stream between the adjacent hillside and roadway. This flooding of 2’+ continues into the roadway approaching Rogers

Ave. due to the confluence of storm drains carrying the runoff south into the channel, and the channel up against the roadway with no available floodplain.

3.3.2 AREA 2 – ROGERS AVE. THROUGH WEST END TO ELLICOTT MILLS DR.

Figure 3.5: Location and Flood Area Map of Area 2.

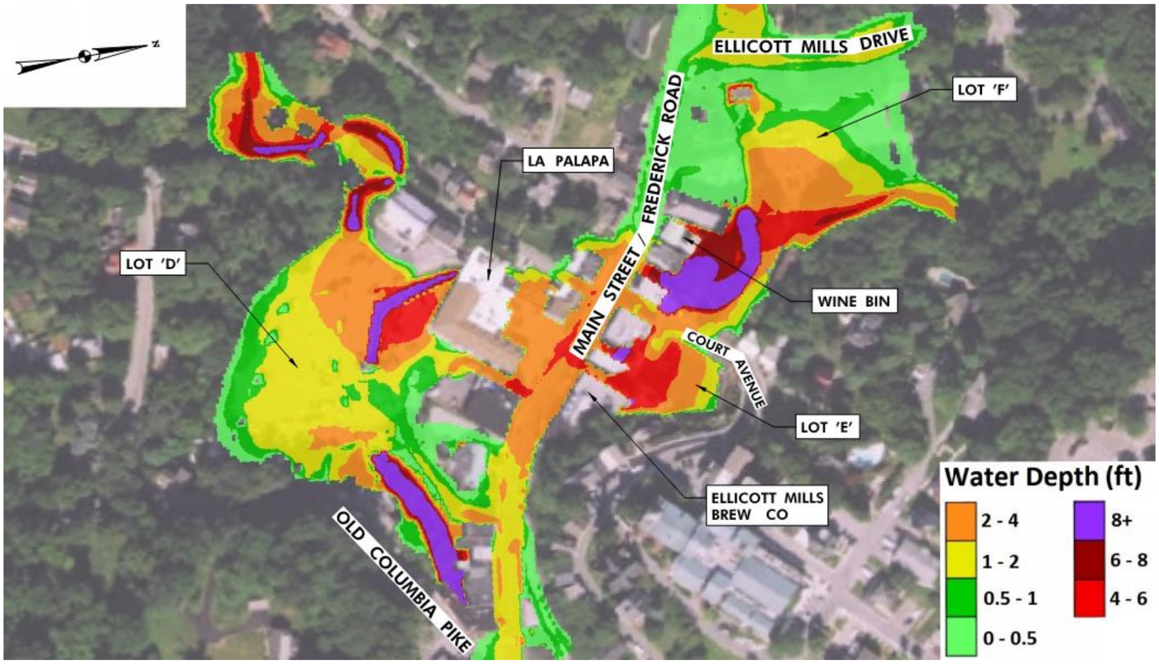


The 2'+ roadway flooding extends past Rogers Ave. to 8672, where the stream goes under a bridge adjacent to the roadway. The flood depth lessens slightly in the roadway then increases dramatically as it approaches the culvert entrance across from West End Service. Residential and roadway flooding in excess of 4' is indicated from 8643 on the south side, through the lower half of the West End Service property. The culvert that carries the stream was originally a 108" CMP culvert that was lined and reduced in diameter to an 88" culvert for part of its length. The interior of the culvert also has several projecting culverts that enter perpendicular to the length of the pipe from the north side, carrying runoff from West End Service and areas above. In total, this storm drain system appears inadequate to convey Hudson Branch for the 10-year storm and above, leading to the significant flooding in this area.

Beyond the West End Service property, the culvert outlets in a channel behind the residential buildings on the north side of the street, with 2'-5' of flooding in the area behind 8560-48. The flow approaches a 96" culvert behind the structure at 8522-26. This constriction, and the lack of available floodplain to the north of the channel results in flow being pushed out into the roadway, where flooding of 2'-4'+ occurs, with the worst of it between 8527 and 8511 in the roadway and between 8522 and 8500 through the residences on the north side. Beyond 8470, the roadway flooding is relatively minor (<1') as the channel becomes significantly deeper/larger as it approaches the culvert under Ellicott Mills Dr. The flow appears to overtop Ellicott Mills Dr. in the 100-year model by a foot or more, as it did during the July 30, 2016 event.

3.3.3 AREA 3 – ELLICOTT MILLS DR. TO OLD COLUMBIA PK.

Figure 3.6: Location and Flood Area Map of Area 3.



The water that overtopped Ellicott Mills Dr. combined with the water outfalling from the 114" x 192" arch culvert carrying the channel under the roadway and adjacent to Parking Lot 'F', and with a small tributary from the north, to backwater in the southeast corner of the lot to a depth of 2'+. The channel floodplain between the stream and the buildings along Frederick Rd. was flooded in excess of 6-8' in places, with that floodwater impacting the rear of the buildings from the Wine Bin to Court Ave. (8390-8340). The 15' wide x 9' high bridge under Court Ave., and confined downstream channel that runs along the south end of Parking Lot 'E' before turning south at 8316, create a constriction that pushes 2'-4' of flooding out into the roadway from 8360 down to Church Rd., where the steeper roadway grade lessens the depth but increases the flood velocity on Main St. The deepest roadway flooding of 4'+ occurs in Lot 'E' and in front of the Ellicott Mills Brewing Company.

The channel flowing south under Main St. is also constricted further by a 63'' x 171" arch culvert under the roadway, that opens up into a 16.5' wide x 14' high box culvert flowing under the La Palapa Restaurant before outfalling into a channel in Lot 'D' just upstream of the confluence with the Tiber Branch. The confluence which re-enters a bridge/box culvert that flows under the lot results in flooding ranging from 1'-2' in lower Lot 'D' to 2'-4'+ in upper (western) Lot 'D' just downstream of the restaurant. The water flowing through the lot eventually re-enters a deep, confined channel downstream of the lot that flows towards Old Columbia Pk. This flow remains separate from the flow down Main St. which left the channel in the Court Ave. vicinity as noted above.

3.3.4 AREA 4 – OLD COLUMBIA PK. TO PATAPSCO RIVER CONFLUENCE

Figure 3.7: Location and Flood Area Map of Area 4.

The channel flow downstream of Lot 'D' continued east under a set of buildings including the Precious Gifts store and under Old Columbia Pk., emerging back into an open channel near the confluence with the New Cut Branch behind 8167 Main St. The channel flow, lacking a floodplain due to steep slopes on the south side and buildings on the north side, flows under several buildings including the Caplan's and Portalli's Restaurant buildings, with the only relief for high water to be found pushing between the buildings onto Main St. or, in the case of the July 30, 2016 flood, through the first floor of the buildings onto Main St. The model was adjusted to reflect this possible flow path due to the results of that flood; without that relief water will backwater in the channel higher than the model above indicates. It remains to be seen in a similar event whether the reconstructed first floor walls and floor of the building above the channel will withstand the tremendous pressure of the flood and raise the back water, or sustain damage allowing the flood to flow through as it did on July 30, 2016. The current model presumes the latter for the purposes of this analysis.

The parallel flow down Main St. accelerates in the steep area from Church Rd. past Old Columbia Pk., flowing 1'-2' deep in excess of 12 feet/second (fps) and increasing the possibility of roadway scour and additional damage from the shear stresses in excess of 10 pounds/square foot (psf), as was witnessed in the July 30, 2016 event. Eventually this accelerated flow enters a relatively flat local low point of the roadway, which, combined with the channel flow pushing through the buildings as noted above, results in 6'-8'+ of flooding through this stretch between Caplan's and the Phoenix Emporium (8137 to 8049). Video at the peak of the July 30, 2016 storm indicated flows nearly touching the bottom of the store awnings in this area, supporting the calculations of the model.

As the flow of the combined three subwatersheds continues in the channel beneath buildings, through Tiber Park, and under the B&O Railroad Bridge, as well as down Main St., the inundation of the two flow paths reconnects them through this last stretch prior to combining with the Patapsco River. In looking at the subsequent improvement strategies for conveyance and stormwater management, this area will prove to be the most challenging to return to a manageable depth for the 100-year and similar storm events due to the flat grade, full watershed contribution and lack of a floodplain in the confined channel under several structures.

4.0 CONCEPTUAL IMPROVEMENTS

This study focused on two main types of conceptual improvements, stormwater quantity management (SWM) to reduce the quantity of flow into the Frederick Rd./Main St. 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. The structure of the model created for this study allows for any variation on, or combination of, improvements to be run through the model as part of a larger long-term planning effort, however for the sake of keeping the large amount of data manageable, the focus of this study looks at a progressively cumulative improvement using four types of approaches in total, and subsequently examines an incremental improvement considering selected individual improvements as defined below. The alternative of retrofitting the existing SWM facilities in the watershed is also examined relative to the other options presented below.

The approach to determining how much SWM storage is necessary to effectively reduce flood elevations and the probability of damaging flooding was based on attempting to store as much of the volume as possible that makes up the difference between the 10- and 100-year events, in order to reduce the peak flow of the 100-year event down to that of the 10-year event. This required temporary storage in the form of ponds as well as underground SWM. The effectiveness of each in reducing peak flow can be seen in *Figures 4.1* through *4.3* below.

For the SWM ponds, all in-line ponds assumed allowance for the 5-year storm event to pass through before accumulating meaningful storage. This is based on the premise that the downstream channels can accommodate this storm event, and that the meaningful storage could then be reserved for the higher storm events. This is also allows for the branches to maintain their existing base flows, and not changing the appearance of the stream running through downtown. Volume was maximized based on available undeveloped area with emergency spillways routing the higher storm events where necessary. During the large storm events, excess runoff would be temporarily stored within the facilities and let out at a controlled rate. At the time of this report, the County has initiated preliminary discussions with the Maryland Department of the Environment (MDE) regarding the in-line nature of the ponds as well as the likelihood of high hazard dams that will require Emergency Action Plans for downstream areas.

Figure 4.1: Peak Flow and Volume, 10- and 100-Year Storm.

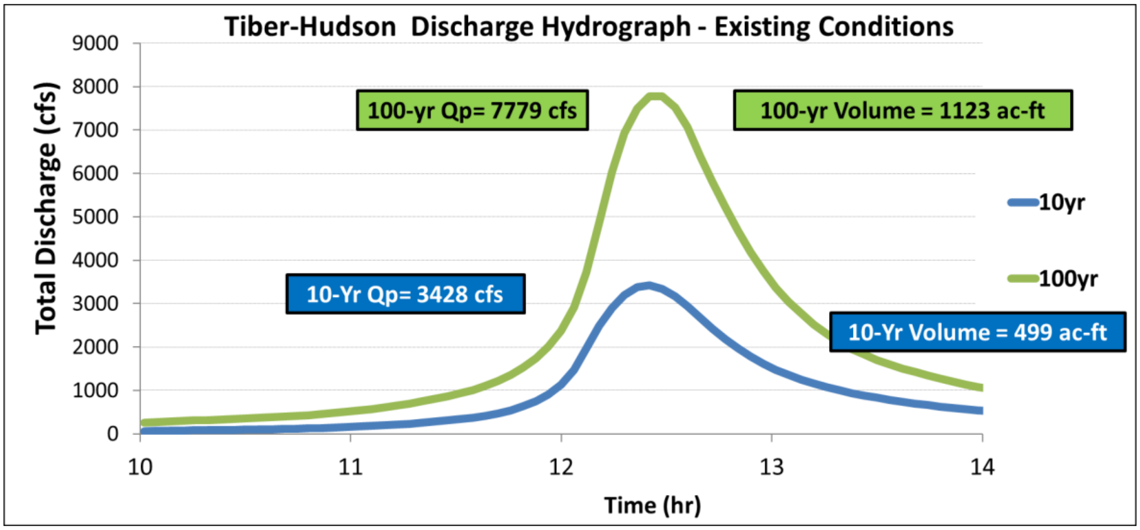


Figure 4.2: Peak Flow and Volume, 10- and 100-Year Storm.

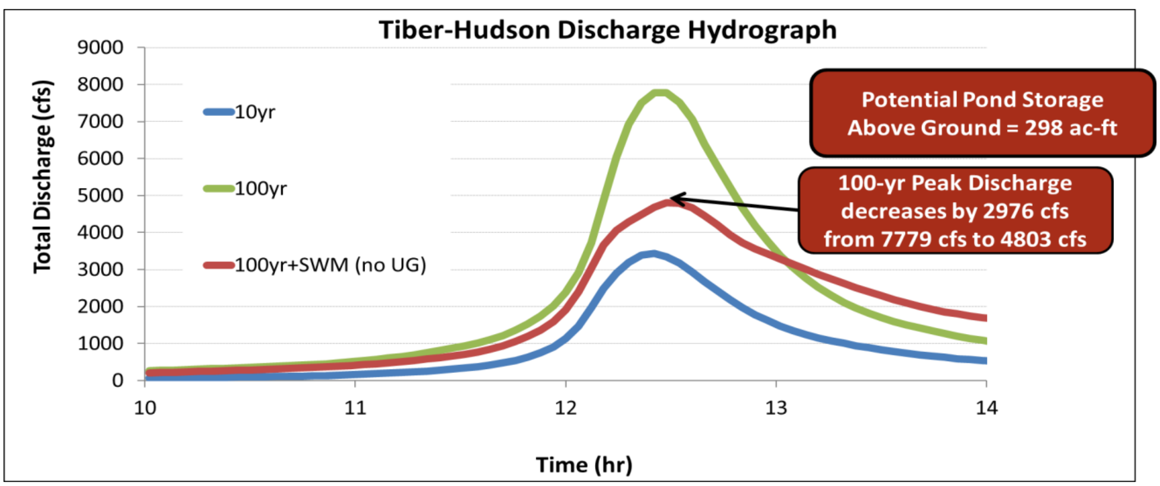
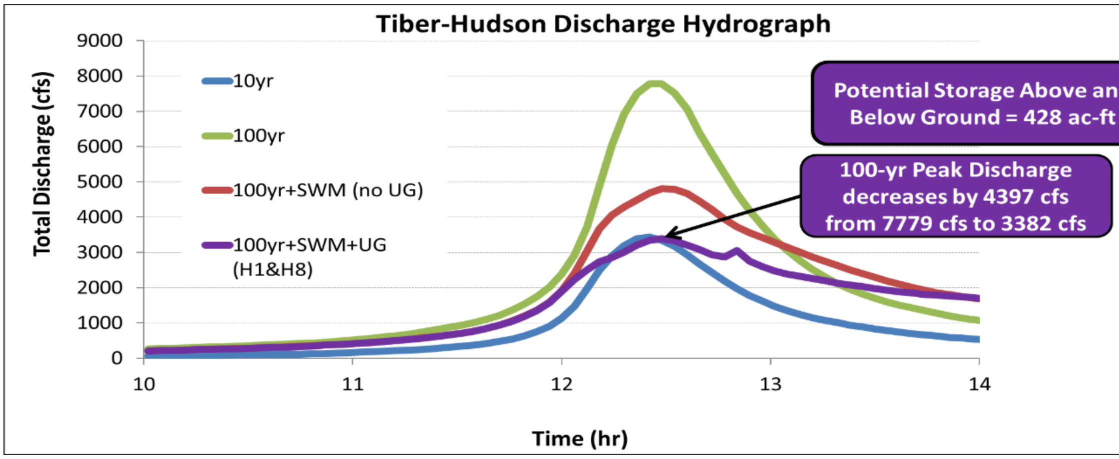


Figure 4.3: Reduction in Peak By Storage, Above and Below Ground SWM



For underground SWM areas, two approaches were considered: underground pipe storage, aka 'pipe farms' which would exist offline, storing diverted flow up to maximum capacity and outletting metered flow by gravity; and underground vaults, which are concrete storage spaces that store diverted excess flow from the channel and drain utilizing pumps over the course of 2-3 days following the storm event. All SWM facility conceptual layouts and grading maps can be found in *Appendix B.*

Capacity improvements examined include supplemental cross culverts where the Hudson Branch crosses the roadway, which are generally only effective at reducing flooding in their local vicinity; bypass culverts which supplement existing culverts carrying Hudson Branch and have effectiveness in reducing flooding in portions of the West End; and tunnels bored through existing rock under adjacent highlands and buildings to carry excess flow underground and divert it away from Lower Main St. Maps of conceptual conveyance improvements are found in *Appendix B.*

4.1 TIBER BRANCH

Improvements in the Tiber Branch focused on a single, large in-line SWM pond (T1), approximately 70 acre-feet in storage size. This was chosen as it was feasible within a wider, undeveloped area of the floodplain without excessive excavation relative to the volume of storage; and also because its size in this smaller subwatershed makes it particularly effective at reducing the peak flows out of this subwatershed. This would likely be a high-hazard dam. Additional details are noted in *Table 4.1*.

4.2 NEW CUT BRANCH

Improvements in this subwatershed included the examination of several in-line SWM ponds which attempted to maximize available undeveloped floodplain area for storage. From that initial set, there was a notable drop off in the effectiveness of the sites below a certain volume threshold of about 12 acre-feet, so going forward the four largest, most effective ponds were chosen for the concept modeling. Three of these ponds (NC1-NC3) were in-line within the Autumn Hill tributary, with the upstream-most pond being the most effective when examined individually. The downstream-most pond of the three, because of its location, which does not have an emergency spillway location, would likely need to be constructed as a concrete dam. All three ponds would likely be high-hazard dams. The fourth (NC-4) is near the headwaters of New Cut in the southeast corner of the watershed, and is the smallest and least effective of the four when examined individually.

4.3 HUDSON BRANCH

The Hudson Branch subwatershed was the most challenging one to find locations for the large in-line SWM ponds that were so effective in reducing peaks within the other two subwatersheds, largely because of the development adjacent to the floodplain, which is denser and more commercial than the other subwatersheds, and also because this branch is very much intertwined with Frederick Rd./Main St. in its lower reaches. Because all of the meaningful flooding takes place within this branch, before and after its confluences, this is where the majority of the improvements are conceptually proposed and examined.

4.3.1 STORMWATER PONDS

Conceptual improvements include three SWM ponds in-line and off-line within the US 40 / US 29 interchange (H5-H7), which is owned by Maryland State Highway Administration (MSHA) as well as three additional ponds adjacent to or within the Hudson Branch (H2-H4), with all but one (H2) upstream of US 29 at Frederick Rd. The pond in the NW loop ramp of the interchange (H7) which is online, is the most effective in this subwatershed when examined individually; the pond in the opposite NE loop ramp (H6) which is offline, the least effective of the six.

4.3.2 UNDERGROUND SWM

Conceptual Improvements include pipe farms and vaults as defined above. The pipe farm in the old Roger Carter Center property above Lot 'F' on Ellicott Mills Dr. (H8-UG1) includes ~4600 LF of 10' diameter pipe. The additional 3 sites (H8-UG2-4) are located west of US 29 in the undeveloped strip of land currently owned by BGE for their high tension power lines. These pipe farms would comprise ~3.3 miles of 10' diameter pipe located near but not in the footprint of the current towers. The total storage of these 4 sites is approximately 40 acre-feet. At the time of this report, BGE has not been contacted by the County to discuss specific locations for use of their Right-of-Way.

There are three concrete vault locations (H1-UG1-3) along the Hudson Branch east of US 29 which combined offer up to 90 acre-feet of storage, and, when used in conjunction with the pipe farm facilities (H8) are effective in significantly reducing the peak flows in this subwatershed. The locations are at Lot 'F', the current West End Service site and the areas between residential structures at 8777-8729 Frederick Rd. These sites represent conceptual storage of volume divided up based on footprint, but in fact their relative sizes and locations could vary depending on subsurface conditions (which may allow easier, deeper excavation, at one site vs another) with their overall effectiveness varying little, so long as the quantity of storage remains the same.

Table 4.1 and 4.2 indicate the volume and reduction in flow resulting from each of the individual SWM alternatives, as well as combined for the subwatersheds.

Table 4.1: Peak Flow Reduction Per Facility and Combined, Tiber Branch and New Cut Branch Watersheds

Tiber Proposed SWM				
	Total Without Concept Management		Total With Concept Management	
	Q10	Q100	Q10	Q100
T1 (Tiber)	497	1078	168	334

Tiber Concept Ponds Treatement Summary	
	Tiber
	T1
Storage	70.0 ac-ft
Emb. Height	24 ft
Change to Q100 - Total Tiber 100YR	-69%

New Cut Proposed SWM				
	Total Without Concept Management		Total With Concept Management	
	Q10	Q100	Q10	Q100
NC1 (New Cut)	1640	3581	1630	3053
NC2 (New Cut)	1640	3581	1396	3052
NC3 (New Cut)	1640	3581	1241	2876
NC4 (New Cut)	1640	3581	1462	3420
Total Combined	**1640**	**3581**	**965**	**2464**

New Cut Concept Ponds Treatment Summary					
	New Cut				Combined New Cut Concepts
	NC1	NC2	NC3	NC4	
Storage	34.0 ac-ft	42.0 ac-ft	63.0 ac-ft	14.4 ac-ft	153.4 ac-ft
Emb. Height	28 ft	18 ft	21 ft	11 ft	
Change to Q100 - Total New Cut 100Y	-15%	-15%	-20%	-4%	-31%

Table 4.2: Peak Flow Reduction Per Facility and Combined, Hudson Branch Watershed

Hudson Proposed SWM				
	Total Without Concept Management		Total With Concept Management	
	Q10	Q100	Q10	Q100
H1 - UG (Hudson)	1203	2907	734	2613
H2 (Hudson)	1203	2907	1124	2821
H3 (Hudson)	1203	2907	1162	2864
H4 (Hudson)	1203	2907	955	2663
H5 (Hudson)	1203	2907	1128	2798
H6 (Hudson)	1203	2907	1161	2823
H7 (Hudson)	1203	2907	1129	2598
H8 (Hudson) BGE/RGR CRTR	1203	2907	903	2459
Total Combined	**1203**	**2907**	**669**	**752**

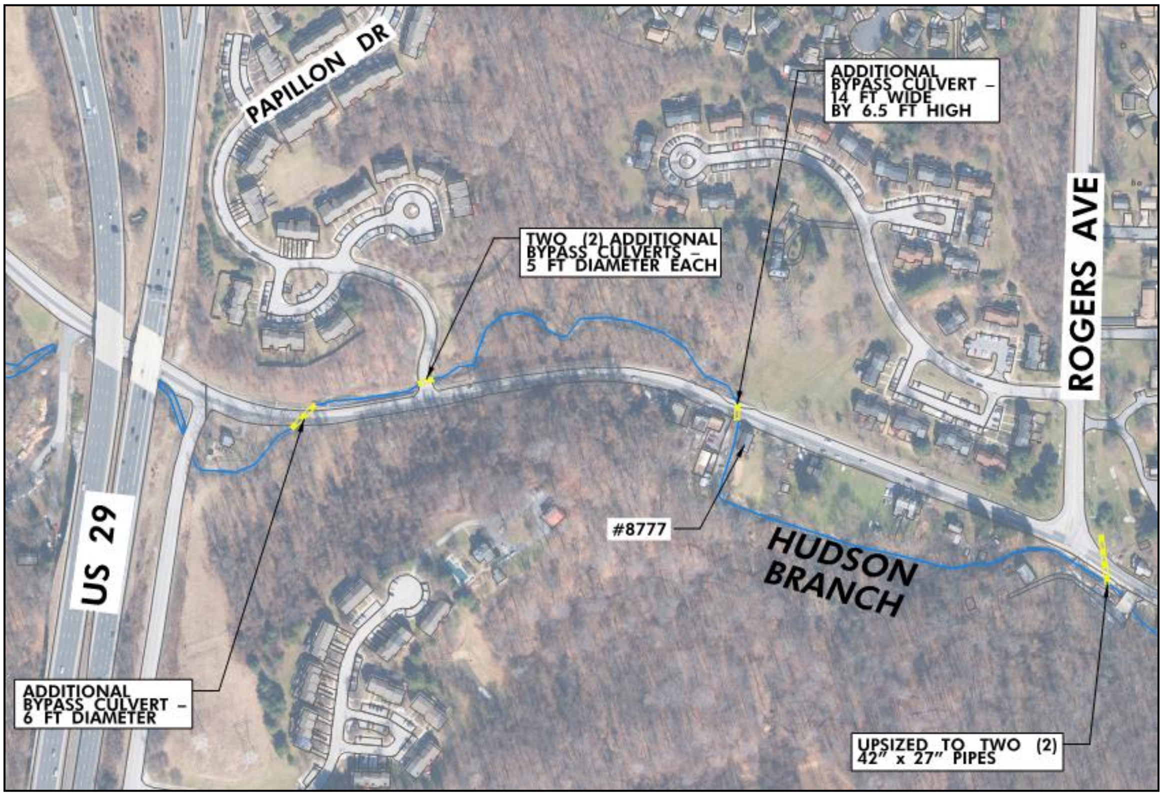
Hudson Concept Ponds Treatment Summary									
	Hudson Branch								Combined Hudson Concepts
	H1-UG 1-3	H2	H3	H4	H5	H6	H7	H8-UG 1-4	
Storage	82.4 ac-ft	15.0 ac-ft	7.7 ac-ft	15.6 ac-ft	11.5 ac-ft	12.0 ac-ft	12.8 ac-ft	40.0 ac-ft	197.0 ac-ft
Emb. Height	N/A	15 ft	11 ft	9 ft	12 ft	14 ft	12 ft		
Change to Q100 - Total Hudson 100YR	-10%	-3%	-1%	-8%	-4%	-3%	-11%	-11%	-74%

4.4 CONVEYANCE IMPROVEMENTS

Conceptual improvements to the capacity of pipe and culvert systems along Frederick Rd./Main St. include supplemental cross culverts added to the model in the following locations:

- 8800 Frederick Rd. – Additional 6' culvert
- Papillon Dr. – 2 Additional 5' culverts
- 8777 Frederick Rd. – Additional 6.5' x 14' box culvert
- 8680 Frederick Rd. @ Rogers Ave. - 2 – 42" x 27" pipes – This carries flow from Rogers Ave. across the road into channel

Figure 4.4: Supplemental Cross Culvert Locations

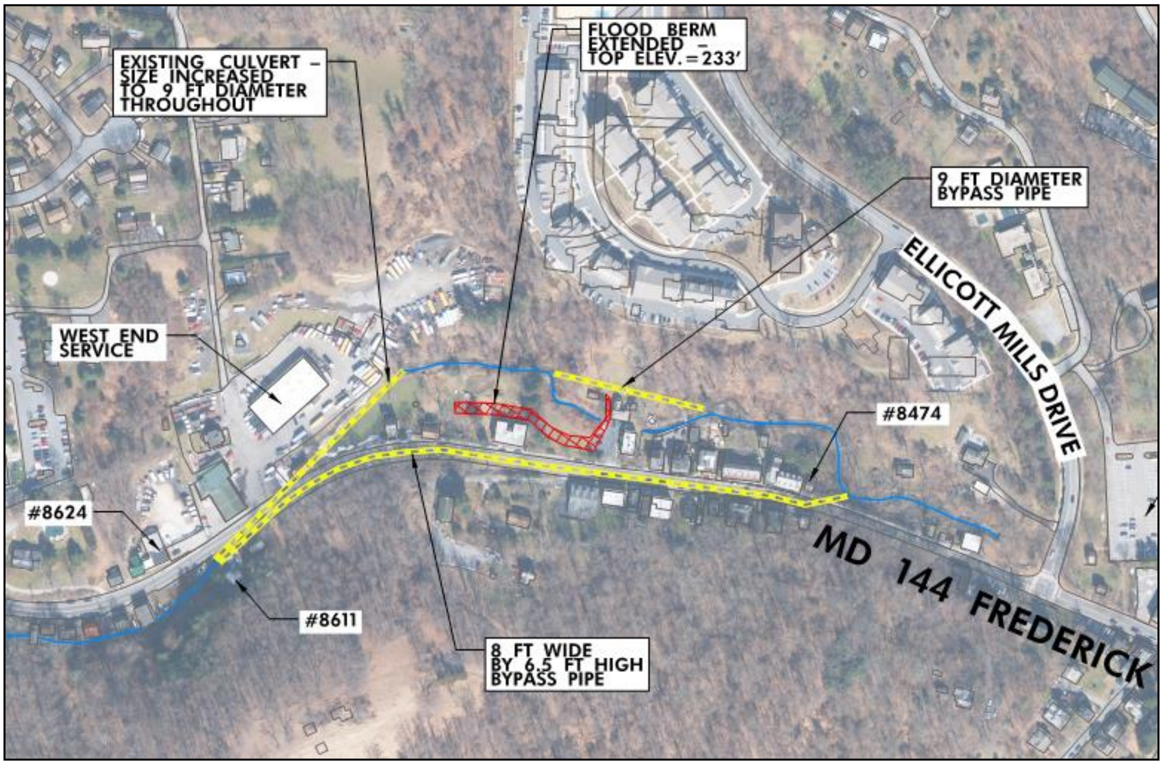


To address the capacity issue at the existing 108"/88" culvert at 8611 Frederick Rd., the model includes the following conceptual improvements:

- Restore the existing culvert to 108" diameter throughout and add a supplemental 6' x 8.5' culvert along the roadway to carry additional flow to an outfall into the channel downstream of 8470

- 8532/34 Frederick Rd.: add a 9' bypass culvert to carry flow behind the houses at 8532 where constricted by the existing culvert, and combine with a flood berm from spanning from 8572 to 8534 to protect adjacent houses from floodplain flow.

Figure 4.5: Supplemental Bypass Culvert Locations



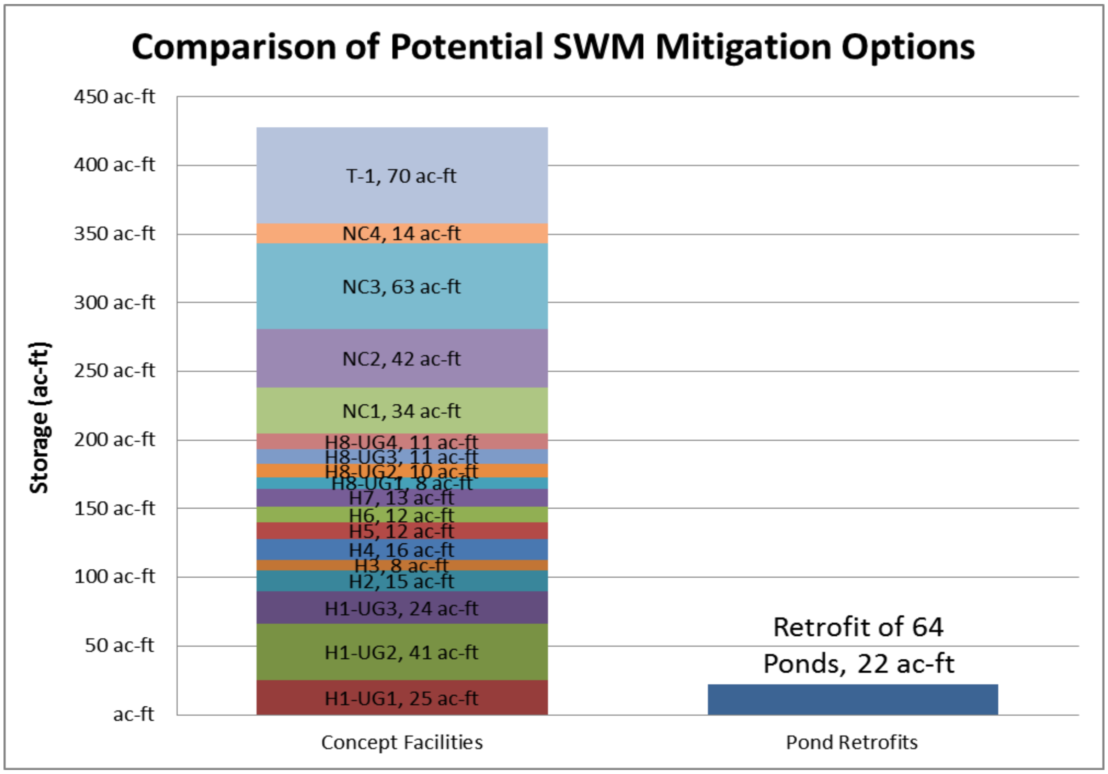
The effects of the capacity improvements on the hydraulic models are shown in more detail and discussed in Section 4.7 below. Larger maps of the options can be found in *Appendix B*; modeling in *Appendix D.*

4.5 EXAMINATION OF RETROFIT OF EXISTING SWM FACILITIES

The analysis considered what the impacts would be on retrofitting the existing 64 SWM facilities throughout the watershed relative to the larger scale SWM improvements noted above. The existing ponds account for about 85 acre-feet of available dry storage combined. Considering a rough assumption that, based on constrictions of adjacent development, right-of-way, natural resources, etc., each facility could be increased by about 25% on average, that would yield approximately 22 additional acre-feet storage.

Relative to the changes observed from the creation of 18 new facilities for 428 acre-feet of additional storage, the approach of retrofitting all 64 existing SWM facilities did not warrant further modeling based on the effective change per each of the 64 individual projects (~1/3 acre-foot per site, on average). A relative scale of this option can be seen in *Figure 4.6*, below.

Figure 4.6: Existing Retrofit Comparison to Conceptual Improvements



4.6 Flow Reduction From SWM Improvements

As discussed, the stormwater management improvements both above and below ground, provide substantial attenuation of the peak flows, resulting in reduced peak discharges into the 2-D hydraulic model. Provided below is a summary of SWM simulated changes in peak flows from the three subwatersheds (*Tables 4.3-4.5*) as well as change in peak flow at the outlet of the 2-D hydraulic model. The discharges summarized for the three subwatersheds were pulled directly from the hydrograph output by the TR-20 hydrologic model. The peak flows in *Table 4.6* reflect the combined peak of all inflow hydrographs for the hydraulic model, assuming all conceptual improvements are constructed.

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

Storm Event	*Peak Flowrate (cfs)*				
	Existing Conditions	*Proposed Above Ground SWM Concepts*	*Percent Change*	*Proposed Above & Below Ground SWM Concepts*	*Percent Change*
10-yr	1203	743	-38%	699	-42%
25-yr	1768	1116	-37%	730	-59%
100-yr	2907	2010	-31%	752	-74%
July 30, 2016	3549	2517	-29%	1396	-61%

Table 4.4 – TR-20 Simulated Peak Flowrate to Tiber Branch Watershed Outlet for Existing Conditions and the Proposed Stormwater Management Concept

Storm Event	*Peak Flowrate (cfs)*		
	Existing Conditions	*Proposed Above Ground SWM Concepts*	*Percent Change*
10-yr	497	168	-66%
25-yr	734	212	-71%
100-yr	1078	334	-69%
July 30, 2016	1169	438	-63%

Table 4.5 – TR-20 Simulated Peak Flowrate to New Cut Watershed Outlet for Existing Conditions and the Proposed Stormwater Management Concept

Storm Event	*Peak Flowrate (cfs)*		
	Existing Conditions	*Proposed Above Ground SWM Concepts*	*Percent Change*
10-yr	1640	965	-41%
25-yr	2330	1411	-39%
100-yr	3581	2464	-31%
July 30, 2016	3967	2519	-37%

Table 4.6 – TR-20 Simulated Peak Flowrate to Hudson-Tiber-New Cut (Tiber-Hudson Branch) Outlet for Existing Conditions and the Proposed Stormwater Management Concept

Storm Event	*Peak Flowrate (cfs)*				
	Existing Conditions	*Proposed Above Ground SWM Concepts*	*Percent Change*	*Proposed Above & Below Ground SWM Concepts*	*Percent Change*
10-yr	3428	1828	-47%	1801	-47%
25-yr	4947	2716	-45%	2511	-49%
100-yr	7779	4804	-38%	3382	-57%
July 30, 2016	8669	5503	-37%	3455	-60%

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*.

4.7 MODELING RESULTS OF PROPOSED IMPROVEMENTS

Water surface elevations, and extent of flooding, are reduced incrementally as stormwater management and conveyance improvements are progressively introduced. Below is a summary of the effect of the 428 acre-feet of SWM storage, and subsequently the addition of conveyance improvements, to the existing conditions models detailed above. Additional, larger graphics, which also include a breakdown of flood modeling results between above and below ground SWM improvements, may be found in *Appendix D*

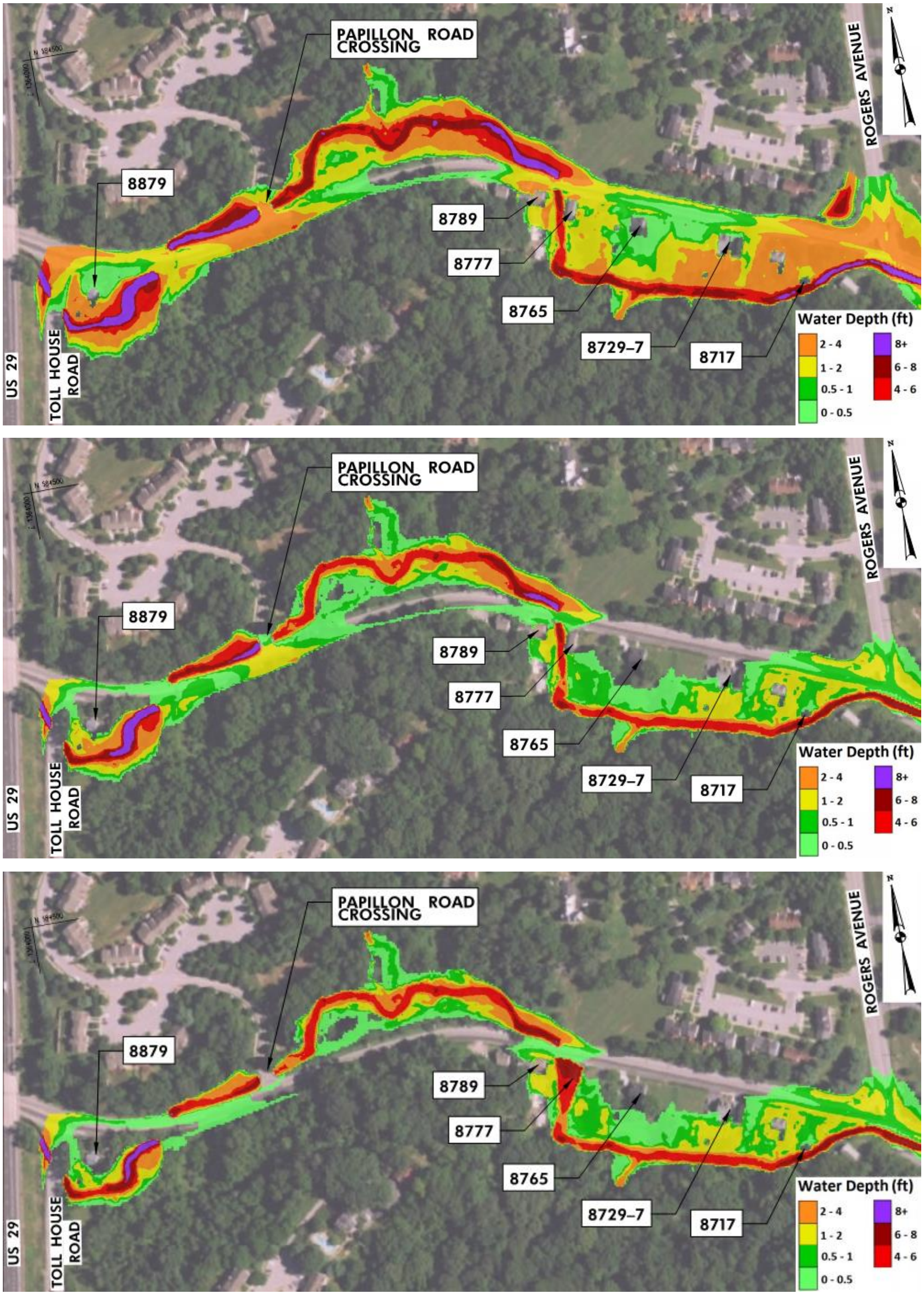
It's important to note that where the model graphics below represent "no flooding" (no color) on the roadway or adjacent areas, that this is indicative of *a lack of flooding resulting from water overflowing out of the channel or overburdened pipe structures only*. This does NOT mean there would be no flow or water depth in the area during this storm event, but rather that the model does not account for all runoff initiated in the immediate vicinity. The model considers the flow directed to the channel from the 10 hydrograph input points within the model and the handling of the major flow 'through' the Frederick Rd./Main St. community. It does not consider the hyper-local runoff between those points that may result in additional minor, local flooding.

4.7.1 AREA 1 – US 29 TO ROGERS AVE.

The roadway flooding at the first point the stream crosses Frederick Rd. just east of Toll House Rd. in the 8800 Block is reduced to under 1' deep, and down below 2' deep at the second crossing of the stream under Papillion Drive. This is a decrease of 1'+. The addition of the supplemental cross culverts at these first two locations further reduces the roadway flooding to about 6" deep.

At the next stream crossing, southward under Frederick Rd. near 8789-77, flooding is reduced below 1' under both scenarios. Flooding of the residential areas on the south side of the roadway is also reduced from 8777 east to the Rogers Ave. intersection, with areas of 2'-4' of flooding now reduced in extent, and in depth down to 0.5'-2', though there are some localized increases at the outlet of the supplemental culvert at 8777. At this culvert it appears either the conveyance or SWM improvement will result in these improvements, but combined they do not provide a significant additional benefit in the immediate vicinity. This is similar with the flooding of the roadway approaching Rogers Ave., which is reduced from 2'+ down to 0.5' to 1' near the roadway edges.

Figure 4.7: Location and Flood Depth Maps of Area 1: Existing, w/ SWM Improvements and w/ SWM+Conveyance (*top to bottom*)

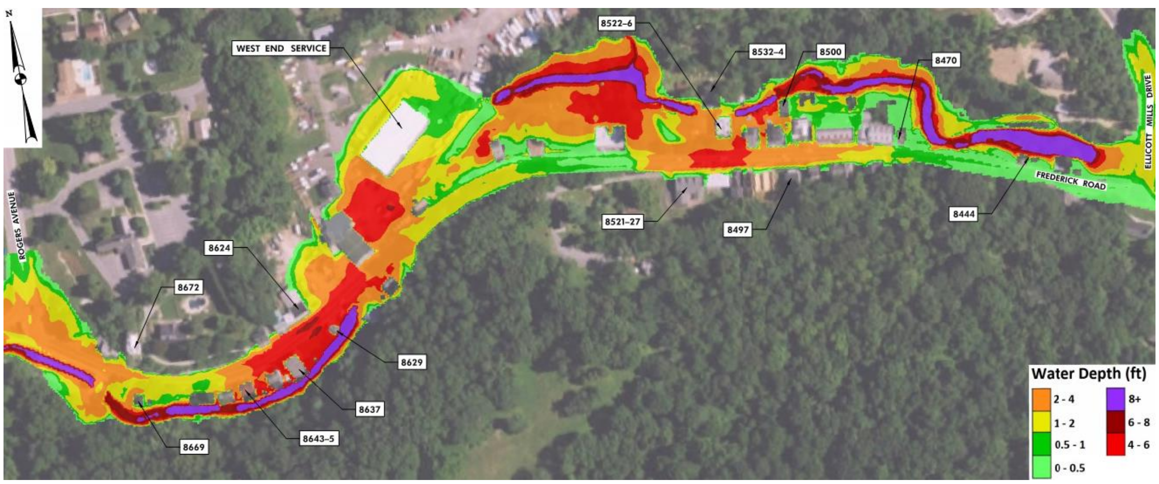


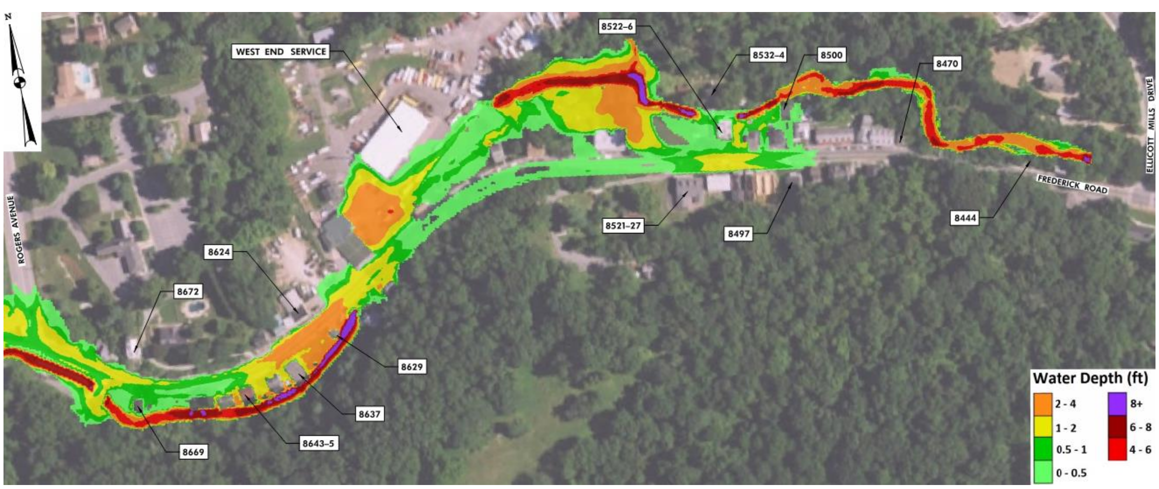
4.7.2 AREA 2 – ROGERS AVE. THROUGH WEST END TO ELLICOTT MILLS DR.

The 2’+ roadway flooding past Rogers Ave. to 8672 is now reduced by about 1’, where the stream goes under a bridge adjacent to the roadway. The significant flooding at the culvert entrance across from West End Service is reduced by 2’+ by the SWM alternatives, and an additional 1’ by adding the bypass culvert. Residential and roadway flooding from 8643 to 8629 on the south side is similarly reduced. Through the West End Service property, the flooding is reduced by SWM and eliminated by the additional bypass culvert.

Beyond the West End Service property, the 2’-5’ of flooding in the area behind 8560-48 is reduced to 1’-3’ and kept away from the residences and roadway entirely the bypass culvert/berm alternative. The flow no longer overtops Ellicott Mills Dr. in the 100-year model under both modeled improvement scenarios.

Figure 4.8: Location and Flood Area Maps of Area 2: Existing; w/ SWM Improvements; w/ SWM + Conveyance (*top, bottom, next page*).

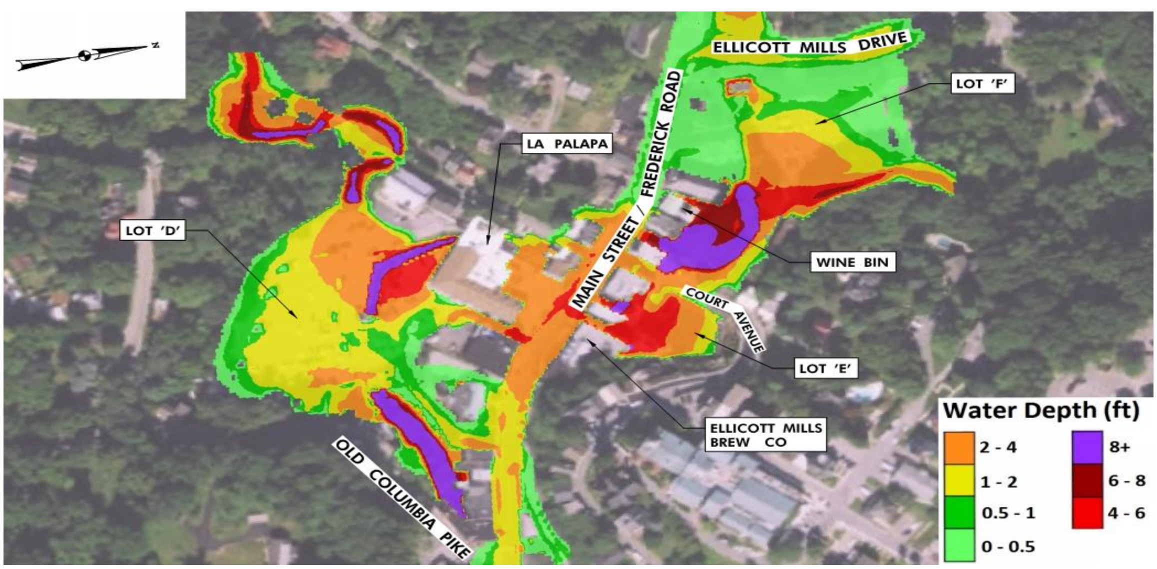


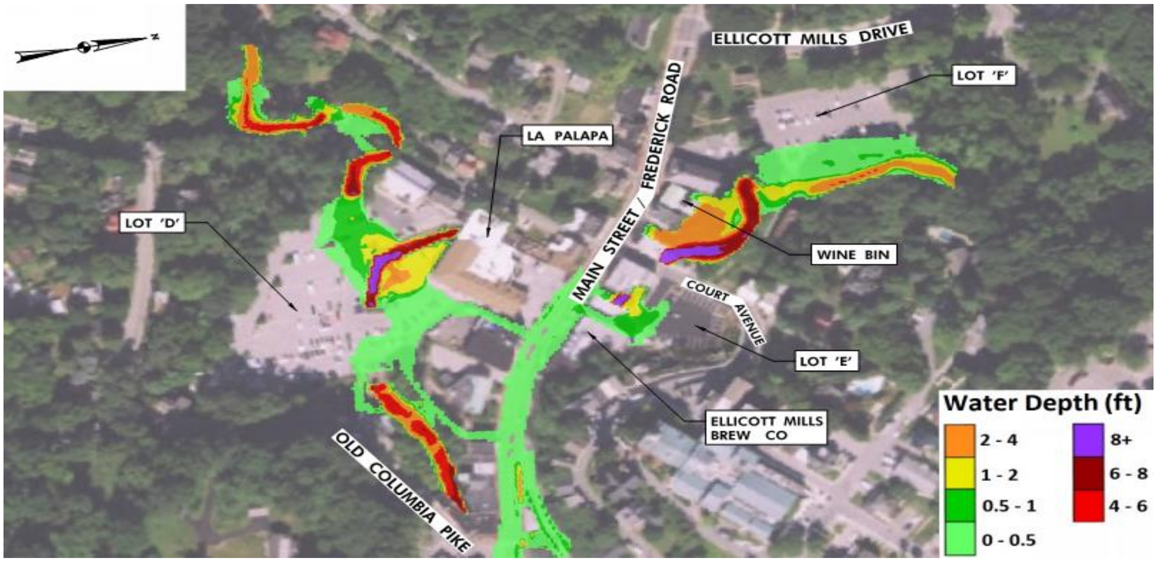


4.7.3 AREA 3 – ELLICOTT MILLS DR. TO OLD COLUMBIA PK.

The conceptual underground management at Lot 'F' stores a large portion of the water that is not already managed upstream, resulting in very little flooding at the lot when modeled. Iterative modeling has demonstrated that the underground management at this site is most effective when combined with the pipe farm storage upstream. The flood depth between the stream and the buildings along Frederick Rd. from the Wine Bin to Court Ave. (8390-8340) is reduced by up to 4'. Similar reductions of 2'-4' are seen along the south end of Parking Lot 'E' and in front of the Ellicott Mills Brewing Company. Flooding in Lot 'D' behind La Palapa is reduced by 2'-3'+. The flow down Main St. resulting from channel overflow is about 6", which is roughly a 1' reduction under the SWM concept improvements. As the conveyance improvements are upstream of this area, the effects on the model are negligible and not shown.

Figure 4.9: Location and Flood Area Maps of Area 3: Existing; w/ SWM Improvements (*below, next page*).





4.7.4 AREA 4 – OLD COLUMBIA PK. TO PATAPSCO RIVER CONFLUENCE

The flow down the steeper section of Main St. past Church Rd. is substantially reduced in depth and destructive force, as compared to existing conditions. Through the flat local sump areas, the SWM concepts reduce depth

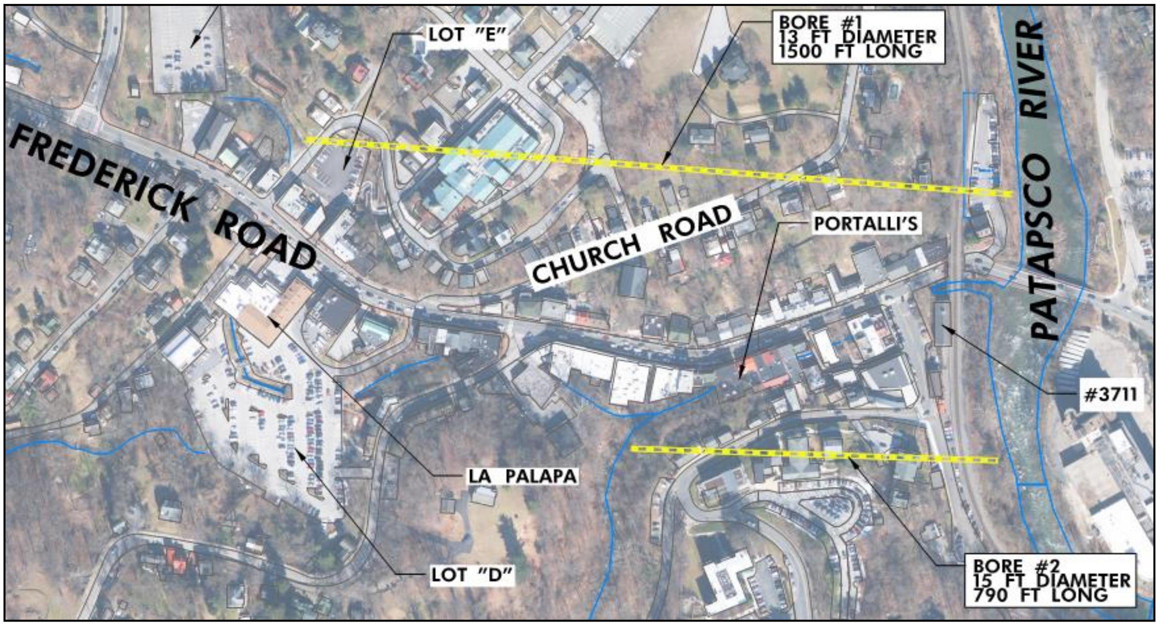
Figure 4.10: Location and Flood Area Maps of Area 4: Existing; w/ SWM Improvements (*below, next page*).

for this storm event by 2'-3'+ however, there is still a section of 4'-6' deep water that is not fully managed through this block. This area still showing over 1' of flooding also coincides with the 100-year flood backwater (elevation 133') from the Patapsco River. It is notable that this model considers flood events that generate from intense rainfall within the Tiber-Hudson watershed (3.7 mi.^{2} which is 1.3% of the 294 mi^2 Patapsco River watershed). In the event of a Patapsco River backwater flooding event (similar to T.S. Agnes in 1972) the proposed concepts will not be effective in reducing flooding from the backwater in this area, though areas upstream of the backwater will experience the reductions modeled here.

4.7.5 Tunnel Bore Improvements

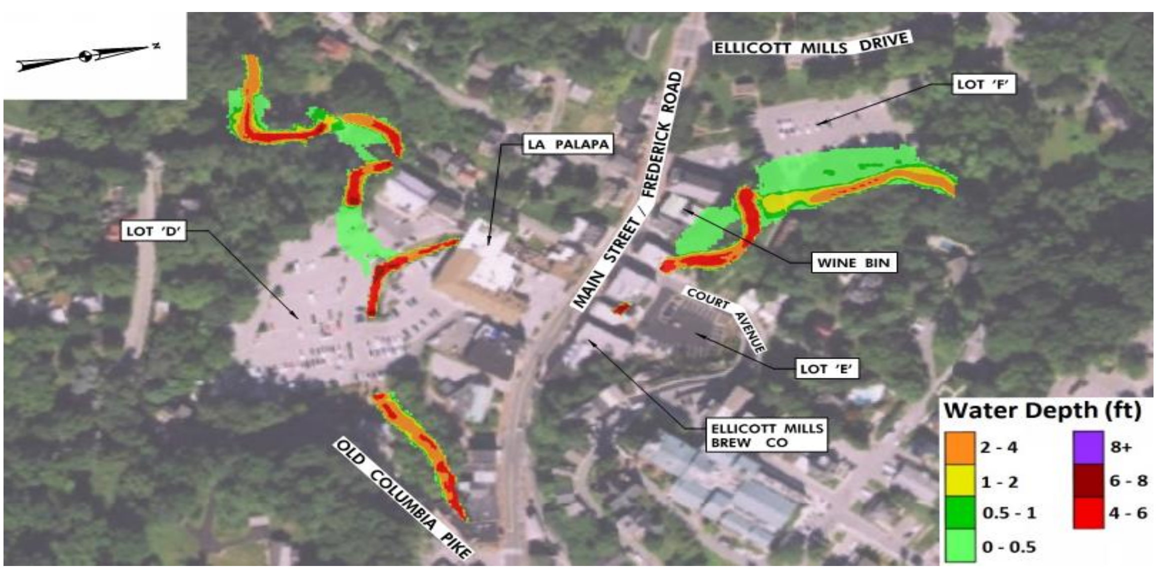
In order to consider a conceptual option that would provide full flood relief for the lower Main St. section for a 100-year event with all of the other SWM conceptual improvements in place, and to address requests made at the inception of this study from the community, the hydraulic analysis examined the concept of tunnels that would bore through the bedrock of Ellicott City in two locations to divert excess flood flows around the Main St. commercial district. Both were located in areas where the terrain goes up very steeply such that the bore would go well beneath any existing structures in the community. The first tunnel would begin upstream of Lot 'E' and would divert flood flows to the Patapsco River approximately 1300' away with a 13' diameter circular bore. The second tunnel, a 15' diameter circular bore, would capture flood flows from the New Cut Branch just upstream of its confluence with Tiber-Hudson and divert through the adjacent hillside to the Patapsco River approximately 790' away.

Figure 4.11: Location of Conceptual Tunnel Bores to Divert Flow around Main St.



The tunnel bores were sized to convey adequate flood flows such that the channel that runs under the buildings on the south side of Main St. would not overflow and flood the adjacent buildings and roadway. The resulting change in the 100-year flooding from channel capacity can be seen for Areas 3 and 4, in *Figure 4.12*. The implementation of such a system would have several challenges relative to the construction, permitting and funding of the tunnels.

Figure 4.12: Flood Area Maps of Area 3 (below) and 4 (next page) w/ Tunnel Bores



4.8 Reduction in Property Impacts

Another metric used to evaluate impact of the proposed improvements was the number of buildings within the floodplain (*Table 4.7*). Buildings within the 2-D modeling boundary that were touched by the 100-year floodplain were quantified for existing conditions and the proposed stormwater management concepts. Buildings defined for this comparison are greater than 200 square feet and may consider contiguous, row-style structures as one building; the same building shapes were used for all comparisons. This comparison was only conducted for storm events evaluated with the 2-D model.

Table 4.7 – Number of Buildings within the Floodplain under Existing Conditions and the Proposed Stormwater Management Concept

Storm Event	*Number of Buildings in Floodplain*				
	Existing Conditions	*Proposed Above Ground SWM Concepts*	*Change from Existing*	*Proposed Above & Below Ground SWM Concepts*	*Change from Existing*
10-yr	85	66	-19	56	-29
25-yr	90	81	-9	68	-22
100-yr	100	91	-9	74	-26
July 30, 2016	101	94	-7	88	-13

5.0 CONCLUSIONS AND RECOMMENDATIONS

The creation of a comprehensive hydrologic and 2-D hydraulic model of the Tiber-Hudson Branch along Frederick Rd. / Main St. east of US 29 provides Howard County with an interactive tool for long term planning and execution of strategies to reduce the probability and severity of flooding in Ellicott City. The results of this study demonstrate that construction of stormwater storage facilities throughout the watershed, combined with stormwater conveyance infrastructure improvements, can make an appreciable difference in the severity of flooding from a 100-year or other similar storm event. However, the nature and scope of such improvements is significant in scope, impact and cost. It will require a long term planning and implementation effort, supplemental to the Master Plan process, to prioritize, design and construct improvements based on the concepts represented in this report. In the shorter term, flood proofing and insurance of buildings and their contents within the floodplain should be a consideration throughout the study area.

In the interest of representing what a subset of selected improvements, of the type that would hypothetically represent the first stage of a multi-stage plan, would result in, the analysis included modeling of a subset of improvements. These SWM improvements were chosen for the subset based on their having the greatest individual impact on their respective subwatersheds in terms of peak flow reduction (see *Sections 4.1-4.3* and *Tables 4.1, 4.2*) and included T1, NC3 and H7 (ponds) and additionally H8 (Underground Pipe Farms) along with the proposed conveyance improvements (not including the tunnel bores). The mapping demonstrating the flooding reductions associated with this subset of improvements may be found in *Appendix E.*

It should be noted that these concepts, particularly those representing stormwater management and storage, are broad-brush representations of practices that can significantly vary in their final detail and location while still achieving the same improvements. The dynamic nature of the model will allow for the continued analysis of chosen alternatives as they are refined in the planning and design of future improvements associated with Ellicott City flood mitigation.

6.0 REFERENCES

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