Twinbrook / Rockcrest Watershed September 1st, 2021 Flood Study at Rock Creek Woods Apartments

Summary Report: March 31, 2022

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1.0 Executive Summary

On September 1st, 2021, Hurricane Ida passed through Rockville, Maryland, and deposited approximately 3 inches of rainfall on the Rockcrest and Twinbrook drainage areas near Twinbrook Parkway and Veirs Mill Road between 3:00 AM and 4:00 AM. Two Rock Creek tributary streams flow through these drainage areas historically known as "Twinbrook." At the Rock Creek Woods Apartments, both of these streams are encased in concrete box culverts and flow below the Rock Creek Woods Apartments' property (on both sides of Twinbrook Parkway) and discharge into Rock Creek to the Southeast. The rainfall from Hurricane Ida caused flooding of both of these streams and ultimately flooding of the Rock Creek Woods Apartments around 4:00 AM. The flooding impacted two apartment buildings, including twelve ground floor apartment units, and caused one fatality.

Mercado Consultants, Inc. was tasked by Montgomery County to conduct a flood study of the two concrete box culverts along the Rock Creek Woods Apartments' property as well as the associated Rockcrest and Twinbrook drainage areas for the flooding event that occurred on September 1st, 2021 (Hurricane Ida). As part of this task, the two box culverts were surveyed in detail to determine if there were any changes between the original, as-permitted plans and the final construction. A historical review was also performed to evaluate any previous flood studies at the flooding location and to evaluate the original design of the two box culverts. Finally, a level of drainage service (LOS) analysis was performed on the two box culverts to determine the design storm characteristics (return frequency and capacity) that overtopped the culvert embankment area just west of the apartment complex. Our sub-consultant, Water Resource Management Associates, Inc. (WRMA) assisted in developing the Hydrology and Hydraulics (H&H) model.

A detailed topographic survey was performed for the portion of the Rock Creek Woods Apartments that flooded as well as locations where the two streams crossed below roadways, pedestrian bridges, or at locations of significant storm drain outfalls. Both Twinbrook box culverts below the Rock Creek Woods Apartments were surveyed using a terrestrial lidar laser scan which created a point cloud of thousands of points of the inside surface of the box culverts. Finally, data pertaining to the existing storm drain system from the city of Rockville and Montgomery County was obtained in GIS format and surveyed or field verified at significant locations pertinent to the study.

The collected topographic survey, laser scan, and storm drain network data were used to develop a survey grade detailed Interconnected Channel and Pond Routing – version 4 (ICPR4) hydrologic and hydraulic model of the upstream catchment storm drains and open channels, through the two box culverts below the Rock Creek Woods Apartments and at the downstream floodplain of Rock Creek. ICPR4 is a state-of-the-art modelling software for hydrologic, hydraulic, and hydrodynamic 1- and 2-Dimensional modelling required for this type of post-storm event flood analysis. The Rockcrest and Twinbrook drainage catchment areas extend from approximately Veirs Mill Road to the north, Rockville Pike to the southwest, and Rock Creek to the east. Rainfall data was obtained from a local gauge on the top of the nearby Twinbrook Community Center. This gauge collected rainfall at 5-minute increments during the Hurricane Ida storm event. This real time rainfall was used as input to the ICPR4 model, and a simulation was run for the Hurricane Ida storm event of September 1, 2021. Two different ICPR4 models were created – a 1D model for evaluating the flood stage (profile view) along the streams and through both of the box culverts and a 2D model for evaluating the horizontal (plan view) distribution of detailed stormwater flow at the Rock Creek Woods Apartments in the specific areas of flooding.

The results from the simulation models indicate that the rainfall from Hurricane Ida caused flooding at the Rock Creek Woods Apartments from three distinct point-sources. First, at the Northwest corner of the Rock Creek Woods Apartment's property, there is an outfall from the storm drain system along Veirs Mill Road. This outfall discharges to a 60 foot +/- long open channel which is then directed to an inlet which conveys the stormwater underground to the Northern concrete box culvert. During Hurricane Ida, the amount of stormwater from this outfall overwhelmed the inlet and open channel leading the water to spill down the slope at the Northwest corner of the property to the low point between two of the apartment buildings. Second, as the volume of water flowing through the underground box culverts increased, the Northern box culvert began to have pressure flow within the culvert. Near the upstream end of the Northern box culvert (near the Northwest corner of the Rock Creek Woods Apartments), there is a rectangular inlet cast into the top slab of the box culvert which drains stormwater from the Rock Creek Woods adjacent parking lots. The water in the concrete box culvert was pushed (surcharged) out of this inlet as the hydraulic pressure of the water in the culvert increased, which flowed down to the same low point between two of the apartment buildings. Finally, the storm water overtopped the berm above the entrance to the Northern box culvert and flowed into the same low point between the apartment buildings. The three flooding discharges resulted from unusually high rainfall precipitation intensities associated with a sub-hourly storm frequency exceeding 300 years and occurring between 3:30 am and 4:00 am.

The two Rock Creek Woods Apartments at the Northwest corner of the property were constructed in a depression that resembles a bowl. Stormwater is conveyed out of this depression via small inlets at the low point that discharge to the same storm drain system that is downstream of the open channel and inlet in the first stage of flooding. There is no path for stormwater to naturally exit this depression without passing through this storm drain system. During Hurricane Ida, the small inlets in the depression were unable to convey the large flow of water from the three stages of flooding. This resulted in water accumulating in the depression up to 8'-10" +/- deep.

The stream that flooded the Rock Creek Woods Apartments was designated as Rock Creek Tributary No. 2 by the Federal Insurance Administration (FIA) and later by the Federal Emergency Management Agency (FEMA) in current & historical Flood Insurance Studies (FISs). Tributary No. 2 begins in the City of Rockville by Broadwood Drive, flows eastward below Ardennes Ave and Atlantic Ave, crosses the City of Rockville boundary line which is coincident with the Rock Creek Woods Apartments' West property line, enters the Northern box culvert below the Rock Creek Woods Apartments, and then exits the box culvert and flows to Rock Creek. The FIA studied this Tributary in-depth in 1978 as part of the City of Rockville FIS. This study included the upstream portions of Tributary No. 2 from Broadwood Drive and stopped at the City of Rockville boundary line. Included in the FIS study is a regulatory mapping of the 100-year floodplain. In the subsequent 1979 FIA FIS for the unincorporated portions of Montgomery County, the downstream portions of Rock Creek Tributary No. 2 from the Rockville boundary line to Rock Creek, including the culvert below the Rock Creek Woods Apartments that flooded, were not studied (in detail or by approximate methods) to establish the 100-year floodplain. The apartment complex area is shown on 1979 FIS maps as a Zone "C" or areas of minimal flood hazards. Both the City of Rockville and Unincorporated Montgomery County FIS were carried forward to the current and effective 2006 FEMA Countywide FIS. This effective FIS shows a detailed, 100-year floodplain regulatory mapping of Rock Creek Tributary 2 from Broadwood Drive in the City of Rockville to 10 feet +/- east of the Rockville Boundary Line and into Montgomery County at the entrance to the Tributary 2 culvert enclosure. However, as in the 1979 FIS, there is no detailed 100-year floodplain mapping for the remaining downstream segment of Tributary 2 to the adjacent Rock Creek flood plain. The Rock Creek Woods Apartment complex area is designated on current effective FEMA maps as unshaded Zone X or areas determined to be outside the 0.2% 500-year annual chance flooding.

A historical review was performed to evaluate the original design methodology of both concrete box culverts that flow below the Rock Creek Woods Apartments' property, and to compare their construction to the as-permitted plans. The culverts were constructed in two phases. First, culverts were constructed below Halpine Road when its horizontal alignment was relocated in 1962. At this time, the culverts only extended below the roadway and approximately 50 feet from the edge of road. Halpine Road was later renamed Twinbrook Parkway. The two box culverts constructed in 1962 consisted of a 14'-0" +/- wide by 7'-6" +/- high box culvert for the Northern stream and a 10'-0" +/- wide by 8'-6" high culvert for the Southern stream. When the Rock Creek Woods Apartments were constructed around 1967, both of these culverts were extended for several hundred feet by encasing the stream in a new 10'-0" +/- wide by 5'-0" +/- high box culvert that connected to either end of the existing culverts. The grade of these extended box culverts was steeper than the grade of the existing box culvert constructed directly below Halpine Road (Twinbrook Parkway).

Based on review of the design plans for the two original box culverts constructed below Halpine Road (Twinbrook Parkway), it was determined that the Talbot methodology was used to size the waterway hydraulic opening. For a roadway with the size and traffic of Halpine Road (Twinbrook Parkway), most municipalities and engineering consultants in the 1960's applied the Talbot methodology and typically provided a culvert hydraulic opening capable of conveying a 10 year storm drainage level of service. This study confirmed the original 10-year LOS design intent. Detailed calculations could not be obtained for the extension of these culverts around 1967 during the construction of the Rock Creek Woods Apartments; however, the culvert extension and original box culverts below Halpine Road (Twinbrook Parkway) were designed by the same engineering firm. Given the limited information available, it is believed that the same hydrologic watershed and peak flow data used with the Talbot hydrology method would also have been used to size the culvert extensions waterway opening. Hydraulically, while smaller than the original box culverts below Halpine Road (Twinbrook Parkway), the 10’-0” +/- wide by 5’-0” +/- high box culvert size would have been appropriate using Manning’s full flow methodology to convey a similar level storm as the original Halpine Road (Twinbrook Parkway) since the culverts are on steeper grades. However, despite few available design tools in the 1960’s, the designer could have checked with readily available Bureau of Roads inlet/outlet control headwater nomographs and determined that certain storm events would induce high inlet control headwater conditions that could overtop the selected culvert design conditions. The lack of original concrete box culvert design data prohibits a conclusive interpretation of the culvert extension design intent below the Rock Creek Woods Apartments’ property.

Final as-built plans were not available for the 10’-0” +/- wide by 5’-0” +/- high box culverts below the Rock Creek Woods Apartments, however, a plan set approved by WSSC permitting was obtained (WSSC had permitting authority of storm drain culverts in the 1960s). Based on this as-permitted set and compared to the laser scan: the hydraulic opening of the culvert was constructed according to the as-permitted plans, the horizontal alignment was constructed to within 4½” of the as-permitted plans alignment, and the vertical alignment was constructed to within 3½” of the as-permitted plans alignment when adjusted for the current NAVD datum.

An analysis was conducted to determine the current level of service of both culverts using The U.S. Department of Commerce National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Precipitation-Frequency point data for design storm durations of 6 and 24 hours. This analysis was performed in ICPR4 using the same model that evaluated the Hurricane Ida flooding with real time rainfall storm event data (i.e., the model calibrated to known flood stages). The storm intensity was iterated until overtopping occurred above the upstream berms above the upstream openings of both culverts. The Northern culvert (culvert that flooded during Hurricane Ida) has an overtopping level of service of a 13 year storm event and the Southern culvert has an overtopping level of service of a greater than a 100 year storm event based on a NOAA 24-hour design storm. The Northern culvert has an overtopping level of service of a 38 year storm event and the Southern culvert has an overtopping level of service of a greater than a 100 year storm event based on a NOAA 6-hour design storm. This is indicative of an imbalance in peak discharge loads for both Rock Creek tributaries.

The modeling assessment also included a recreation of the original HEC-2 model used for the determination of the 1978 City of Rockville Flood Insurance Study 100-year flood stages depicted in regulatory floodway maps, which were later incorporated without changes into the current effective Montgomery County Countywide 2006 Flood Insurance Study. This study verified that the HEC-2 Backwater Water Surface Profile model developed for the original City of Rockville FIS in 1978 was not modified in subsequent FIS updates for Montgomery County. Thereby, the original regulatory 100-year floodplain of 1978 has been historically carried over without change into the current effective 2006 Montgomery County FIS. This study further verified that the Rock Creek Tributary 2 culvert enclosure was not modeled in 1978 or thereafter by FIA or FEMA contractors.

2.0 Introduction

On September 1st, 2021, Hurricane Ida passed through Rockville, Maryland, and deposited approximately 3 inches of rainfall on the Rockcrest and Twinbrook drainage areas near Twinbrook Parkway and Veirs Mill Road between 3:00 AM and 4:00 AM. Two Rock Creek tributary streams flow through these drainage areas. At the Rock Creek Woods Apartments, both streams are encased in concrete box culverts and flow below the Rock Creek Woods Apartments' property and discharge to Rock Creek to the Southeast. The rainfall from Hurricane Ida caused flooding of both of these streams and ultimately flooding of the Rock Creek Woods Apartments around 4:00 AM. The flooding impacted two apartment buildings including twelve ground floor apartment units and caused one fatality.

At the request of Montgomery County, Mercado Consultants, Inc. and subconsultant WRMA were tasked to investigate how the storm event interacted with the area surrounding the apartment complex to recreate the storm event (using hydrologic and hydraulic modeling), analyze the sources of flooding (including the hydraulics of the concrete box culverts that convey flow below the Rock Creek Woods Apartments' property), and review historical documentation on the construction and permitting of the storm drain system and culverts in the area around the Rock Creek Woods Apartments.

This report summarizes our review of the history of the construction and permitting of the area of flooding, methods of collecting data for the study, a comparison of actual and as-permitted conditions, an overview of the hydraulic and hydrologic analyses performed, and results detailing the causes of flooding.

3.0 Background

The Rock Creek Woods Apartments are in the Rockcrest and Twinbrook drainage areas of the City of Rockville (see Figure 1). Figure 2 shows the major points of interest in the area of the study. Two Rock Creek tributaries (denoted Tributary No. 1 and Tributary No. 2 from the current FEMA Flood Insurance Study) are encased in two box culverts and flow from west to east below the apartment complex, connecting at the culverts' outfall, and feeding into Rock Creek. Tributary 1 carries water from the Twinbrook drainage area located approximately to the South of the Rock Creek Woods Apartments along Twinbrook Parkway, while Tributary 2 carries water from the Rockcrest drainage area located approximately to the West of the Rock Creek Woods Apartments between Veirs Mill Road and Rockville Pike. The City of Rockville municipal boundary follows the West property line of the apartment complex and is generally parallel to Twinbrook Parkway. The boundary is located approximately ten feet west of the entrance to the Tributary 2 concrete box culvert below the Rock Creek Woods Apartments.

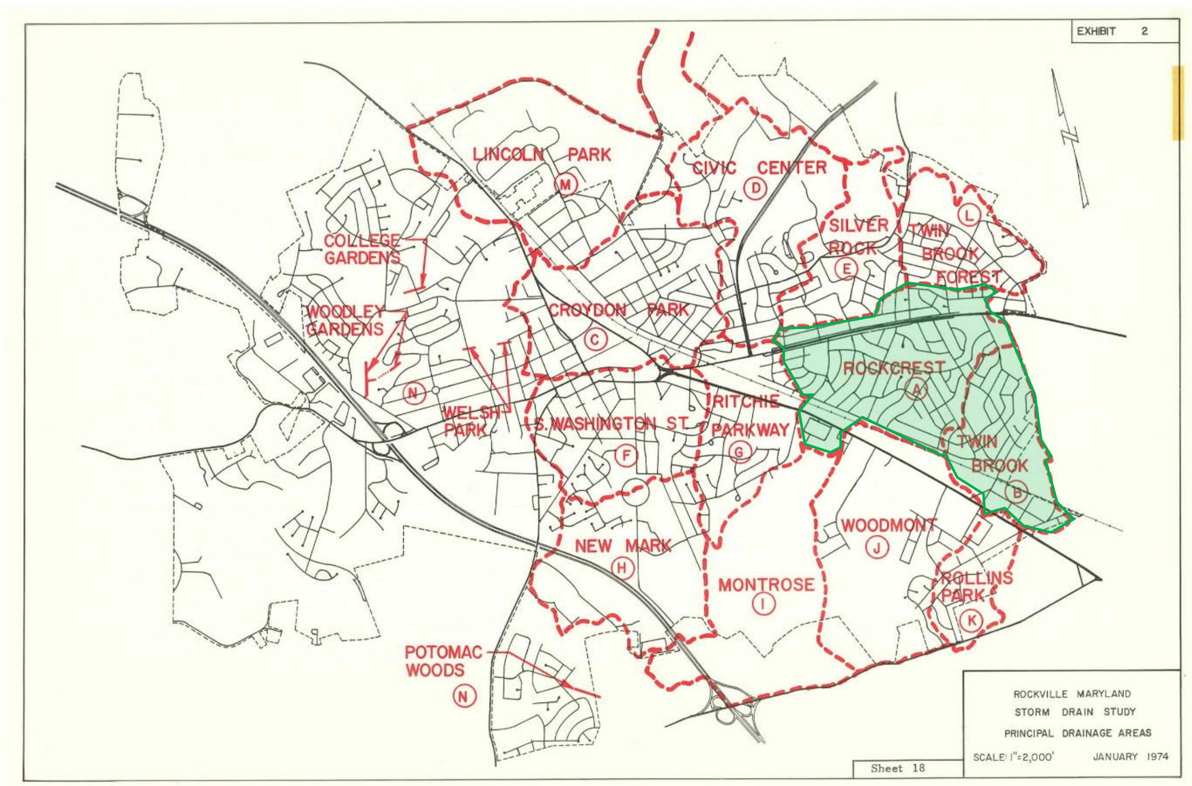

Figure 1 - City of Rockville Drainage Areas (1974 Rockville Storm Drain Study). Tributaries 1 and 2 are in the Rockcrest and Twinbrook drainage areas, respectively (green highlight).

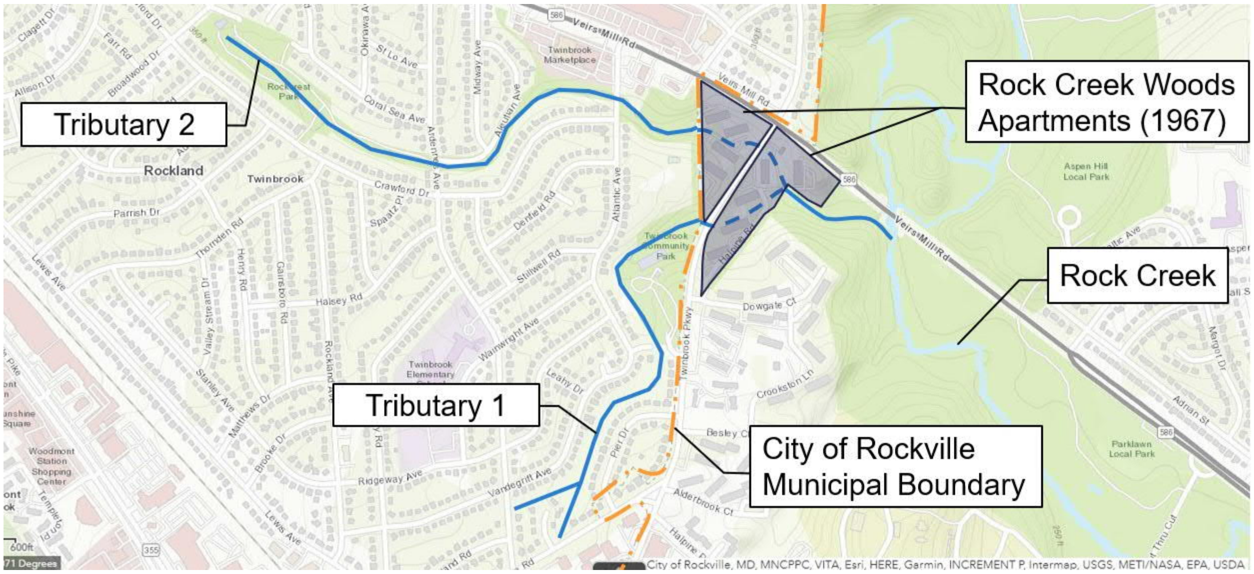

Figure 2 - Topographic view of the Twinbrook area with locations of note labeled. Tributary 1 and Tributary 2 each enter a box culvert below the Rock Creek Woods Apartments (blue dashed lines) and connect at the culvert outfall.

Overall, the land use and impervious area in the Rockcrest and Twinbrook drainage areas has not changed significantly since 1970. In the 1970 aerial imagery in Figure 3, the Rock Creek Woods Apartments are visible, as are the commercial properties along Rockville Pike and Veirs Mill Road, and the residential neighborhoods throughout the area. Noted changes in the 2020 imagery are the addition of the Twinbrook Metro station which opened in 1984, and the construction or renovation of high-density residential properties near the Southern end of Twinbrook Parkway (see Figure 4).

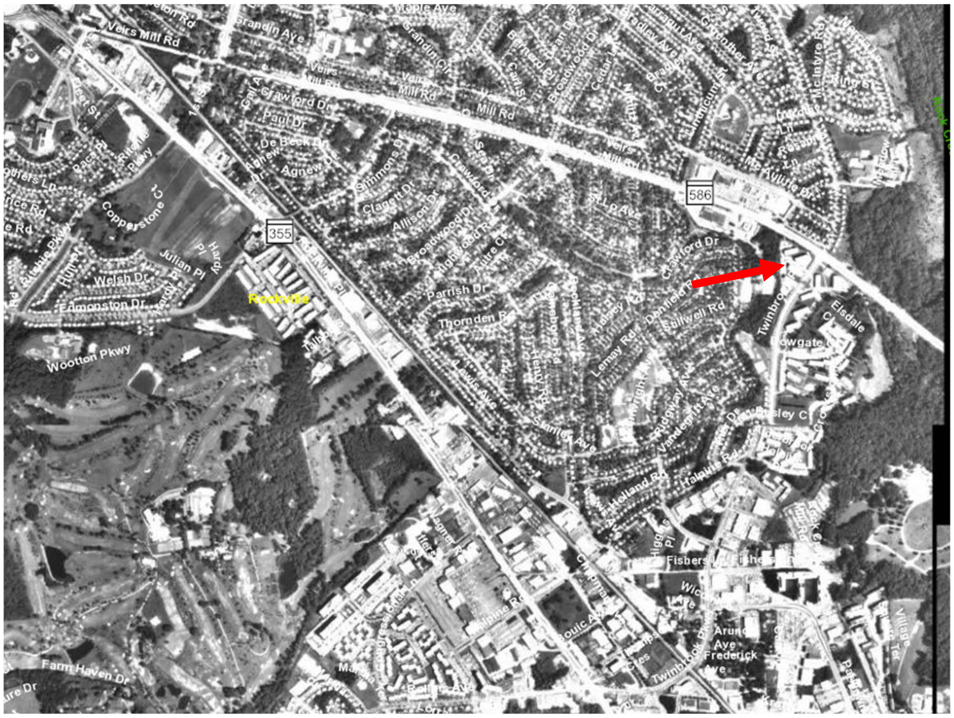

Figure 3 - City of Rockville Aerial Imagery (1970). Note the location of the Rock Creek Woods Apartments (arrow).

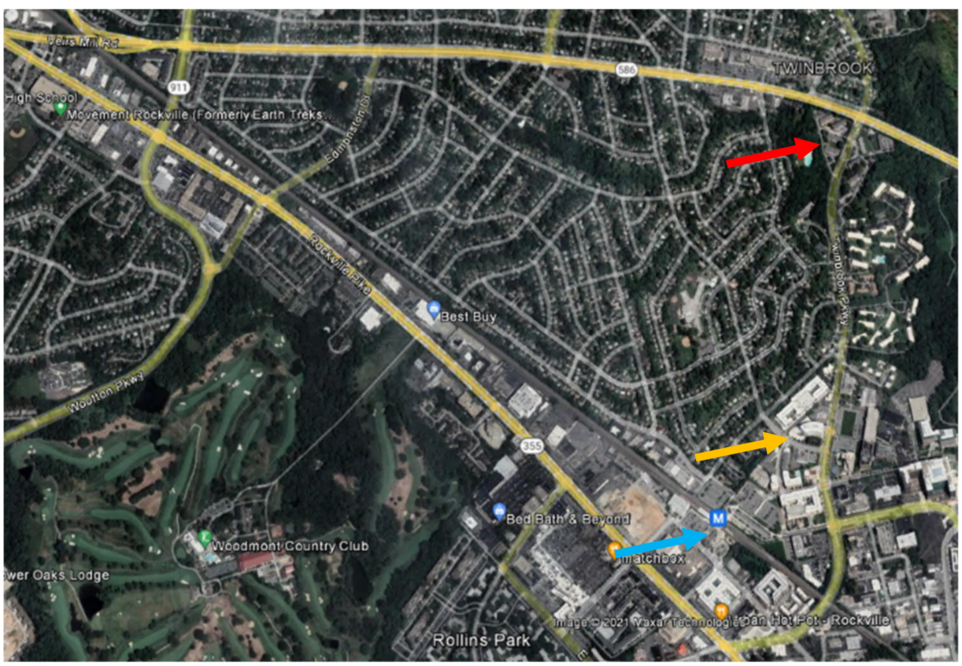

Figure 4 - City of Rockville Aerial Imagery (2020). Note the location of the Rock Creek Woods Apartments (red arrow), Metro station (blue arrow), and high-density residential properties (orange arrow).

3.1 Timeline of Site Development

A 1955 topographic survey documents the two tributaries that run through the current-day Rock Creek Woods Apartment property and connect to Rock Creek.[1] The tributaries are also recorded in a 1959 survey of "Prevention Tract" along the former location Halpine Road, near the "Old Location of Veirs Mill Road." A 1964 plan set outlines the relocation of a portion of the historic Halpine Road to the current-day Twinbrook Parkway and includes the diversion of the two tributaries through two concrete box culverts beneath the new roadway.[2] The two box culverts extended below the roadway and approximately 50 feet from the edge of road. A section of each tributary was also altered at the culvert outlet with the installation of grass-lined and riprap-lined channels before reconnecting with the existing stream bed.

At the intersection of Twinbrook Parkway with Veirs Mill Road, there are two land parcels, one on either side of Twinbrook Parkway, which were recorded in a 1966 land conveyance[3]. This document outlines two storm drain easements which contain the two tributaries that run through the 12.21-acre property. The City of Rockville owns the land immediately upstream of the culvert (approximately ten feet from the culvert headwall), and the Maryland National Capital Park and Planning Commission (MNCPPC) owns the land immediately downstream of the culvert outfall. WSSC approved a set of storm drain permit plans on May 5, 1966[4] depicting the construction of culverts to further encase several hundred feet of each of the two tributaries across the length of the property. These culverts connect with the two previously constructed box culverts that run beneath Twinbrook Parkway (formerly Halpine Road), and later join at an outfall to the southeast of the property. As-built construction documents were not available at the time of our review. WSSC approved a revision to the storm drain layout on April 17, 1967[5].

This storm drain project made way for the 1967 construction of the Rock Creek Woods Apartment complex which contains a total of 270 residential units across nine garden-style apartment buildings. Each building typically contains four levels, with the lowest level partially below grade. The apartments currently provide market-affordable rental housing for households who earn less than 80% of the area median income. As-built construction documents of the apartment complex buildings were not reviewed as part of this study.

Adjacent to the subject property, the City of Rockville conducted a project to improve the Twinbrook Parkway and Veirs Mill Road intersection between 1982 and 1984[6]. This

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[1] Maddox & Hopkins, *Topographic Rock Creek Park units – 6 & 7*, February 1955

[2] Greenhorne & O'Mara Consulting Engineers, *Relocated Halpine Road, Rockville Pike to Veirs Mill Road*, Revised April 13, 1964

[3] Bradshaw, Harold M., Sterling R. Maddox & Associates, *Parcels A & B Twinbrook Hills Plat 8304*, Approved July 1, 1966

[4] Greenhorne & O'Mara Engineers, *Windham District Storm Drain* Revised May 23, 1966

[5] Greenhorne & O'Mara Civil Engineers Land Surveyors, *Storm Drainage Extension - Rock Creek Woods Apartments,* February 1967

[6] Montgomery County Maryland Department of Transportation, *Channelization and Paving Plan Twinbrook Parkway and Veirs Mill Road Intersection*, 1983

project included widening and reconfiguring the intersection but did not include changes to the property lines or storm drain system of the apartment complex.

In 2019, a memorandum[7] summarizing the meeting of the Montgomery County Planning, Housing, and Economic Development (PHED) Committee on February 25, 2019, included record that the committee recommended rezoning the apartment complex to allow for high-density residential redevelopment of the site as part of the Veirs Mill Corridor Master Plan. This memo also states that the owners of the Rock Creek Woods apartment complex supported this rezoning measure and desired to redevelop the property, in part because of challenges with the aging and/or deteriorated utility infrastructure.

3.2 History of Drainage and Floodplain Studies in the Rockcrest & Twinbrook Drainage Areas

Figure 5 shows a timeline of available drainage studies in the Twinbrook project area.

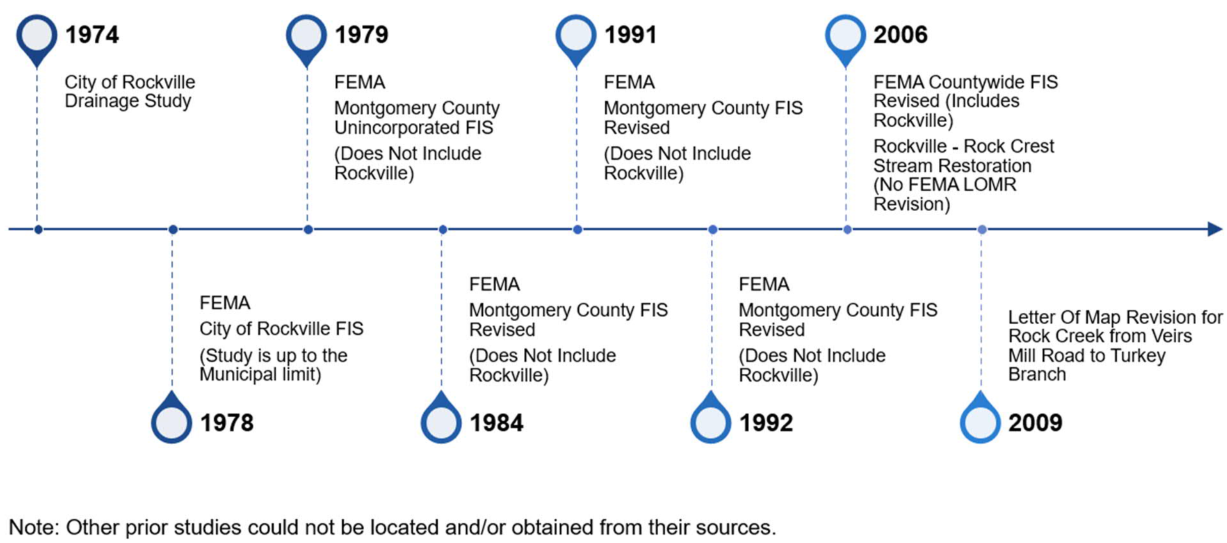

Figure 5 - Timeline of Drainage Studies in the Twinbrook Project Area

In 1974, the City of Rockville performed a drainage study of the City watersheds including the Twinbrook and Rockcrest catchment drainage areas corresponding to Rock Creek Tributaries 1 and 2, respectively. Tributary 2 was studied up to the municipal boundary of the city which is located approximately ten feet upstream of the entrance to the Tributary 2 culvert, below the Rock Creek Woods Apartments (see Figure 2). The National Flood Insurance Program (NFIP) issued initial Flood Insurance Study (FIS) reports for the City of Rockville and for unincorporated Montgomery County in 1978 and 1979, respectively. Revised FIS reports were issued for Montgomery County in 1984, 1991, 1992 and 2006.

[7] Dunn, Pamela, *Veirs Mill Corridor Master Plan*, February 21, 2019

The 2006 Montgomery County Countywide FIS, with an effective date of September, 2006 is the current FIS report used for floodplain regulation by the Federal Emergency Management Agency (FEMA) and Montgomery County.

In 2006, the City of Rockville implemented a stream restoration project at Rock Crest Park, along Tributary 2, and immediately upstream of the Rock Creek Woods Apartments[8]. Improvements included regrading, bank stabilization, outfall protection, and sediment control measures. Although a Stream Restoration Report was prepared, no Letter of Map Revision (LOMR) was issued for any changes to the FEMA regulated flood plain following the restoration project.

In 2009, the Army Corps of Engineers performed a flood study of Rock Creek from Veirs Mill Road to Turkey Branch[9]. A LOMR revised the flood plain around Rock Creek, with backwater effects forming the flood plain downstream of the intersection of Tributary 1 and 2 culvert outfalls. The study did not include analysis of the Tributary 1 and 2 culverts. To date, no studies are available or on record that have analyzed the hydraulics of the Tributary 1 and 2 box culverts below the Rock Creek Woods Apartments property.

3.3 FEMA Flood Insurance Study Floodway Determination for Project Area

3.3.1 The National Flood Insurance Program

The National Flood Insurance Program (NFIP) is a program created by the Congress of the United States in 1968 through the National Flood Insurance Act of 1968 (P.L. 90-448). The NFIP was created with two main purposes: to share the risk of flood losses through flood insurance and to reduce flood damages by restricting floodplain development.

Participation in the NFIP is based on an agreement between local communities and the federal government that states that if a community will adopt and enforce a floodplain management ordinance to reduce future flood risks to new construction in Special Flood Hazard Areas (SFHA), the federal government will make flood insurance available within the community as a financial protection against flood losses. The intent of the NFIP is to reduce future flood damage through community floodplain management ordinances and provide protection for property owners against potential human life or property losses.

The NFIP enables property owners in participating communities to purchase insurance protection, administered by the government, against losses from flooding, and requires flood insurance for all loans or lines of credit that are secured by existing buildings, manufactured homes, or buildings under construction, that are located in the Special Flood Hazard Area (SFHA) in a community that participates in the NFIP. The participating communities must adopt, by resolution, adequate land use and control measures with

[8] Biohabitats, Inc., *Rock Crest Park Stream Restoration,* June 9, 2006.
[9] U.S. Army Corps of Engineers, *Floodplain Revision for Rock Creek Near Parklawn Cemetery,* May, 2009.

effective enforcement provisions to reduce flood damages by restricting development in areas exposed to flooding.

The chronology of the NFIP implementation in Maryland and in the project area, in particular, is as follows:

- 1968: The National Flood Insurance Act of 1968 launches the NFIP with two primary goals: reducing future flood damage and protecting property owners.
- 1973: The Flood Insurance Protection Act of 1973 requires the purchase of flood insurance for some homeowners in high-risk flood zones.
- 1978: The City of Rockville, Maryland, by resolution, agrees to meet the requirements of the NFIP and was accepted for participation in the program on January 5, 1978, the initial effective date of the City of Rockville Flood Insurance Study and Rate Maps.
- 1979: Montgomery County, MD (unincorporated areas) joins the NFIP with an effective date of July 2, 1979, the initial effective date of the Montgomery County Flood Insurance Study and Rate Maps. Additional maps became effective during the following years as additional flood hazard areas were studied in detail.
- 1979: Executive Order 12127 officially makes the NFIP part of the Federal Emergency Management Agency (FEMA).

3.3.2 Flood Insurance Study

FEMA determines a community's risk to flood hazards, by performing a Flood Insurance Study (FIS) of the community watersheds. An FIS is a compilation and presentation of flood hazard areas along rivers, streams, coasts, and lakes within a community. An FIS is based on:

- Historic information (such as river, stream flow, storm tide, and rainfall data)
- Meteorologic data
- Topographic data
- Hydrologic data
- Hydraulic data
- Land use conditions
- Flood-control works
- Urban, suburban development

The result of the FIS is a technical report and flood areas of special flood hazard shown on FEMA’s Flood Insurance Rate Maps (FIRMs). The date of the FIS publication, as approved by FEMA and published in the Federal Register, is also the Effective date for FIRMs and associated 1% Base Flood (100-year) information used to regulate development in SFHA’s or regulated floodplains.

Before the results of an FIS are shown on a legally adopted FIRM, there are certain procedural steps that a FIRM goes through as part of the adoption process. These include:

- *Final Consultation Coordination Officer (CCO) Meeting:* Meeting at which the preliminary results of a FIS are reviewed and discussed with community officials.
- *Appeal Period:* The statutory 90-day appeal period begins on the date of the second publication of the notice of proposed flood hazard determinations in the local newspaper, during which community officials and individual residents may appeal the proposed flood hazard information.
- *Final Letter Sent:* The Letter of Final Determination (LFD) is sent to the community to establish the FIRM and FIS report effective date and initiate a formal sixth-month period during which the community must adopt the FIRM and FIS report to become or remain eligible for participation in the National Flood Insurance Program (NFIP).
- *Effective Date:* The date on which the FIRM and FIS report for a community goes into effect and the flood insurance and floodplain management requirements based on the new map apply.

3.3.3 Floodplain and Floodway Delineation

FEMA defines the floodplain as the area that would be flooded by a base flood, which is "the flood which has a one percent chance of being equaled or exceeded in any given year." A base flood is synonymous with a 100-year flood, and a floodplain is synonymous with a Special Flood Hazard Area (SFHA).

The base flood is used in the NFIP to indicate the minimum level of flooding to be used by a community in its floodplain management regulations. FEMA regulatory floodplains and SFHAs are critical determinations made by a FEMA engineering consultant evaluating the community's watersheds, historical flooding storm events, rainfall patterns, river stream flow data, topography, wind velocity, tidal surge, flood control measures, building development (existing and planned) and community maps.

44 CFR § 9.4 defines parts of the floodplain as follows, "Floodway means that portion of the floodplain which is effective in carrying flow, within which this carrying capacity must be preserved and where the flood hazard is generally highest, i.e., where water depths and velocities are the greatest. It is that area which provides for the discharge of the base flood so the cumulative increase in water surface elevation is no more than one foot."

Flood Fringe means that portion of the floodplain outside of the floodway (often referred to as "floodway fringe"). Figure 6 is a schematic representation of FEMA's floodplain, floodway and flood fringe representation.

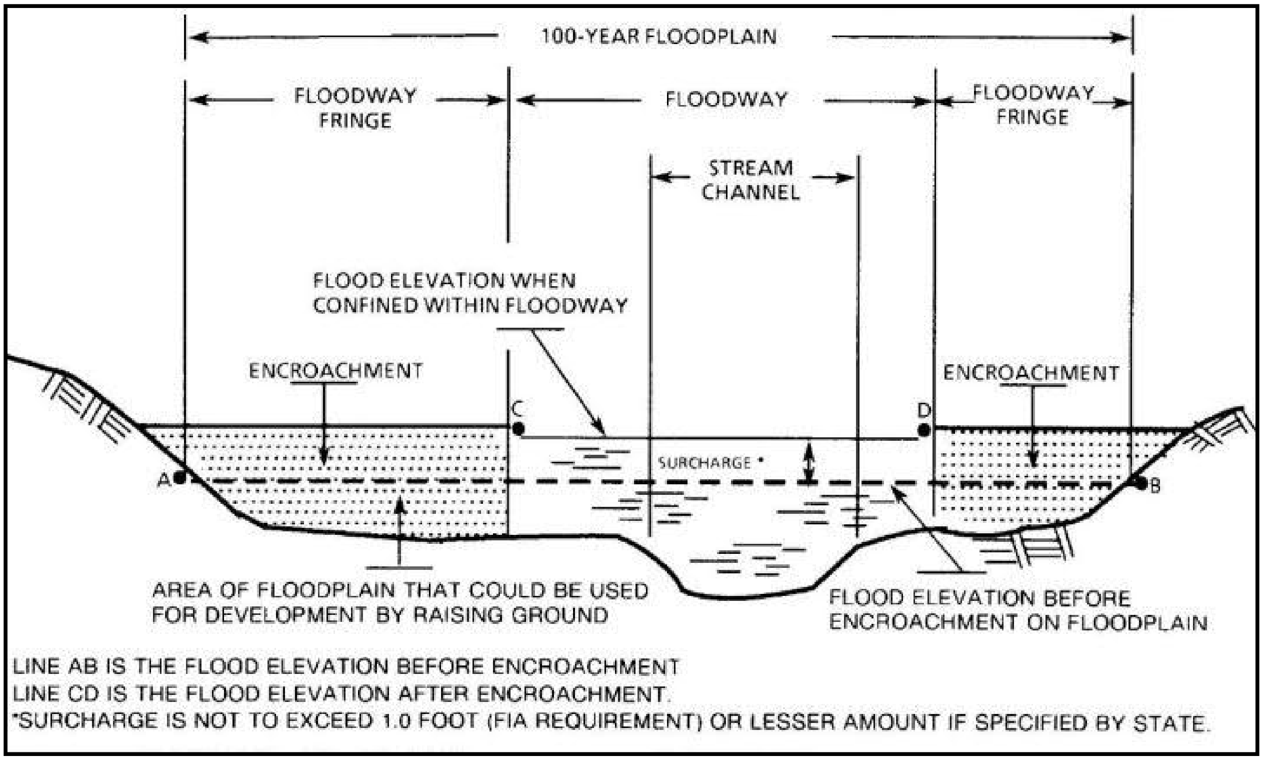

Figure 6 - Typical Cross Section of Floodplain/Floodway Delineation (Source: FEMA, Guidance for Flood Risk Analysis and Mapping)

Surcharge: NFIP regulations allow up to a one-foot rise in flood stage when designating the floodway. If development occurs outside of the floodway in the floodway fringe and there is an increase in flood stage, there will be an increase in potential flood damages to adjoining and upstream property. Some communities with intense urban development may select to adopt a more restrictive floodway (surcharge less than one foot) to prevent an increase in damages.

Most municipalities, including Montgomery County, joined the NFIP in the 1970's. Upon joining, most communities entered the "Emergency" phase in which regulatory responsibilities were limited because of the limited flood hazard data. At that time, they were provided with a Flood Hazard Boundary Map (FHBM). FHBM's, initially prepared to provide flood maps to communities in a short period of time, were based on approximate studies (i.e., no hydrologic or hydraulic modeling) and showed only the approximate boundaries of the floodplain. They were intended for interim use until more detailed studies could be carried (as it was the case with Montgomery County).

Upon completion of the FIS report, the associated SFHA mapping, and after the six month review period, the community was converted to the "Regular" phase. The SFHA is the area where the NFIP's floodplain management regulations, as adopted by the participating member, must be enforced by the community as a condition of continuing participation.

Prior to 1986, communities received a Flood Boundary and Floodway Map (FBFM) as a component of the FIS for each community studied. The FBFM maps showed the floodplain divided into the floodway and flood fringe where streams were studied in detail. Base flood 100-year water surface elevations (BSE's) were provided at selected cross section locations to identify the 100-year flood stage for regulation and also to represent different insurance zones.

The base flood computed water surface elevation (BFE) is the elevation to which floodwater is anticipated to rise during the occurrence of a 100-year design storm event in the Flood Insurance Study) watershed area. The FIS computes BFEs based on two types of analysis.

- Approximate Methodology: Base floodplain elevations determined from available/historical flood stages, no hydrologic or hydraulic modeling performed.
- Detail Methodology: Hydrologic/hydraulic studies performed using best available computer model methodology (HEC-2, WSPRO, WSP2, etc), topographic (USGS 7.5 Quads, surveying), soils (SCS) and development (land use) data.

The engineering studies included a floodway encroachment analysis whereby the delineated floodplain area was encroached by a maximum one-foot rise elevation to establish the limits of the floodway. The floodway is the restricted portion of the floodplain reserved for the safe passage of the 100-year flood. No development is allowed in the floodway unless it can be proved via detail H&H modeling that zero rise in BSE's occur at the location of encroachment.

Original (Vintage 1970's) Flood Insurance Study zones included the following designations:

- A: The base flood plain determined by approximate methods (i.e., BFE were not determined)
- A1-A30: Base floods for which detailed methods were applied to determine BFE's
- AO: Base floodplain with sheet flow, ponding or shallow flooding with Base Flood Depths provided.
- AH: Shallow flooding base floodplain. BFE's provided.
- A99: Area to be protected from base flood by levees or federal flood protection systems under construction. BFE's not determined.
- AR: Base floodplain resulting from the decertification of a previously accredited flood protection system in the process of being restored to provide a 100-year or greater level of flood protection
- V: Coastal area subject to a velocity hazard where BFE's were not determined.
- B: Areas of moderate flood hazard usually the area between the 100 and 500-year floods.
- C: Areas of minimal flood hazard
- D: Areas of undetermined but possible flood hazards

Flood Insurance Study reports in the 1970's included:

- An appraisal of the communities' flood problems including historic floods, areas of flood hazards, and the methodology applied for hydrologic/hydraulic analyses.
- A Vicinity map of the community and pictures of historical floods
- Tables summarizing peak discharges for specific drainage areas
- Tables for Floodway Data including Flooding source (cross section along flooding source, distance), Floodway (width, section area, mean velocity), and Base Flood Water Surface Elevation (w/floodway, w/o floodway, difference).
- Tables for Flood Insurance Zone Data: Base Flood Elevation, Elevation difference between the 100-year and the 10, 50 and 500-year flood, Zone computed flood profiles for the 10, 50, 100 and 500-year recurrence floods. Flood profiles included the stationing along the stream bed of key features such as culverts and bridges (showing the low chord and top of roadway), and roadway crossings.

3.3.4 FIRM's and Countywide Studies

After 1986, the NFIP selected to replace the Flood Hazard Boundary Map (FHBM) with a Flood Insurance Rate Map (FIRM). Floodways and other floodplain management information, such as cross sections that were previously provided on separate Flood Boundary and Floodway Maps were no longer being prepared. Additionally, simplified flood insurance zones were introduced. The previous Zones A1-A30 and V1-V30 were replaced by designations AE and VE, Zones B and C were replaced by Zone X. The 500-year floodplain was shown as shaded portions of Zone X.

The Countywide FIS report covers the unincorporated areas of the specific county and all/some incorporated areas within the county. Previously, FHBM, FBFM, and FIRMs were prepared separately for each jurisdiction. The Countywide FIS mapping shows all of the identified flood hazard areas within the boundaries of the county on one set of maps along with all the floodway information maps. The Countywide format offers a number of advantages. The user can see the relationship and simultaneously the effect of each floodplain on a number of communities regardless of jurisdictional boundaries. The Countywide title block lists the communities mapped on that panel along with the six-digit community ID. All previous map dates for each community in a County wide FIS are located on the community map history table. The initial FIRM date for each community is shown on the FIRM index.

3.3.5 City of Rockville Flood Insurance Study

The hydraulic and hydrologic analyses for the City of Rockville FIS were completed by Dalton, Dalton, Newport and Little (DDNL) in 1977 for the Flood Insurance Administration (FIA) under contract # H-3810. Areas studied in detail included Rock Creek Tributaries 1 and 2. The initial FIS map products became effective on January 5, 1978.

The 1978 Rockville FIS report indicated that flooding in Rockville had been historically of no consequence primarily because the City was located on a ridge between drainage basins. The exception was a small tributary to Rock Creek (Tributary 1) along the eastern boundary with Montgomery County. It was noted that the lots along Pier Drive extended to the Tributary 1 overbank area effectively blocking storm conveyance efficiency by a multitude of fences, garages, and sheds. The resulting loss of conveyance resulted in minor flooding during storm events. Figure 7 shows the City of Rockville 1978 FIS Table 1 Summary of Peak Discharges.

TABLE 1 - SUMMARY OF DISCHARGES

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-YEAR	50-YEAR	100-YEAR	500-YEAR
CABIN JOHN CREEK					
Downstream Corporate Limit	2.2	1,377	1,988	2,406	3,830
Above Junction of Tributary No. 4	1.11	1,034	1,486	1,743	2,700
West Lynfield Drive (Extended)	0.21	233	321	376	583
TRIBUTARY NO. 1					
Downstream End of Detailed Study	0.39	540	696	781	1,060
TRIBUTARY NO. 2					
Downstream End of Detailed Study	0.74	1,041	1,447	1,714	2,540
Cross Section J	0.27	408	567	672	996

Figure 7 - Summary of Peak Discharges (Source: City of Rockville FIS, 1987)

Figure 7 indicates that drainage areas of 0.39 and 0.74 square miles were used to calculate peak discharges for the culvert design of Tributaries 1 and 2, respectively. Likewise peak discharges of 1041 cfs for the 10-year flood, 1447 cfs for the 50-year, 1714 cfs for the 100-year and 2,540 cfs for the 500-year were used in the floodway calculation analysis.

The 1978 Rockville FIS also indicates that peak discharges were computed using the Snyder method, which uses a modified Rational Formula approach. The 500-year peak discharge was obtained by plotting the 10, 25, 50 and 100-year peak flows on log-probability paper and extrapolating to the 500-year frequency.

Flood elevations for the selected recurrence intervals were calculated for each stream using the US Army Corps of Engineers HEC-2 standard step-back water surface profiles computer model[10].

An important element of the backwater profile computation is a determination of the beginning point of the initial water surface elevation. Normally, the initial point corresponds to the downstream location of confluence with a waterbody with known water

[10] US Army Corps of Engineers, The Hydrologic Engineering Center, Generalized Computer Program, HEC-2, Water Surface Profiles, Users Manual, October 1973.

surface profile elevations for the 10-, 25-, 50-, and 100-year flood stages. In this case, the initial water surface elevation for the Twinbrook Rock Creek Tributaries 1 and 2 would be provided by the corresponding Rock Creek flood stages downstream of the Viers Mill Road bridge over Rock Creek. However, at the time of the City of Rockville FIS, the corresponding Montgomery County FIS was underway and the water surface profile elevations for the 10-, 25-, 50-, and 100-year flood stages were not yet available.

Alternatively, to establish a beginning point water surface profile elevation, the FEMA consultant, DDNL, applied an iterative method of computation in which the initial estimate of the energy slope and water surface elevation at the beginning cross section are successfully adjusted with respect to a downstream section until the true energy gradient and water surface elevations are computed for the beginning cross section within a specified limit of tolerance. In this method one survey section at a suitable downstream location is required. Figure 8 shows the Tributary 2 mapped floodplains and the location of the initial water surface elevations for the Tributary 2 HEC-2 model at the City/County boundary line.

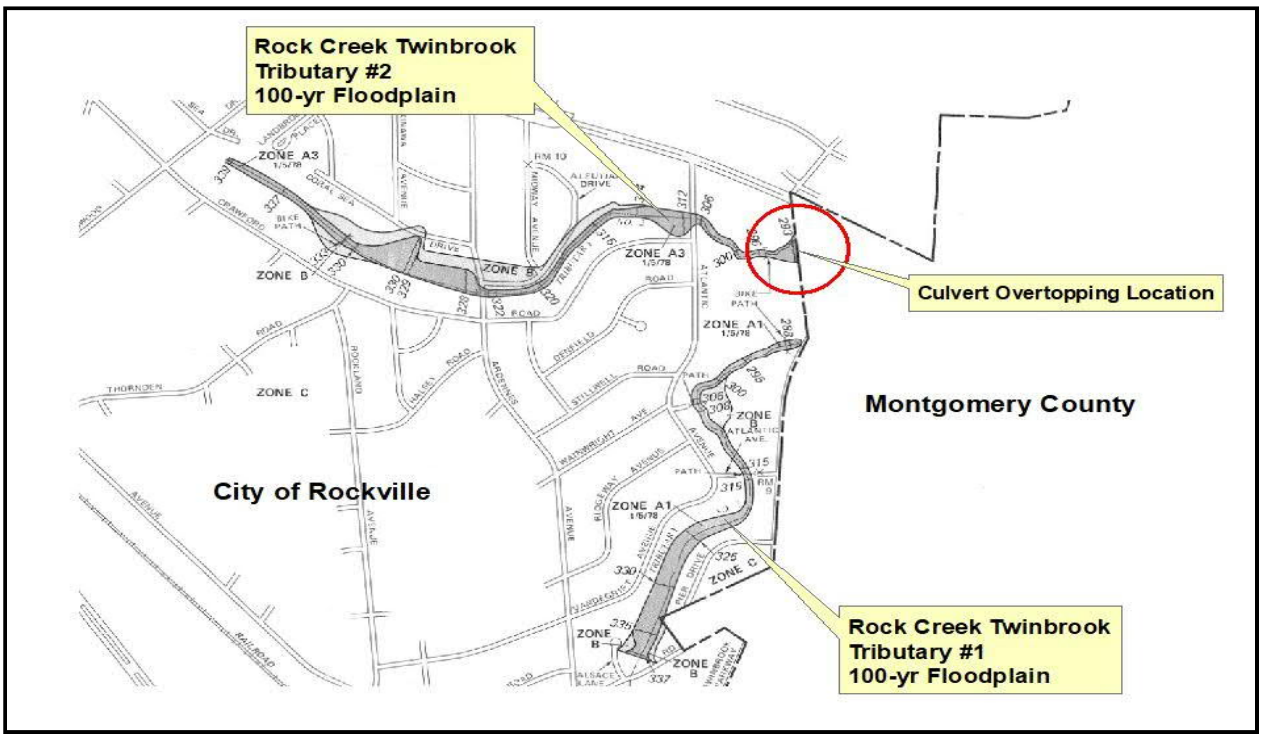

Figure 8 - Twinbrook Tributaries 1 and 2 Floodplain Boundaries, Annotated (Source: City of Rockville 1978 FIS)

Figure 8 shows that 100 and 500-year floodplain boundaries for Tributaries 1 and 2 were mapped to the Montgomery County boundary. A Zone A3 with a Base Flood Elevation of 293 Feet (NGVD 29) was designated along the boundary just west of the culvert entrance (Located 10 feet +/- east of the jurisdictional boundary line within Montgomery County).

Figure 9 shows the profile plot of the 10-, 25-, 50-, and 100-year flood stages (NGVD 29).

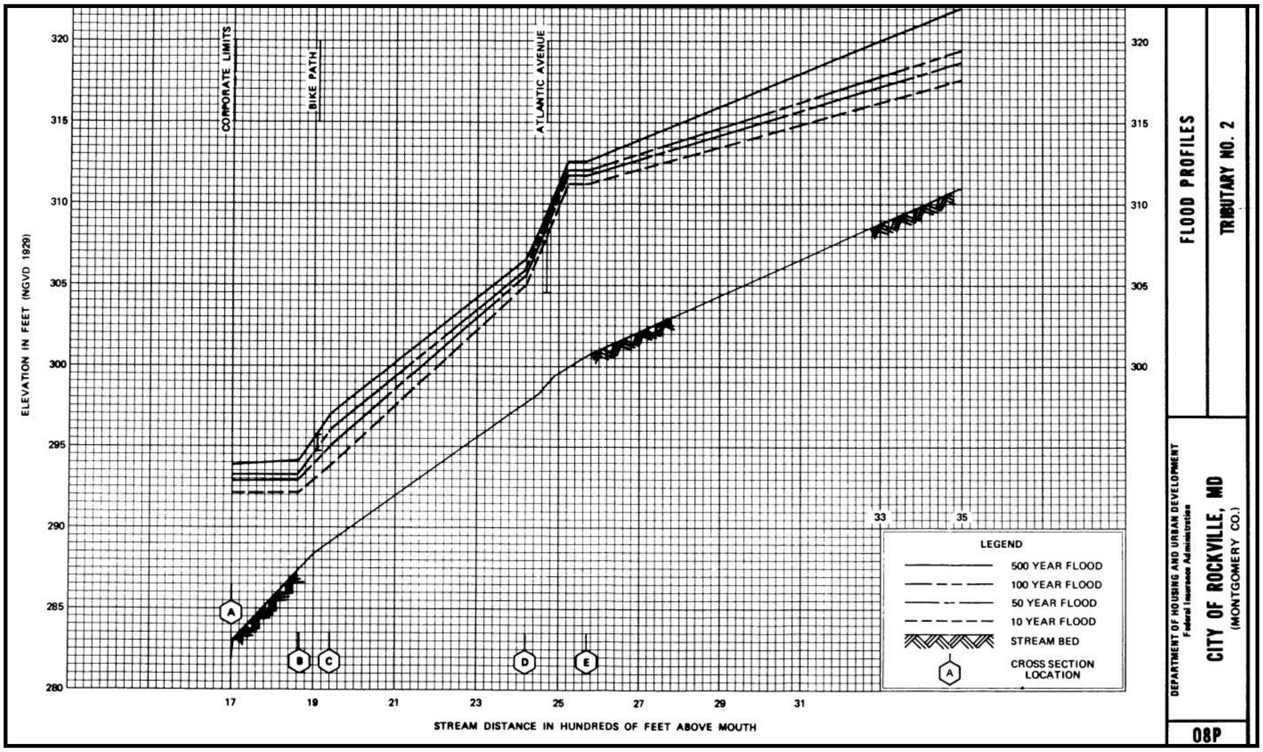

Figure 9 - City of Rockville 1978 FIS Water Surface Elevation Profile Plot for Tributary 2

Figure 9 indicates that for Tributary 2, cross section A, located along the County boundary is the first cross section of the HEC-2 Backwater model. As previously stated, the 10, 25, 50 and 100-year flood initial water surface elevations were computed per a surveyed downstream cross section that most likely was located at the floodplain of Rock Creek. The HEC-2 model initial water surface elevations at cross section A were listed as follows (NGVD 29 datum):

- 10-year = 292.2 feet
- 25-year = 293.0 feet
- 50-year = 293.3 feet
- 100-year = 293.8 feet

Note: The area of flooding of the Rock Creek Woods apartments is approximately 10 feet +/- downstream of cross section A in the Tributary 2 profile plot. The plan set showing the box culvert construction below the Rock Creek Woods Apartments' property shows a berm above the upstream headwall 10 feet +/- from the Rockville Municipal Boundary at Elevation 292.00 (NGVD 29), which is below the 10-year water surface elevation (see Figure 10).

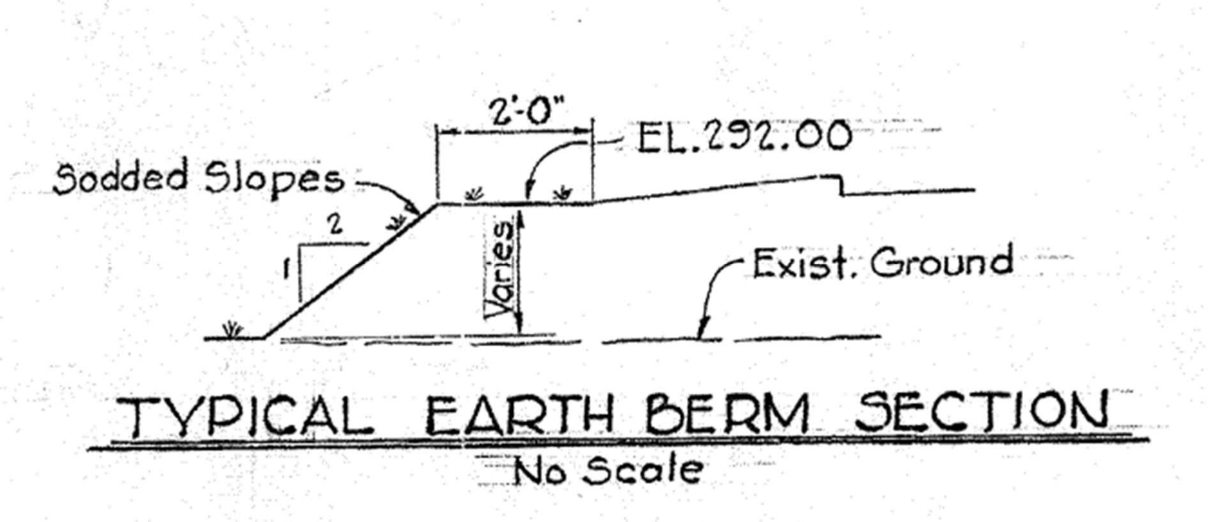

Figure 10 - Berm above upstream headwall of box culvert conveying Tributary 2 below Rock Creek Woods Apartments' Property. Elevation is in NGVD 29 datum.

3.3.6 Montgomery County Flood Insurance Study

3.3.6.1 Historic Effective Flood Insurance Studies

The hydraulic and hydrologic analyses for the Montgomery County FIS were completed by Dalton, Dalton, Newport and Little (DDNL) for the Flood Insurance Administration (FIA) under contract # H-3810. DDNL (and/or its subconsultants) coordinated with the Montgomery County Department of Environmental Protection, the Montgomery County Department of Transportation, and the Maryland National Capital Park & Planning Commission for the provision of technical data and reports to undertake studies of the initially designated watersheds with high hazard flood areas. Initial SFHA maps became effective July 2, 1979.

The July 2, 1979 FIS included the unincorporated areas of Montgomery County and excluded the many incorporated Towns including the City of Rockville. Figure 11 shows the delineated Rock Creek floodplain south of the Viers Mill Road bridge. The Rock Creek Tributary 2 is depicted as a stream line connecting to the Rock Creek floodplain.

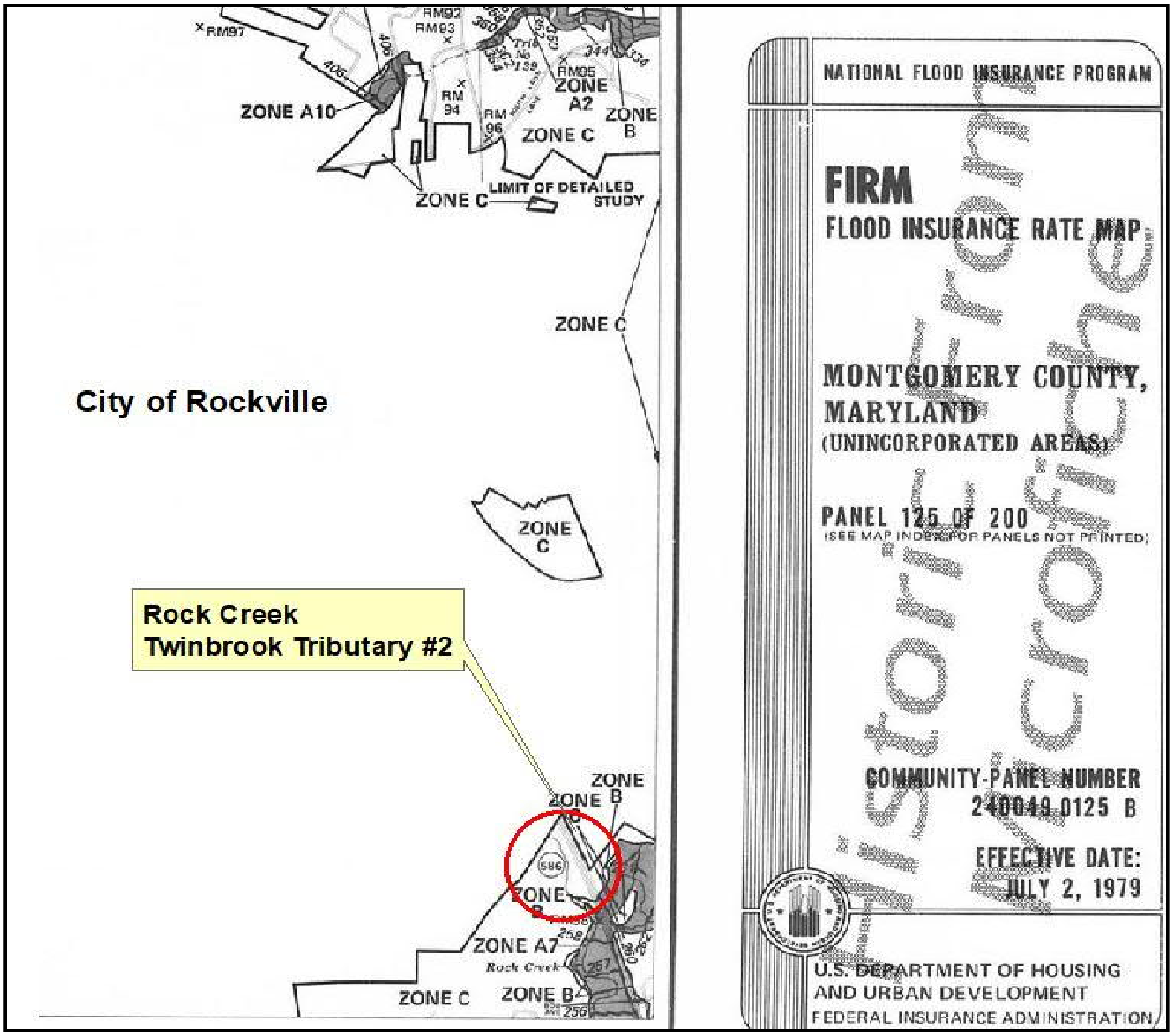

Figure 11 - Montgomery County July 2, 1979, FIRM Panel 125 of 200

Additional Flood Hazard Boundary Maps were subsequently prepared and published with effective dates in August 1988 and 1992. Figure 12 shows the delineated Rock Creek floodplain south of the Veirs Mill Road bridge as mapped in the 1992 FIS. As with the 1979 FIS, the Rock Creek Tributary 2 is depicted in the 1992 FIS as a stream line connecting to the Rock Creek floodplain.

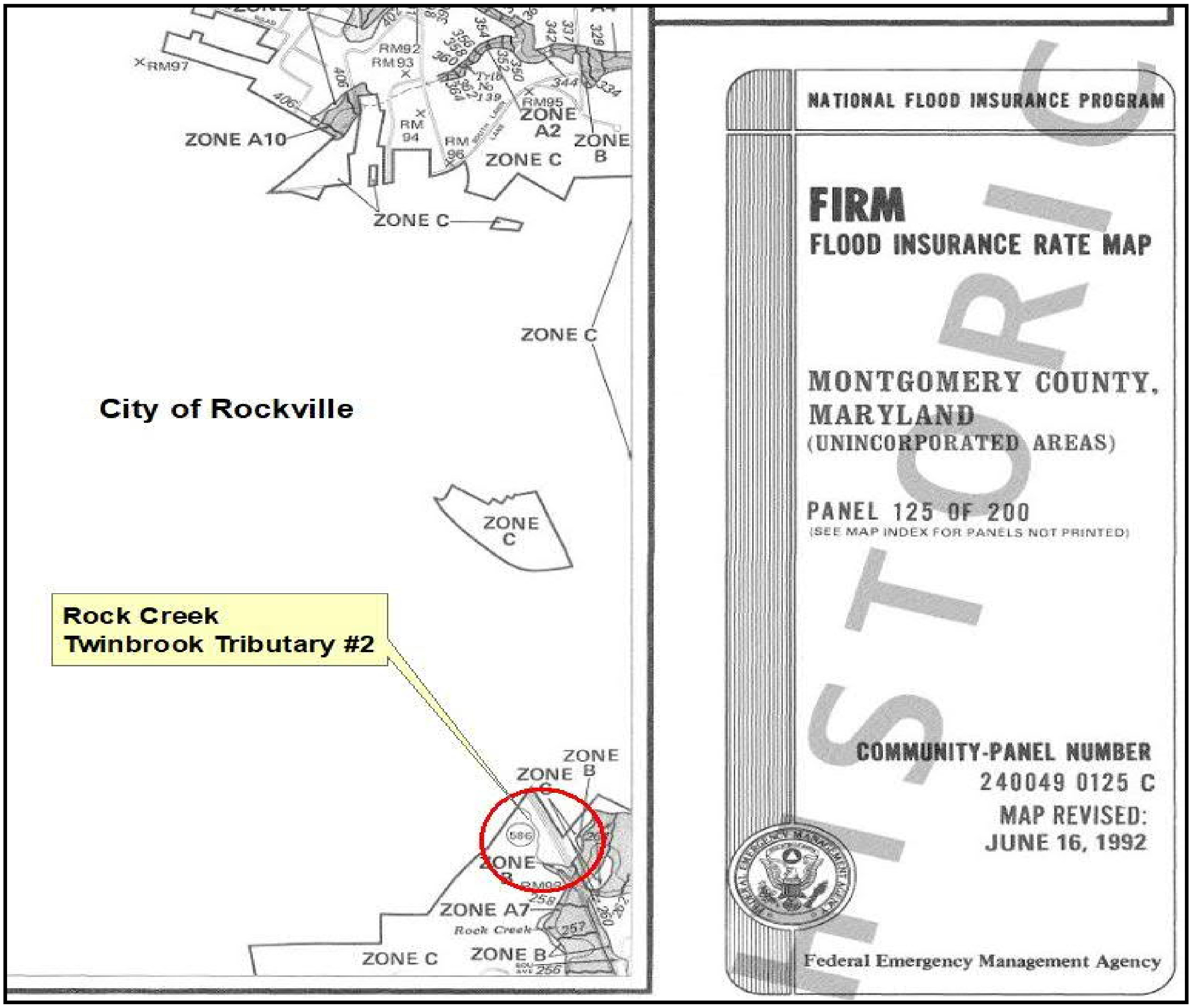

Figure 12 - Montgomery County June 16, 1992, FIRM Panel 125 of 200

3.3.6.2 Current Effective Flood Insurance Studies

The current Montgomery County regulatory floodplains were established with the adoption of the 2006 Countywide Flood Insurance Study for Montgomery County, Maryland and Incorporated Areas on September 29, 2006. The 2006 FIS revises and supersedes the previous FIS reports and/or Flood Insurance Rate Maps, and Flood Boundary and Floodway Maps adopted by the FIA.

The "Countywide" 2006 Montgomery County Flood Insurance Study report # 24031CV001A contains maps and tables of the consolidated floodplains of all the incorporated communities listed in the FIS report including the City of Rockville (Community #2140051). The Rock Creek Tributaries 1 and 2 floodplain areas are shown in Panel 353 of 480 of the 2006 FIS (Figure 13).

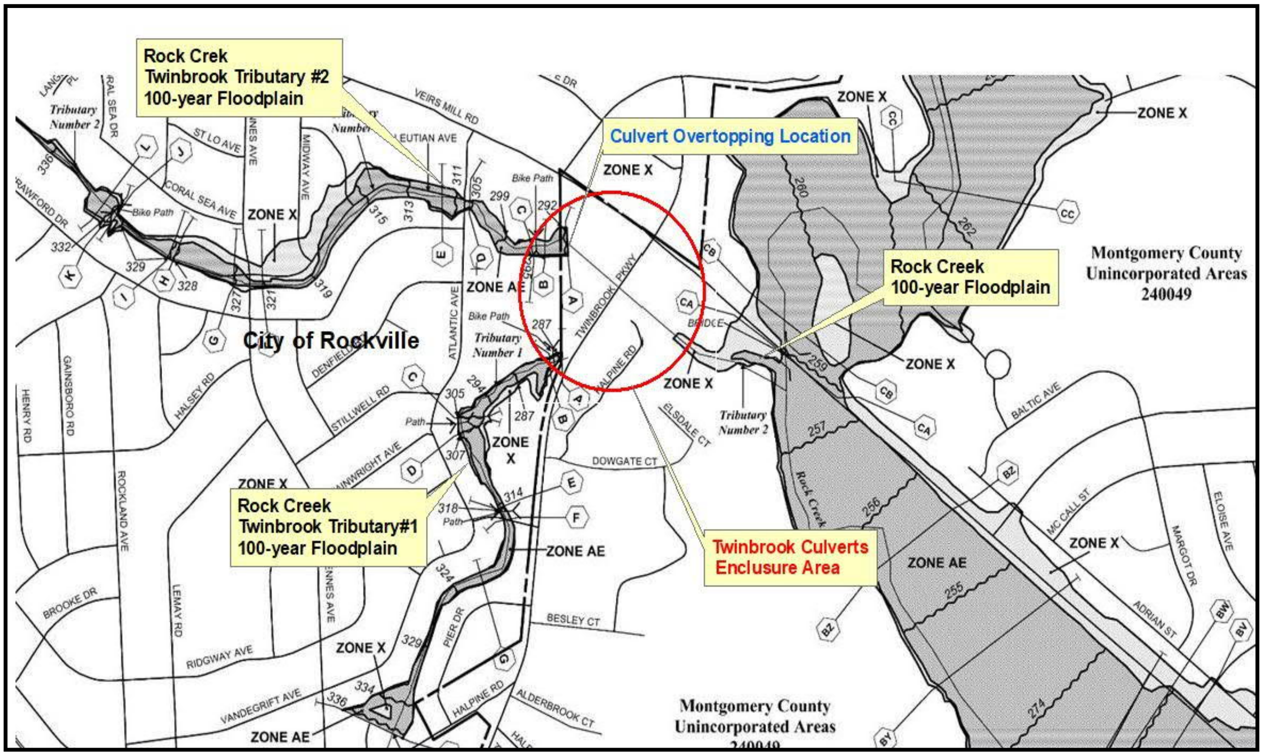

Figure 13 - Montgomery County September 29, 2006 FIS, FIRM Panel 353 of 480

Figure 13 indicates that the Rock Creek Tributaries 1 and 2 enclosure area is labeled as unshaded Zone X (Areas determined to be outside the 0.2% 500-year flood annual chance floodplain).

FIS Flood Profile sheet 190P presents the Rock Creek Tributary 2 profile for the 10-, 25-, 50-, 100 and 500-year floods water surface elevations as a function of the streambed, roads, corporate limits and HEC-2 model cross section locations (Figure 14).

The Figure 9 and Figure 14 profile views of the Rock Creek Twinbrook Tributary 2 water surface elevations are identical when the datum is converted from NGVD29 (used in 1978 profile) to NAVD88 (used in 2006 profile). This indicates that the project area was not restudied with updated HEC-RAS models (HEC-RAS had replaced HEC-2 in 2006) for the Countywide FIS preparation.

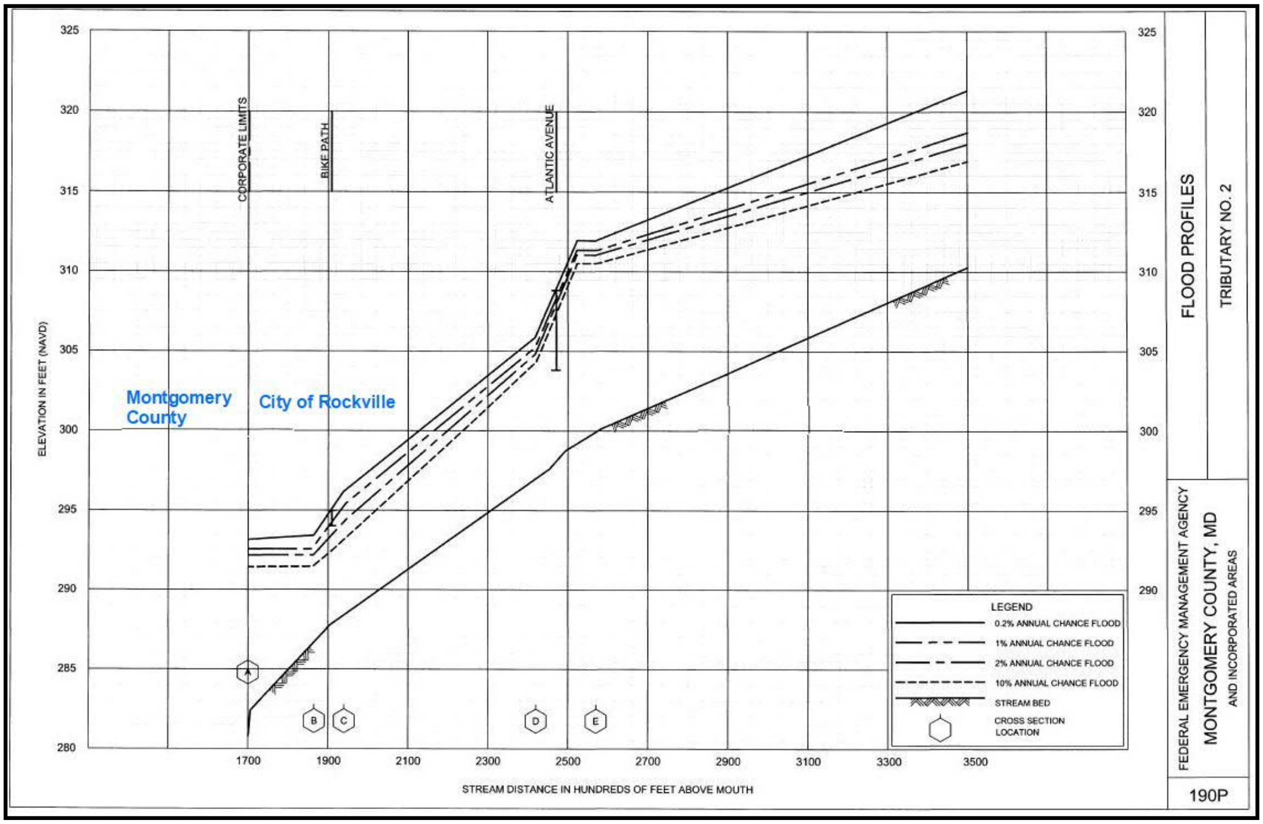

Figure 14 - Montgomery County September 29, 2006 FIS, Flood Profile Sheet 190-P (NAVD 88 Datum)

During the preparation of the 2006 Countywide FIS, a categorical methodology was developed for the adoption and mapping of floodplains resulting from previous studies of the participating incorporated communities in the 1970's, 1980's and 1990's. These were:

- *Category A*: Streams with no records of past flood studies
- *Category B*: Streams previously studied by requiring restudy due to substantive changes in hydrologic or hydraulic conditions.
- *Category C*: Streams previously studied with published reports which, upon review by DDLN for methodology and content, met HUD guidelines and criteria for accuracy and sufficiency.
- *Category D*: Streams for which a study is currently under way by a Montgomery County agency with review by DDLN of the methodology criteria and specifications indicated in the agreement with HUD. Streams in this category were subcontracted by DDLN to the study contractor for the county agency (Montgomery County) in the interest of establishing exact agreement between study conclusions, and of economy, in order to eliminate duplication efforts.

The City of Rockville 1978 FIS fell within Category D. Per this category, the 2006 Countywide FIS selected to incorporate the City of Rockville 1978 FIS floodplains "as is" without further studies to extend the Rock Creek Tributary 2 floodplain further east into

Montgomery County to the junction with Rock Creek (i.e., Modeling of the Tributaries 1 and 2 box culvert enclosure as a function of the Rock Creek backwater).

Figure 15 shows that the 2006 FIS-calculated 100 and 500-year floodplains were mapped beginning approximately 10 feet east of the City of Rockville/Montgomery County boundary or to the location of the 10-ft x 5-ft concrete box culvert entrance. A Zone AE with a base flood elevation of 292.0 feet NAVD 88 is shown for the mapped floodplain.

Figure 15 - Montgomery County September 29, 2006, FIS FIRM Panel 353 of 480 (Close Up)

Figure 16 is a copy of "Table 2-Floodway Data" from the 2006 Montgomery County FIS for Tributaries 1 and 2. It indicates that a regulatory 1% annual chance flood water surface elevation at cross section A (Tributary 2 culvert entrance) was calculated as 292.6 feet NAVD 88. The table also includes a notation (1) that cross section A was located 1700 feet above the mouth or at the junction with Rock Creek. Note that the topographic survey conducted in this study found the top of berm at cross section A above the Tributary 2 box culvert headwall was Elevation 291.07 (NAVD88)[11].

[11] Refer to section **4.4.1 Culvert Inspection** for surveyed culvert dimensions.

FLOODING SOURCE		FLOODWAY			1-PERCENT-ANNUAL-CHANCE-FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY (FEET NAVD)	WITHOUT FLOODWAY (FEET NAVD)	WITH FLOODWAY (FEET NAVD)	INCREASE (FEET)
Tributary No. 1 (continued)								
D	2,615	23	88	8.92	307.1	307.1	307.1	0.0
E	3,090	35	87	8.95	314.2	314.2	314.2	0.0
F	3,160	40	160	4.87	318.4	318.4	318.5	0.1
G	3,550	40	105	7.44	322.0	322.0	322.0	0.0
H	4,625	40	172	4.53	336.4	336.4	337.0	0.6
Tributary No. 2								
A	1,700	41	381	4.50	292.6	292.6	293.6	1.0
B	1,865	47	206	8.34	292.3	292.3	293.3	1.0
C	1,940	37	192	8.95	295.2	295.2	295.2	0.0
D	2,420	67	210	8.18	305.1	305.1	305.1	0.0
E	2,570	76	411	4.17	311.2	311.2	311.3	0.1
F	3,840	21	124	13.83	321.3	321.3	321.3	0.0
G	4,000	73	300	5.72	327.0	327.0	327.0	0.0
H	4,350	36	202	8.48	328.3	328.3	328.4	0.1
I	4,370	91	377	4.55	329.2	329.2	329.3	0.1

[1]Feet above mouth

[2]Shown on flood boundary and floodway map as 50 feet

TABLE 2

FEDERAL EMERGENCY MANAGEMENT AGENCY

MONTGOMERY COUNTY, MD
AND INCORPORATED AREAS

FLOODWAY DATA

TRIBUTARY NO. 1 - TRIBUTARY NO. 2

Figure 16 - Table 2-Floodway Data from the 2006 Montgomery County FIS for Tributaries 1 and 2

3.3.6.3 2009 Montgomery County FIS Letter of Map Revision (LOMR)

As Figure 17 indicates, in 2009 a Letter of Map Revision (LOMR) to the Montgomery County 2006 FIS was prepared by the US Army Corps of Engineers (USACE) and processed by FEMA. A LOMR report titled “Floodplain Revisions for Rock Creek near Parklawn Cemetery, May 2009” was submitted to FEMA on May 14, 2009.

A review of the technical report indicates that the purpose of the study was to complete a floodplain revision for a portion of Rock Creek, in Montgomery County, Maryland on behalf of FEMA and the Maryland Department of the Environment. FEMA had indicated that the effective 100-year flood elevations may be inaccurate in the project area based upon present day conditions. The revision of the Montgomery County floodplain maps in 2006 used modeling that was over 20 years old with newer (circa 2006) topographic data. FEMA requested that a new hydraulic model be prepared for this portion of Rock Creek to more accurately determine 100-year flood elevations. The study limits were the segment of Rock Creek south of the Viers Mill Road bridge downstream to the confluence with Turkey Branch. This portion of Rock Creek runs through the Parklawn Cemetery.

To perform the restudy, the USACE collected physical stream/structure data, aerial photography and developed a project specific GIS database including 2-ft contour elevation and a 6-ft x 6-ft Digital Elevation Model (DEM) to map resulting water surface or 100-year flood stage elevations. The USACE acquired the original 2006 FIS HEC-2 computer model data and converted these to HEC-RAS River Analysis System version 4.0 models. Figure 17 shows the results of the HEC-RAS models near Viers Mill Road.

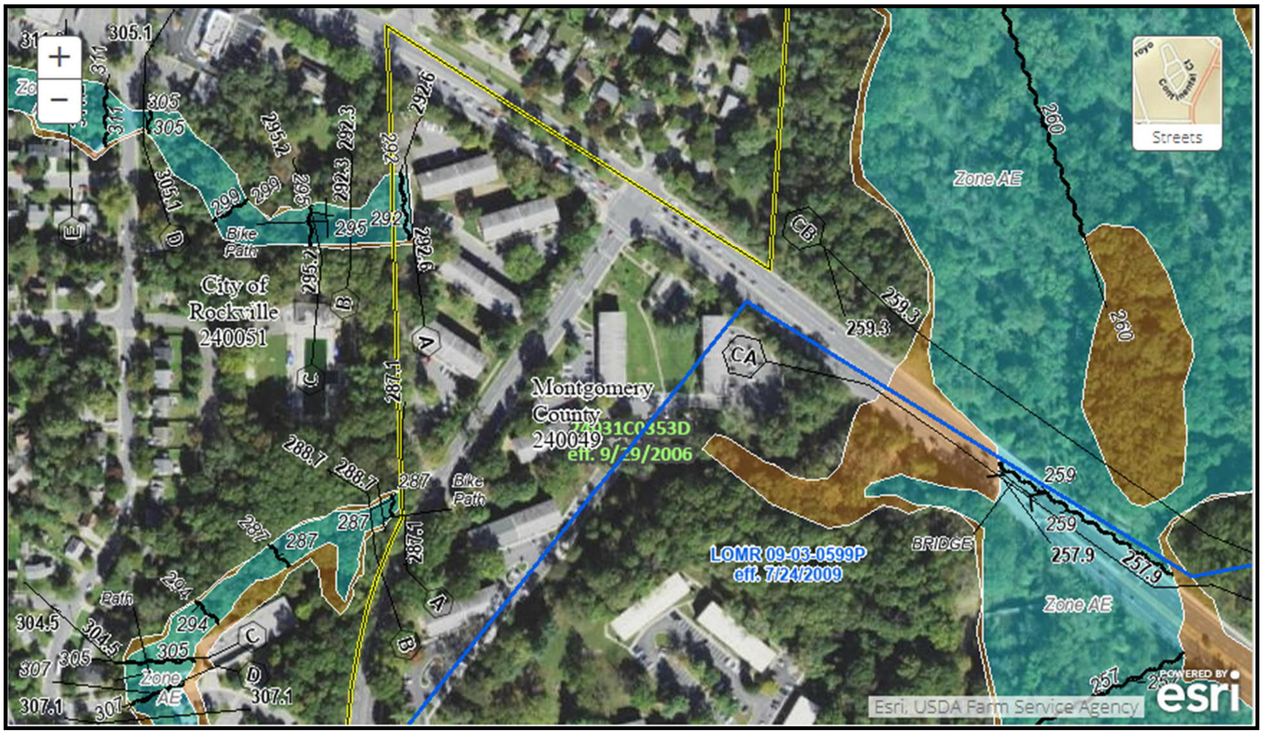

Figure 17 - 2009 Parklawn-Rock Creek LOMR Floodway Map Update (blue areas are within the 100 year floodplain and orange areas are within the 500 year floodplain).

Figure 17 indicates that the first project HEC-RAS cross section CA started just downstream of the Viers Mill Road bridge. It also indicates that modeling of the enclosed culverts was not undertaken to extend the Rock Creek Tributary 2 floodplain east to the junction with Rock Creek.

3.3.7 Summary of Flood Insurance Study Regulatory Floodway Determination

The analysis of historical and current effective Flood insurance studies for the Rock Creek Twinbrook Tributaries 1 and 2 indicates the following:

- The Rock Creek Tributaries 1 and 2 100-year regulatory floodplains were originally mapped in 1978 as part of the City of Rockville Flood Insurance Study, showing a base flood elevation of 293 +/- NGVD 29 for Tributary 2 to the limits with Montgomery County.
- The 100-year water surface or flood elevation was calculated using the HEC-2 model with an initial water surface elevation placed at the entrance to the Tributary 2 culvert enclosure located approximately 10 feet +/- east of the City/County

jurisdictional line (Cross Section A). Cross section "A" is shown along the City/County boundary on the City of Rockville FIS profiles.

- In 1979, the Montgomery County Flood Insurance Study becomes effective but no floodplain is calculated for the segment of Rock Creek Tributary 2 within Montgomery County (i.e. The culvert enclosure of Tributary 1 and 2 is not modeled).
- In 2006 the current Montgomery County Flood Insurance Study becomes effective incorporating the City of Rockville FIS floodplains as part of the Countywide FIS update/revision.
- In the 2006 Countywide FIS update the Rock Creek Tributary 2 floodplain downstream limit is located at the entrance to the enclosed culvert approximately 10 feet east of the City/County jurisdictional line (Cross section "A"). A regulatory Zone AE with a 100-year base flood elevation of 292 Feet +/- (NAVD 88) is shown. Cross section "A" is shown within Montgomery County.
- The culvert enclosure of Tributary 1 and 2 and the remainder of the Tributary 2 downstream channel is not modeled in the 2006 Countywide FIS update.
- A 2009 Letter of Map Revision (LOMR) for the 2006 Countywide FIS segment of Rock Creek between Viers Mill Road and the Turkey Branch Rock Creek tributary is performed to update regulatory floodway mapping using new (HEC-RAS) modeling techniques and DEM-based topography. The LOMR update does not include the modeling of the Rock Creek Twinbrook Tributaries 1 and 2 culvert enclosure to extend the Tributary 2 floodplain from the City/County limits to the Rock Creek floodplain.

3.4 Flooding on September 1, 2021

The flooding at the Rock Creek Woods Apartment complex occurred during Hurricane Ida, which caused high rainfall totals and intensities on September 1, 2021. Twelve ground floor apartment units at Rock Creek Woods were flooded, with fifty total units in two apartment buildings (13212 & 13214 and 13208 & 13210 Twinbrook Parkway) affected. The location of the flooding was near the upstream end of the Tributary 2 box culvert that passes below the apartment complex property. The rain that moved through the Rockville area was unevenly geographically distributed and significantly concentrated in the immediate area around the Rock Creek Woods Apartments. For example, the rain total at the Rock Creek Woods Apartments was approximately 3.1 inches, with a maximum intensity of 6.3 inches per hour between 3:00 am and 3:45 am. Over the same period, the rainfall at the Fallsgrove sewage pumping station (located near the intersection of W Montgomery Avenue and Darnestown Road) recorded a rain total of 1.23 inches with a maximum intensity of 2.2 inches per hour.

4.0 Data Acquisition

The study reviewed all readily available existing documentation, collected field data, and performed multiple site visits to gather information on the geography of the site, the extent of flooding along the tributaries, and the existing condition of the culverts.

4.1 Topographic Survey

Mercado Consultants, Inc. performed a topographic survey of the apartment complex at the location of the flooding, the area upstream of the culverts, and the area downstream of the culverts in September through November of 2021. The detailed topographic survey provided high accuracy horizontal and vertical elevation data at the critical flooding locations to accurately model the flooding conditions.

The survey of the apartment complex included the following:

- Apartment building corners,
- Bottom floor elevations,
- High water marks inside and outside the apartments,
- Top of curb elevations,
- Roadway centerlines,
- Spot grade elevations to establish the grading at the apartments,
- Storm drain inlets and pipe inverts; and
- Top of berm elevations at the upstream end.

The upstream and downstream surveys followed each tributary for 150 feet along the stream centerline and included a 50-foot-wide area on either side of the stream centerline. Mercado Consultants also surveyed the upstream outfalls for both tributaries, the Twinbrook Community Recreation Center access driveway culvert carrying Tributary 1, the Ardennes Avenue and Atlantic Avenue culvert crossings carrying Tributary 2, and any pedestrian bridges that crossed the tributaries. Cross sections along each tributary were taken at regular intervals. Figure 18 summarizes the locations where Mercado Consultants collected field data relative to each tributary, in addition to the material types and sizes of outfalls, crossings, and culverts.

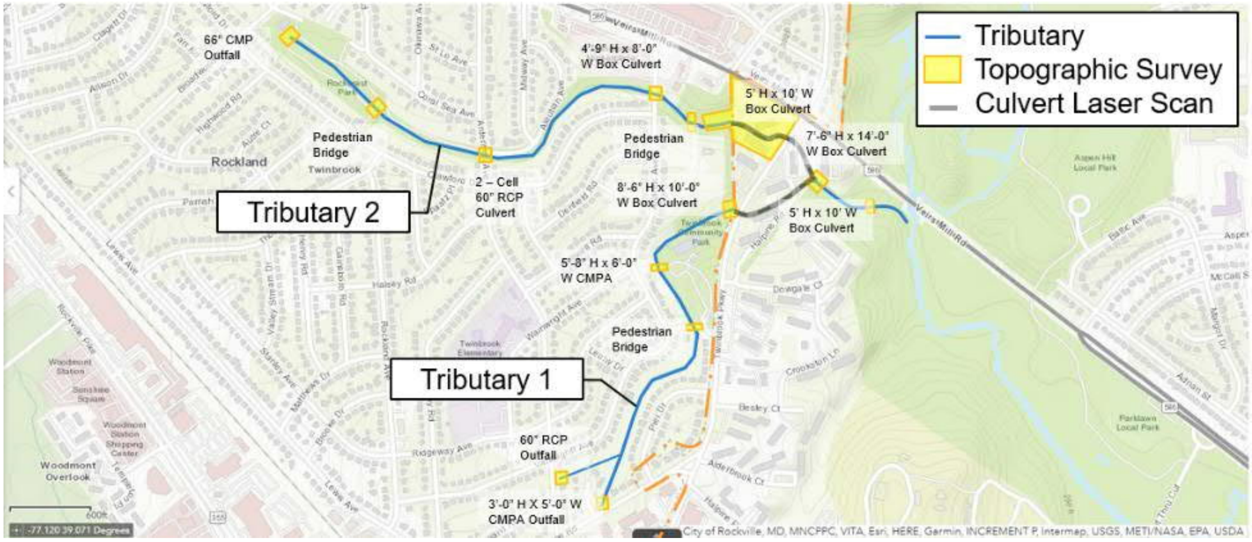

Figure 18 - Location and Type of Field-Collected Survey Data.

4.2 Laser Scan

Mercado Consultants performed a terrestrial lidar survey (as-built laser scan) of both existing, concrete box culverts below the Rock Creek Woods Apartment complex (see Figure 19). This laser scan collected thousands of points known as a point cloud along the inside surfaces of both box culverts. Mercado Consultants cut cross sections and an elevation profile from the point cloud to compare to the original permitted design profile, hydraulic area, alignment, and locations of storm drain tie-ins to determine if the culverts were constructed as per the approved plans (see Section 5.1 Comparison of Culvert to Permitted Plans).

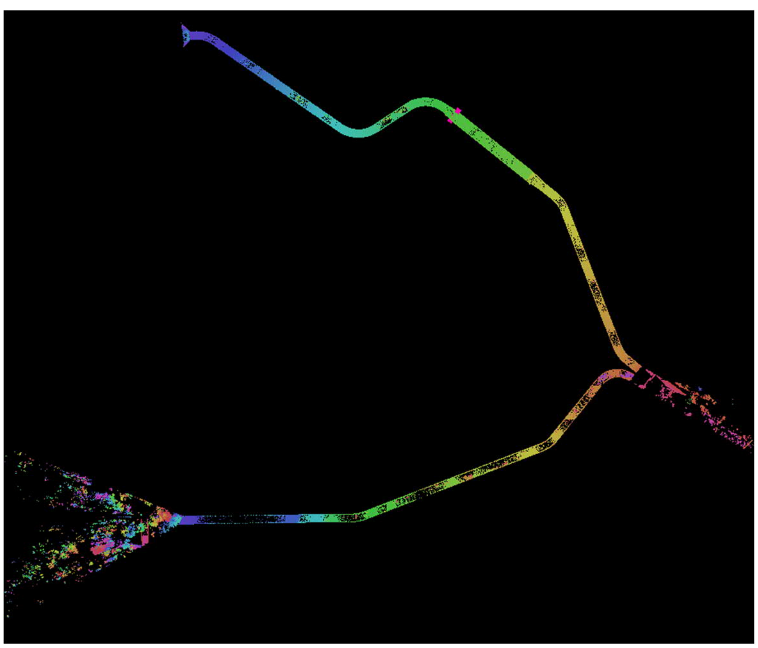

Figure 19 - Point Cloud Plan View of the Tributary 1 and 2 Culverts Created from Laser Scan Data.

4.3 Storm Drain GIS Data

Storm drains in areas outside of the topographic survey limits were modeled using Montgomery County and City of Rockville storm drain GIS data. These GIS data sets contain over 500 storm drain features used in modelling the Rockcrest and Twinbrook drainage areas. All storm drains were reviewed before being included into the study computer models, and approximately 5% of these storm drains were confirmed in the field to verify the accuracy of the data set and to obtain additional detail as needed for the study computer models. See Figure 20 for the storm drain network from GIS within the study drainage areas.

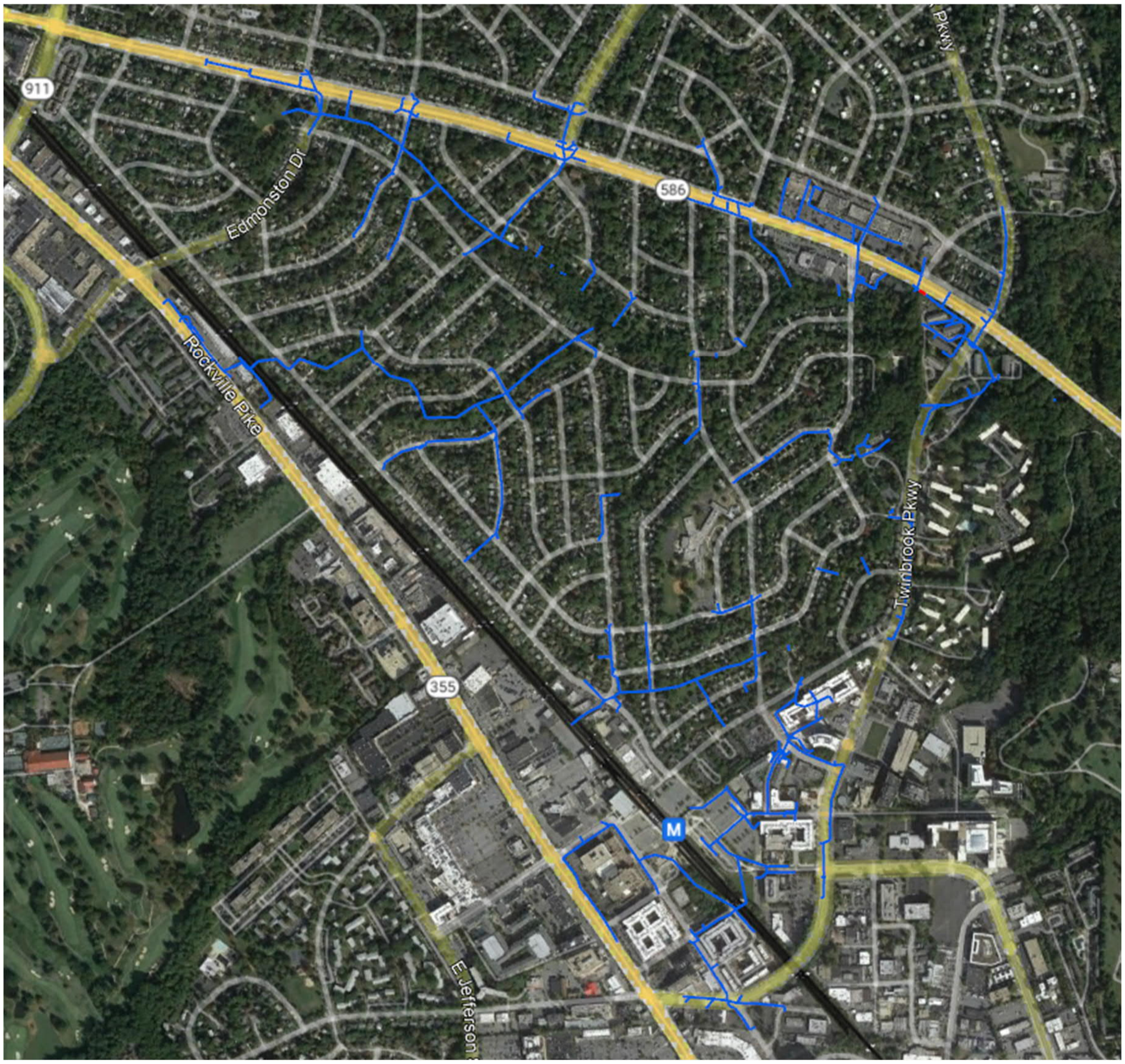

Figure 20 - GIS Storm Drains within Study Area (Shown in Blue)

4.4 On-Site Inspection

Mercado Consultants' engineers performed on-site inspections of the subject area on September 7, September 10, and October 12, 2021, to review the existing culverts, the conditions upstream and downstream of the culverts, and the affected apartment buildings. During the inspections, the engineers also received eye-witness accounts of the flooding event.

4.4.1 Culvert Inspection

Bill Mercado, P.E. and Michael Mercado, P.E. conducted an inspection of both culverts on September 7, 2021. The inspection was carried out to evaluate the structural condition of the culvert, investigate potential blockage along the entire length of the culvert including the headwall openings at both the upstream and downstream ends, and review the size and configuration of the box culverts relative to the permitted plan set.

Hands on inspection of the structures revealed that the openings were clear with very little siltation and no signs of large blockages, such as tree limbs or boulders, that would have restricted flow during the rain event (see Figure 21). In several locations, there were block-outs in the side walls where storm drains could have been placed during construction. These block-outs had exposed, corroded reinforcing steel with exposed fill and voids up to 2 feet deep beyond the box culvert wall (see Figure 22). It appears these block-outs were never filled with concrete after construction was completed. Overall, the concrete box culverts were found to be in sound condition with only minor areas of concrete spalling or exposed reinforcing steel (see Figure 23). The inspection was filmed by Mercado Consultants' staff.

Figure 21 - Typical View inside of Box Culverts.

Figure 22 - Blockout in Side Wall of Box Culvert

Figure 23 - Typical Minor Spall with Exposed Reinforcing Steel on Top Slab of Box Culvert

The culvert inspection also verified the overall culvert dimensions and configuration as compared to the WSSC permitted plan set from May 23, 1966. A final as-built set was not available in any historical records archives. The permitted plan set shows two culverts that convey storm water below the Rock Creek Woods Apartments' property (see Figure 24). The culvert to the North which carries Tributary No. 2 (the tributary that reported flooding in the Rock Creek Woods Apartments on 9/1/2021) consists of the following starting from the upstream end:

1. An upstream concrete headwall with an earthen berm on top. The low point of the top of the earthen berm was surveyed at EL. 291.07 (NAVD 88) (see Figure 25)
2. 10'-0" +/- wide x 5'-0" +/- high concrete box culvert that extends 409'+/- long (see previous Figure 21)
3. 14'-0" +/- wide x 7'-6" +/- high concrete box culvert that was originally constructed around 1964 (see Figure 26)
4. 10'-0" +/- wide x 5'-0" +/- high concrete box culvert that extends 302'+/- long (see previous Figure 21)
5. A downstream concrete headwall that is shared with the southern box culvert (see Figure 27)
6. Concrete velocity dissipaters approximately 19'-0" +/- downstream of the downstream headwall (see Figure 28)

The culvert to the South which carries Tributary No 1 consists of the following starting from the upstream end:

1. An upstream concrete headwall (see Figure 29)
2. 10'-0" +/- wide x 8'-6" +/- high concrete box culvert that was originally constructed around 1964 (see Figure 30)
3. 10'-0" +/- wide x 5'-0" +/- high concrete box culvert that extends 507'+/- long (see previous Figure 21)
4. A downstream concrete headwall that is shared with the northern box culvert (see Figure 27)
5. Concrete velocity dissipaters approximately 19'-0" +/- downstream of the downstream headwall (see Figure 28)

There were several locations of inlets placed directly in the top slab of the box culverts. These locations were:

1. Northern Culvert: 71' +/- downstream of the upstream headwall (see Figure 31 and Figure 32)
2. Northern Culvert: 75' +/- upstream of the downstream headwall
3. Southern Culvert: 219' +/- upstream of the downstream headwall

Overall, both the configuration, hydraulic openings, bends, and changes in grade appeared to match the permitted plan set based on the site visit. A more in-depth review was performed using the point cloud and is presented in Section 5.1 Comparison of Culvert to Permitted Plans.

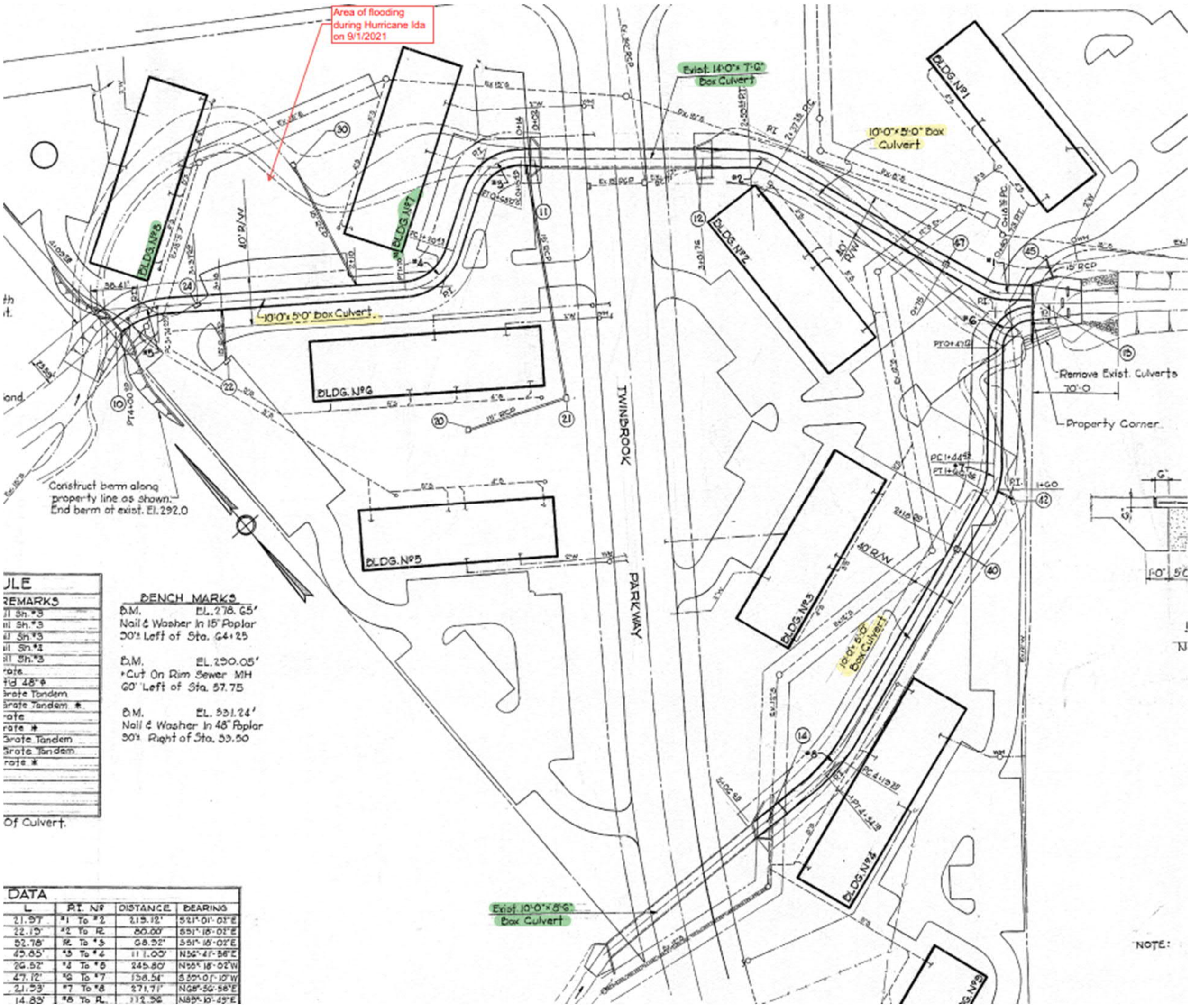

Figure 24 - Permitted Plan Set Showing Northern and Southern Culverts

Figure 25 - Northern Culvert, Upstream Headwall

Figure 26 - Inside 14'-0" x 7'-6" Box Culvert Looking Downstream towards 10'-0" x 5'-0" Box Culvert

Figure 27 - Downstream Headwall

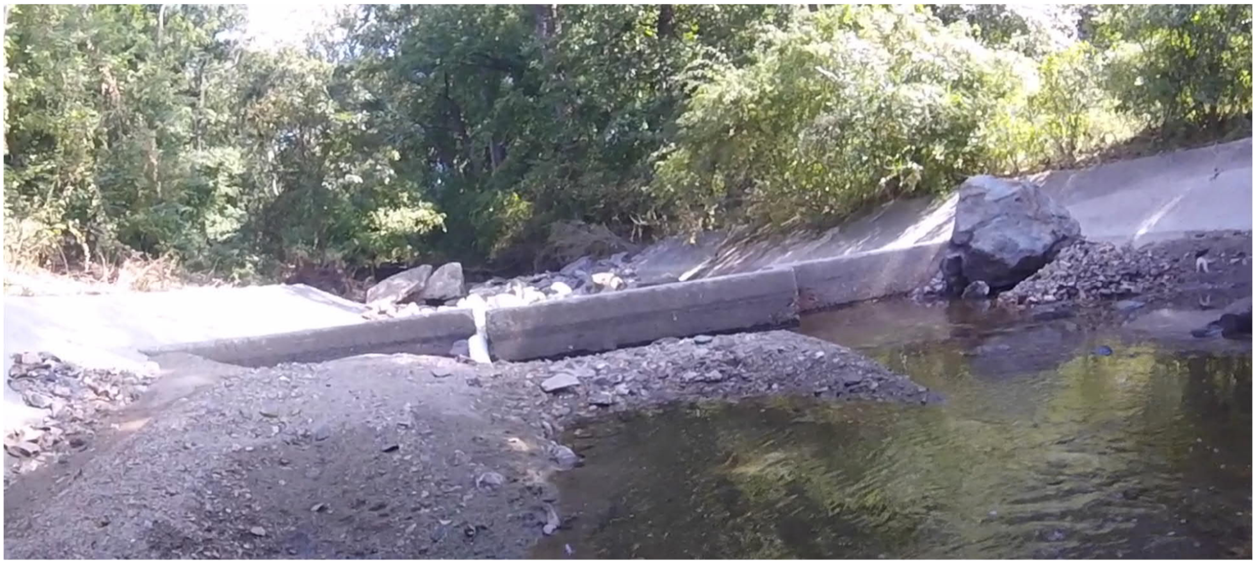

Figure 28 - Downstream Velocity Dissipators

Figure 29 - Southern Culvert Upstream Headwall

Figure 30 - Inside 10'-0" x 5’-0” Box Culvert Looking Upstream towards 10’-0” x 8’-6” Box Culvert

Figure 31 - Underside of Inlet Directly in Box Culvert Top Slab

Figure 32 - Top of Inlet Directly in Box Culvert Top Slab

4.4.2 Apartment Inspection

Bill Mercado, P.E. and Michael Mercado, P.E. conducted an inspection of the area of flooding at the Rock Creek Woods Apartments on September 7, 2021. The area that flooded was designated as Buildings Numbers 7 and 8 on the WSSC permitted plan set from May 23, 1966, and are located on the Northwest corner of the property near the Northern concrete box culvert (see Figure 33).

Figure 33 - Approximate Area of Flooding at Rock Creek Woods Apartments (Shown by Red Circle)

The inspection was performed while clean-up efforts were underway to remove flood damage for the basement level apartments for both of the flooded buildings (see Figure 34). The watermarks from the flood were evident on both buildings. Building No. 8 (Western building that flooded) had a high water mark of approximately 5.0’ +/- from the concrete patio in front of the apartments which was evident on the drywall (see Figure 35 and Figure 36). Building No. 7 (Eastern building that flooded) had a high water mark of approximately 7.9’ +/- from the concrete patio in front of the apartments which was evident on the siding outside the apartments (see Figure 37 and Figure 38). Building 7 is lower in elevation compared to Building 8.

Figure 34 - Area of Flooding and Debris Clean Up

Figure 35 - High Water Mark Building 8

Figure 36 - High Water Mark Building 8

Figure 37 - High Water Mark Building 7

Figure 38 - High Water Mark Building 7

The area of flooding between Buildings 7 and 8 was observed to be in a depression in the shape of a bowl. A review of aerial lidar confirmed the shape of the topography (see Figure 39). Within the depression, there is no way for stormwater to naturally exit by gravity except through the storm drain system. Six inlets were connected by a series of 4” to 6” PVC drain pipes and ultimately discharged into an 18” reinforced concrete pipe (RCP) and 15” RCP at the Northern edge of the depression. Five of the inlets had green plastic grate coverings (see Figure 40), and one of the inlets had a metal grate covering (see Figure 41). All inlets were clear of debris during the inspection. The configuration of the inlets did not match the WSSC approved permitted plan set from 1966, (see Section 5.2 Comparison of Storm Drain System to Permitted Plans). A final as-built plan for the apartments’ storm drain system was not available in historical records and it is not known if any of the changes to the storm drain system were approved during construction.

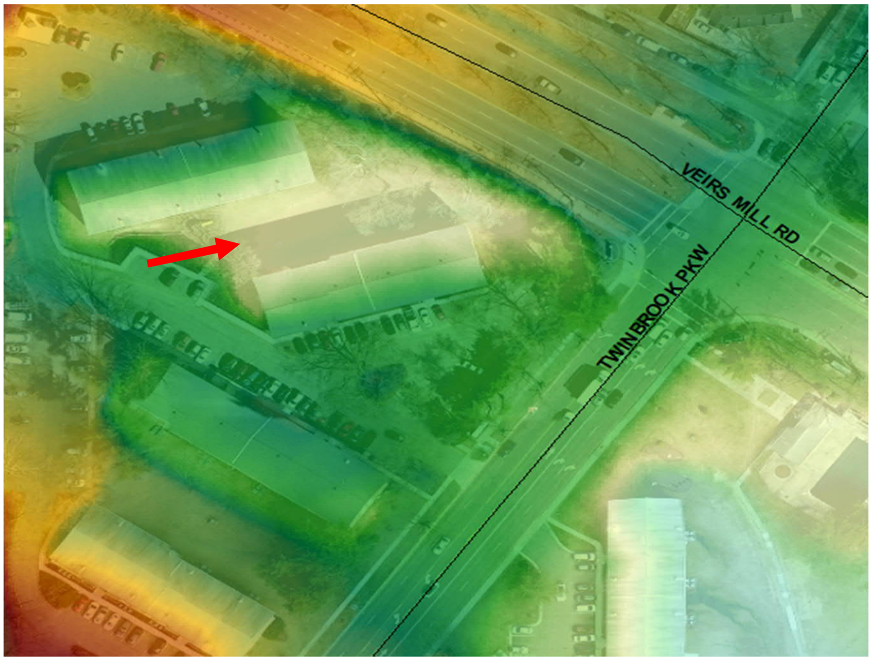

Figure 39 - Lidar Showing Depression (Bowl) in Area of Flooding.
Colors from High to Low: Brown, Red, Yellow, Green, White, Light Blue

Figure 40 - Inlet with Plastic Grate in Depression of Rock Creek Woods Apartments

Figure 41 - Inlet with Metal Grate in Depression of Rock Creek Woods Apartments

4.4.3 Stream Inspection

Mercado Consultants conducted visual inspections of Tributary 1, Tributary 2, and the culvert outfall up until its connection to Rock Creek to investigate the condition of the streams, locate any visible high water level marks or erosion produced by the storm, and to solicit eyewitness accounts of the storm, including observations of high flood water levels.

The banks of the tributaries were found to be in satisfactory condition with sufficient vegetation along the banks and signs of bank erosion at isolated locations. Levels of high water were evident throughout the entire length of the tributary and are organized by location below. A summary of the locations of the crossings for each tributary are presented earlier in Figure 18.

4.4.3.1 Tributary 1

Bill Mercado, P.E. and Michael Mercado, P.E. conducted a visual inspection of Tributary 1 from the most upstream storm drain outfall to the box culvert entrance on October 12, 2021. Throughout the length of the tributary, there was low flow of several inches of water. Stone riprap, trees, and vegetation line its banks (see Figure 42).

Figure 42 - Typical stream and bank conditions of Tributary 1.

At the upstream end of Tributary 1, there are two outfalls: a 60-inch reinforced concrete pipe (RCP) outfall by Ardennes Avenue and a three-cell, 3’-0” high by 5’-0” wide corrugated metal pipe arch (CMPA) outfall near Halpine Road (see Figure 43 and Figure 44). The streams from these two outfalls connect approximately 400’ +/- downstream and form Tributary 1. A pedestrian bridge crosses the tributary at Vandergrift Avenue (see Figure 45). The tributary flows through a 5’-8” high by 6’-0” wide corrugated metal pipe arch (CMPA) below the access driveway between Atlantic Avenue and the Twinbrook Community Recreation Center (see Figure 46). Another pedestrian bridge crosses the tributary before the culvert entrance (see Figure 47). Debris was observed at an elevation that was below the top of the headwall (see Figure 48). No signs of water overtopping the head wall were found on Twinbrook Parkway.

Figure 43 - 60-inch RCP Outfall to Tributary 1.

Figure 44 - CMPA Outfall to Tributary 1.

Figure 45 - Pedestrian Bridge over Tributary 1 at Vandergrift Avenue.

Figure 46 - Tributary 1 CMPA Crossing South of the Twinbrook Community Recreation Center.

Figure 47 - Pedestrian Bridge Crossing Tributary 1 North of the Twinbrook Community Recreation Center.

Figure 48 - Headwall of the Tributary 1 Culvert.

4.4.3.2 Tributary 2

Bill Mercado, P.E. and Michael Mercado, P.E. conducted a visual inspection of Tributary 2 from the most upstream storm drain outfall to the box culvert entrance on October 12, 2021. Throughout the length of the tributary, there was low flow of several inches of water. Stone riprap, trees, and vegetation line its banks (see Figure 49).

Figure 49 - Tributary 2 Typical Stream and Bank Conditions.

The open-channel tributary begins at a 66-inch corrugated metal pipe (CMP) outfall at its most upstream end (see Figure 50). A pedestrian bridge crosses above the tributary approximately halfway between the outfall and Ardennes Avenue (see Figure 51). The tributary then flows through two 60-inch reinforced concrete pipes (RCP) beneath Ardennes Avenue and a rectangular box culvert (4’-9” high by 8’-0” wide) beneath Atlantic Avenue (see Figure 52 and Figure 53). Debris and erosion found along the bank and the broken fence were visual signs of flood water overtopping Atlantic Avenue (see Figure 54). Debris was also present in a fence adjacent to Tributary 2 parallel to Aleutian Avenue (see Figure 55). Another pedestrian bridge crosses the tributary between Atlantic Avenue and the culvert entrance (see Figure 56). Debris was observed within sections of the chain link fence above the top of the culvert entrance headwall (see Figure 57).

Figure 50 - 66-inch CMP Outfall to Tributary 2.

Figure 51 - Pedestrian Bridge Crossing Tributary 2 Downstream of Outfall.

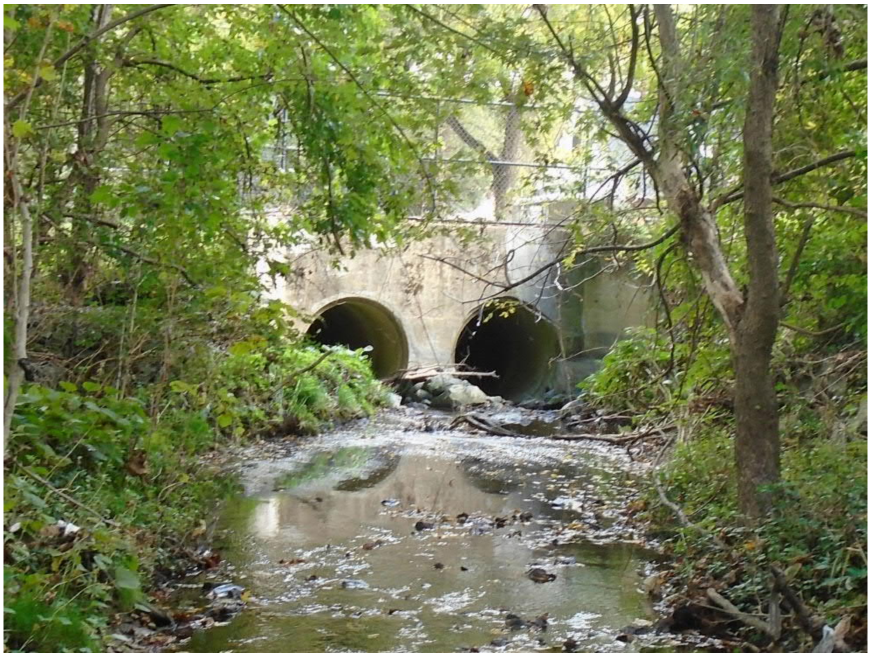

Figure 52 - Ardennes Avenue Crossing Tributary 2.

Figure 53 - Atlantic Avenue Crossing Tributary 2. Note: Damaged fence from flooding.

Figure 54 - Debris and Erosion at Atlantic Avenue Crossing.

Figure 55 - Debris in Fence Adjacent to Tributary 2.

Figure 56 - Pedestrian Bridge Crossing Tributary 2 Downstream of Atlantic Avenue.

Figure 57 - Tributary 2 Culvert Headwall. Note Debris in Fence (Arrows).

4.4.3.3 Downstream of Culvert Outfall

Bill Mercado, P.E. and Michael Mercado, P.E. conducted a visual inspection from the box culvert outfall to Rock Creek on September 10, 2021. The tributary flows around and through an abandoned roadway culvert consisting of two 42” RCPs approximately 600’ +/- downstream of the box culvert below the Rock Creek Woods Apartments’ outfall (see Figure 58 and Figure 59). The tributary flows into Rock Creek adjacent to the crossing of Veirs Mill Road over Rock Creek Bridge (see Figure 60). A pedestrian bridge crosses Rock Creek downstream of the Veirs Mill Road over Rock Creek Bridge (see Figure 61).

Figure 58 - Abandoned Roadway Culvert Downstream of Culvert Outfall (Photo 1)

Figure 59 - Abandoned Roadway Culvert Downstream of Culvert Outfall (Photo 2).

Figure 60 - Veirs Mill Road Over Rock Creek.

Figure 61 - Pedestrian Bridge Crossing Downstream of Viers Mill Road Bridge.

Debris was found along the entire bank of the tributary, with erosion present adjacent to the concrete apron at the box culvert outfall (see Figure 62). There was no evidence along Veirs Mill Road that the street had flooded during the storm. At the confluence with Rock Creek, there was evidence of high flood water based on bent grass through the downstream area beyond the Viers Mill Road Bridge (see Figure 63). There was no evidence of water overtopping the Veirs Mill Road Bridge crossing at Rock Creek.

Figure 62 - Typical Erosion Along Bank Above Concrete Apron.

Figure 63 - High Water Indicator Near Veirs Mill Road.

4.4.4 Witness Testimony

Mercado Consultants discussed the Hurricane Ida storm event and flooding with the following individuals during the field investigations:

1. A resident living at 13200 Ardennes Ave. (Northwest corner of the Tributary 2 crossing at Ardennes Ave.) stated he had witnessed the storm event on the morning of September 1, 2021. The resident confirmed that the storm water had overtopped Ardennes Avenue and spread out laterally over the shared use path up to the edge of the resident's porch. This distance was approximately 75' +/- from the centerline of the stream.
2. The manager at the Twinbrook Community Recreation Center stated that the storm water the morning of September 1st from Tributary 1 flowed approximately halfway up the lawn in front of the Recreation Center. This distance is approximately 80' +/- from the centerline of the stream.
3. A man was residing approximately 600' +/- downstream of the outfall of the box culverts below the Rock Creek Woods Apartments where the stream passes by an abandoned roadway before entering Rock Creek. The man stated he was sleeping at the time of the flooding and woke up as the water began to rise. He stated that he witnessed the water level rising approximately 7' from the streambed.

5.0 Culvert Design and Construction

5.1 Comparison of Culvert to Permitted Plans

Using the information gathered from the laser scan and on-site inspection, Mercado Consultants compared the existing conditions of the culvert alignment to the 1966 permitted plan set approved by WSSC. A Benchmark manhole cover elevation listed on the plan set was surveyed to adjust the plan set elevations to the current North American Vertical Datum 1988 (NAVD88). The difference between the Benchmark elevation noted on the plans and the NAVD88 survey elevation (1.02 ft) was subtracted from each of the plan set elevations and compared to the corresponding laser scan survey value at each location (see Table 1). The surveyed horizontal alignments of the culverts varied from ½" to 4½" of the permitted plan set. The surveyed vertical alignment varied from 0" to 3½ inches lower than the plans.

Table 1 - Comparison of Permit Plan and Laser Scan Dimensions and Elevations

Location Description	Station	Width (ft)		Height (ft)		Invert Elevation (NAVD 88; ft)		Difference in Alignment	
		Plans	Laser Scan	Plans	Laser Scan	Plans	Laser Scan	Horizontal (ft)	Vertical (ft)
Tributary 2									
Culvert Outfall	0+00	10	10	5	5	258.98	258.97	N/A	-0.01
East of Twinbrook Pkwy	0+50	10	10	5	5	260.88	260.6	0.28' North	-0.28
	1+00	10	10	5	5	261.56	261.57	0.20' North	0.01
	1+50	10	10	5	5	262.21	262.05	0.25' North	-0.16
	2+00	10	10.4	5	5	262.87	262.68	0.32' North	-0.19
	2+50	10	10.6	5	5.1	263.52	263.34	0.12' North	-0.18
Twinbrook Pkwy Culvert Downstream	3+01.76	14	13.8	7.5	7.7	264.2	264.11	N/A	-0.09
Twinbrook Pkwy Culvert Middle	N/A	14	14.2	7.5	7.5	264.93	264.71	0.18' South	-0.22
Twinbrook Pkwy Culvert Upstream	0+00	14	13.8	7.5	7.3	265.55	265.66	N/A	0.11
West of Twinbrook Pkwy	0+50	10	10	5	5	269.6	269.47	0.32' South	-0.13
	1+00	10	10	5	4.9	271.26	271.18	0.34' South	-0.08
	1+50	10	10	5	4.9	273.12	272.92	0.30' South	-0.2
	2+00	10	10	5	5	274.57	274.5	0.20' South	-0.07
	2+50	10	10.1	5	5	275.73	275.64	0.23' South	-0.09
	3+00	10	10	5	5	277.56	277.47	0.17' South	-0.09
	3+50	10	10	5	5	279.55	279.32	0.13' South	-0.23
Culvert Entrance	4+09.28	10	10	5	5	280.58	280.45	0.37' South	-0.13
Tributary 1									
Culvert Outfall	0+00	10	10.1	5	5	258.98	258.86	0.13' North	-0.12
East of Twinbrook Pkwy	0+50	10	10.1	5	5.1	260.16	260.06	0.08' South	-0.1
	1+00	10	10	5	5	261.34	261.13	0.02' North	-0.21

Location Description	Station	Width (ft)		Height (ft)		Invert Elevation (NAVD 88; ft)		Difference in Alignment	
		Plans	Laser Scan	Plans	Laser Scan	Plans	Laser Scan	Horizontal (ft)	Vertical (ft)
	1+50	10	10	5	5	262.52	262.36	0.04' South	-0.16
	2+00	10	10	5	5	263.7	263.40	0.08' North	-0.3
	2+50	10	10	5	5	264.88	264.68	0.05' North	-0.2
	3+00	10	10	5	5	266.06	265.94	0.17' North	-0.12
	3+50	10	10	5	4.8	267.24	267.17	0.05' North	-0.07
	4+00	10	10	5	5	268.42	268.24	N/A	-0.18
	4+50	10	10	5	5	269.6	269.55	0.05' North	-0.05
Twinbrook Pkwy Culvert Downstream	5+06.65	10	10	5	5	274.61	274.36	0.04' North	-0.25
Twinbrook Pkwy Culvert Middle	N/A	10	10	8.5	8.5	275.85	275.85	0.18' North	0
Twinbrook Pkwy Culvert Upstream	N/A	10	10	8.5	8.5	277.03	276.895	0.06' South	-0.14

5.2 Comparison of Storm Drain System to Permitted Plans

Mercado Consultants compared the 1966 and 1967 permitted plans approved by WSSC to the current existing storm drain system at and around the apartment complex (see Figure 64, Figure 65, and Figure 66). An update to the 1966 plan set was approved in 1967 which redirected storm water to a series of pipes that run parallel to Veirs Mill Road and discharge into the side of the concrete box culvert at a more downstream location. There was no additional documentation or notes outlining why this change was made. In addition, a 15-inch pipe and manhole at the south side of the culvert were not built in the location shown in the 1966 plans. An inlet and 12-inch pipe were constructed approximately 50 feet east and connect into the culvert.

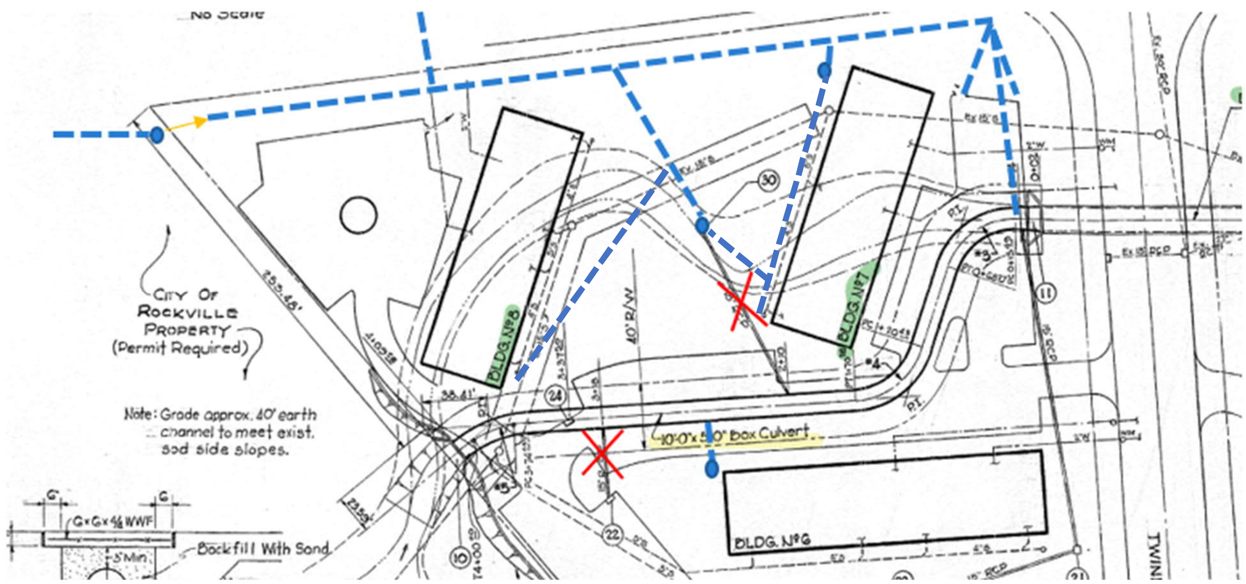

Figure 64 - Excerpt of 1966 Permit Plans showing inlets (blue circles), storm drain pipes (blue dashed lines), concrete flume (yellow arrow) and pipes that do not match the actual surveyed conditions (red X's).

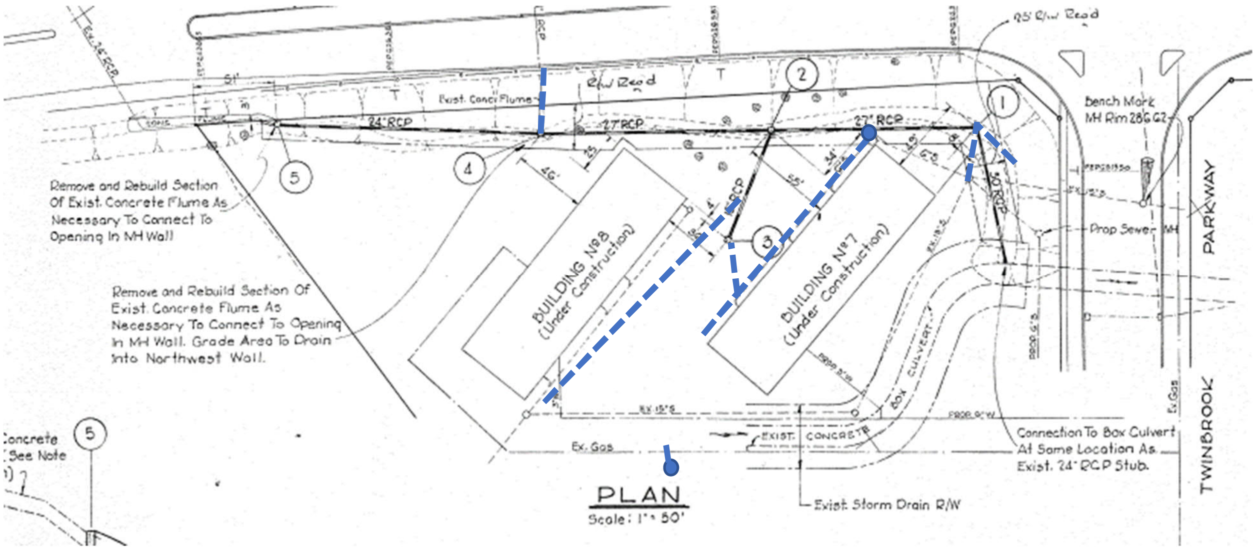

Figure 65 - Excerpt from the 1967 Permit Plans for the Rock Creek Woods Apartments showing revised storm drain layout.

Figure 66 - Existing Storm Drain System with Pipe Diameters (inches) and Culvert (Dashed line, dimensions in feet).

The 1967 plans note that an open concrete flume feeds into an inlet at the west end of the property (see Figure 65 and Figure 67). This open concrete flume is at the outfall of a storm drain system that drains portions Viers Mill Road to the West and a BMP from the adjacent Laundromat (see Figure 68). The inlet at the end of the concrete flume conveys flow into the same storm drain network that drains the low point between Buildings 7 and

8 and then discharges into the concrete box culvert below the Rock Creek Woods Apartments' Property.

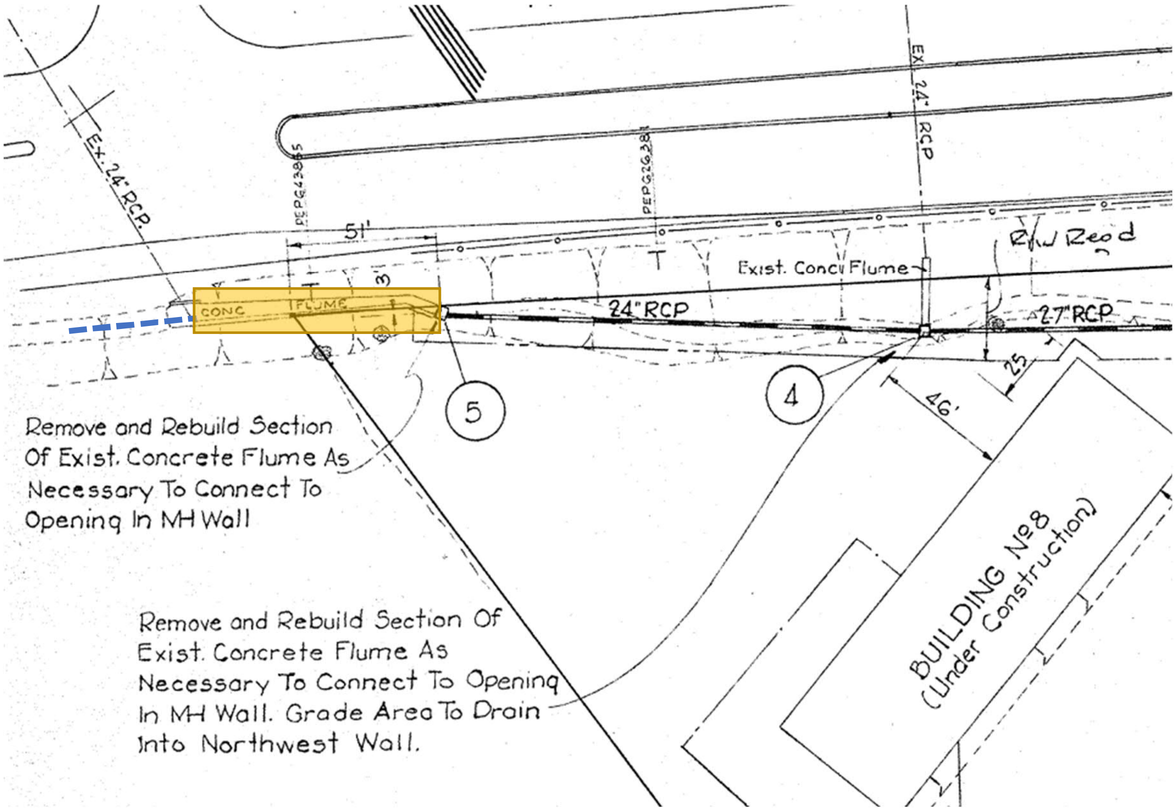

Figure 67 - Zoomed In Excerpt of 1967 Permitted Plans. Concrete Flume is highlighted in yellow. Storm drains from Viers Mill Road are shown by dashed blue line.

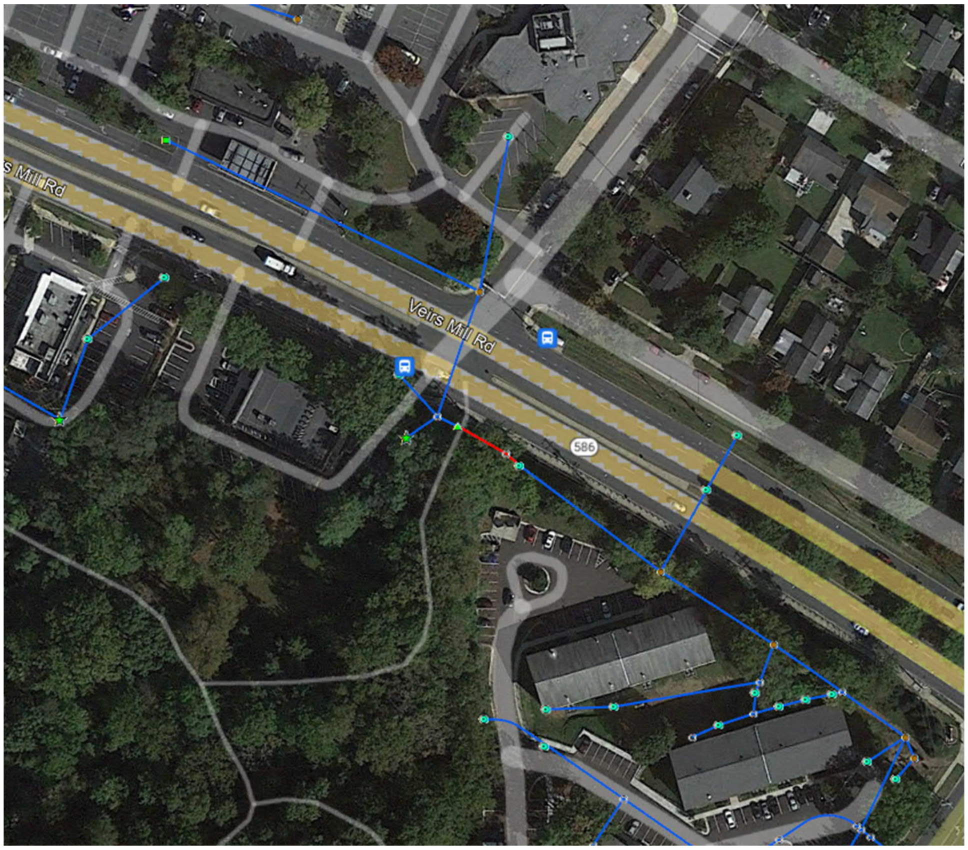

Figure 68 - Storm drain network. Concrete flume shown by red line. Storm drain pipes shown by blue lines.

Figure 69 - Upstream storm drain outfall at concrete flume.

Figure 70 - Downstream end of concrete flume (blue arrow) connected to inlet (red arrow) at the west end of the apartment complex.

5.3 Original Culvert Design Assessment

The original culvert design can be traced to the relocation of Twinbrook Parkway (originally Halpine Road) in the early 1960's. Civil Engineering plans by Greenhorne & O'Mara Consulting Engineers labeled "*Relocated Halpine Road, Rockville Pike to Viers Mill Road, Revised April 13, 1964*", include detailed information of the culvert design under Twinbrook Parkway to pass through flows from the two Twinbrook area tributaries to the Rock Creek floodplain. Figure 71 shows the location of the Tributary 1 culvert crossing at Twinbrook Parkway station 58+50 and Tributary 2 culvert crossing at Twinbrook Parkway station 64+50.

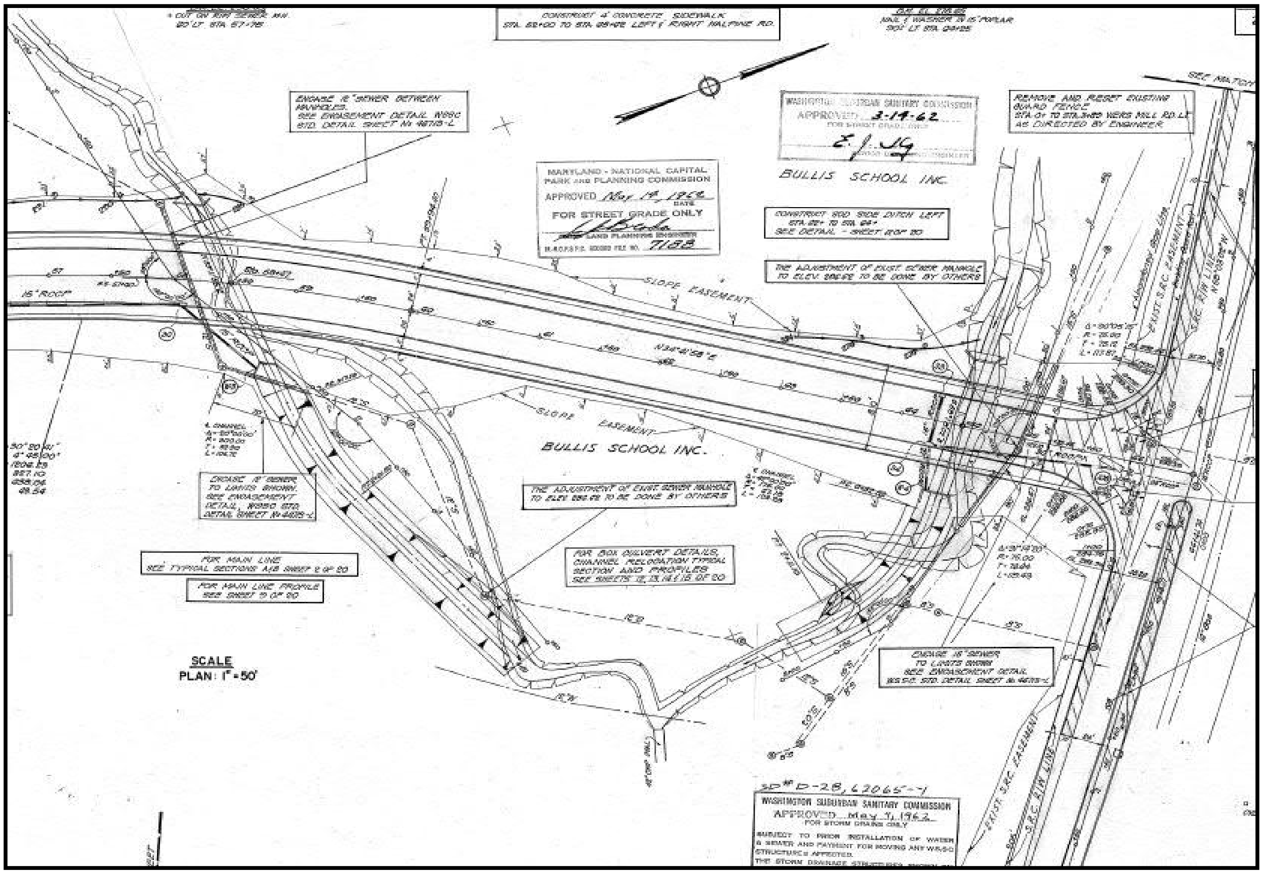

Figure 71 - Twinbrook Parkway Relocation Culvert Crossing Locations (Source: G&O, 1964)

Figure 71 indicates that the tributaries were also redirected at the culvert outlets by way of grass- and riprap-lined channels to reconnect with the existing stream bed prior to discharging into the Rock Creek floodplain. Figure 72 and Figure 73 show the northern (Tributary 2) culvert crossing site plan details and hydrologic & hydraulic design specifications.

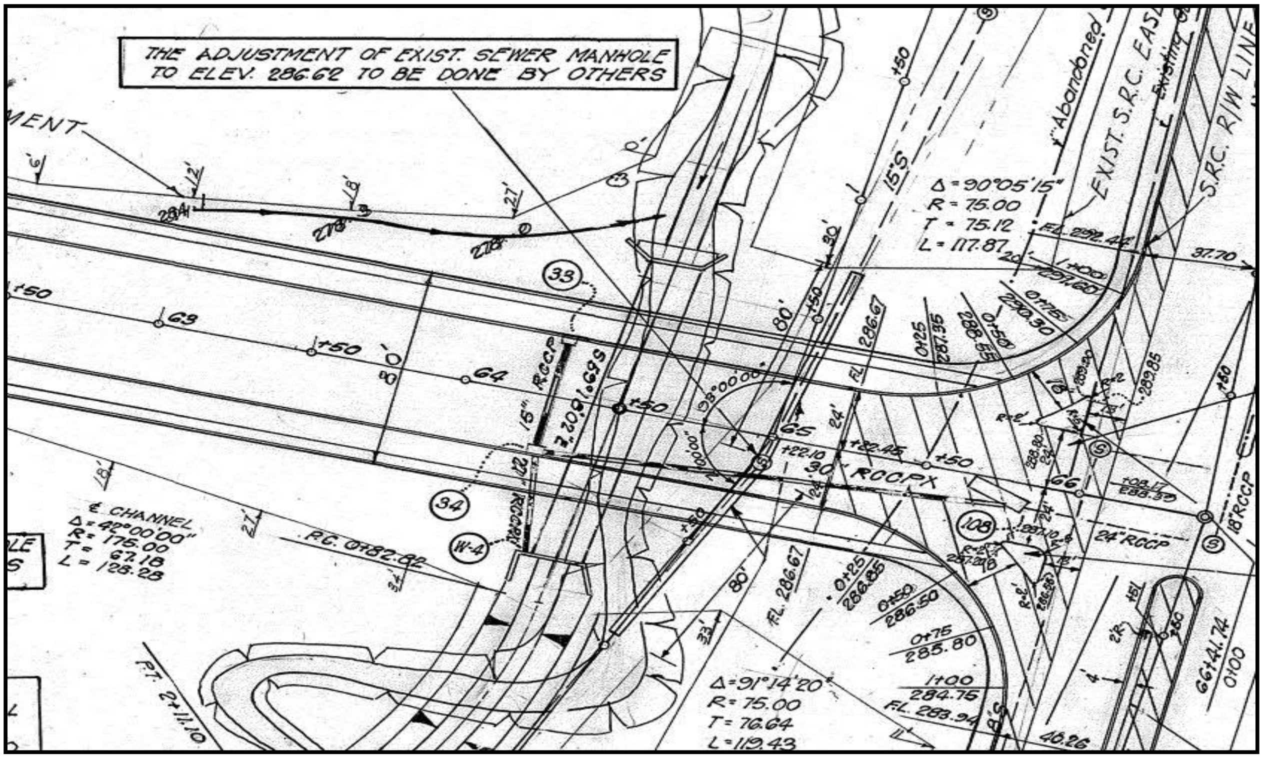

Figure 72 - Northern Culvert Crossing at Tributary 2

HYDRAULIC DATA

DRAINAGE AREA = 0.74 SQ. MI. = 473 ACRES

STORMWATER DISCHARGE C.F.S. = 1031

TIDAL FLOW PER HOUR C.F.S. = 0

TOTAL MAXIMUM DISCHARGE C.F.S. = 1031

MAXIMUM FLOW DEPTH AT H.W. FEET = 8.93 Inlet, 6.00 Outlet

OPENING BY TALBOT

OPENING TO H.W. SQ. FT. = 105.0 $

VELOCITY AT OUTLET FT. PER SEC. = 12.27 1/5

Figure 73 - Tributary 2 Culvert Design Specifications

Figure 74 and Figure 75 show the southern (Tributary 1) culvert crossing site plan details and hydrologic/hydraulic design specifications.

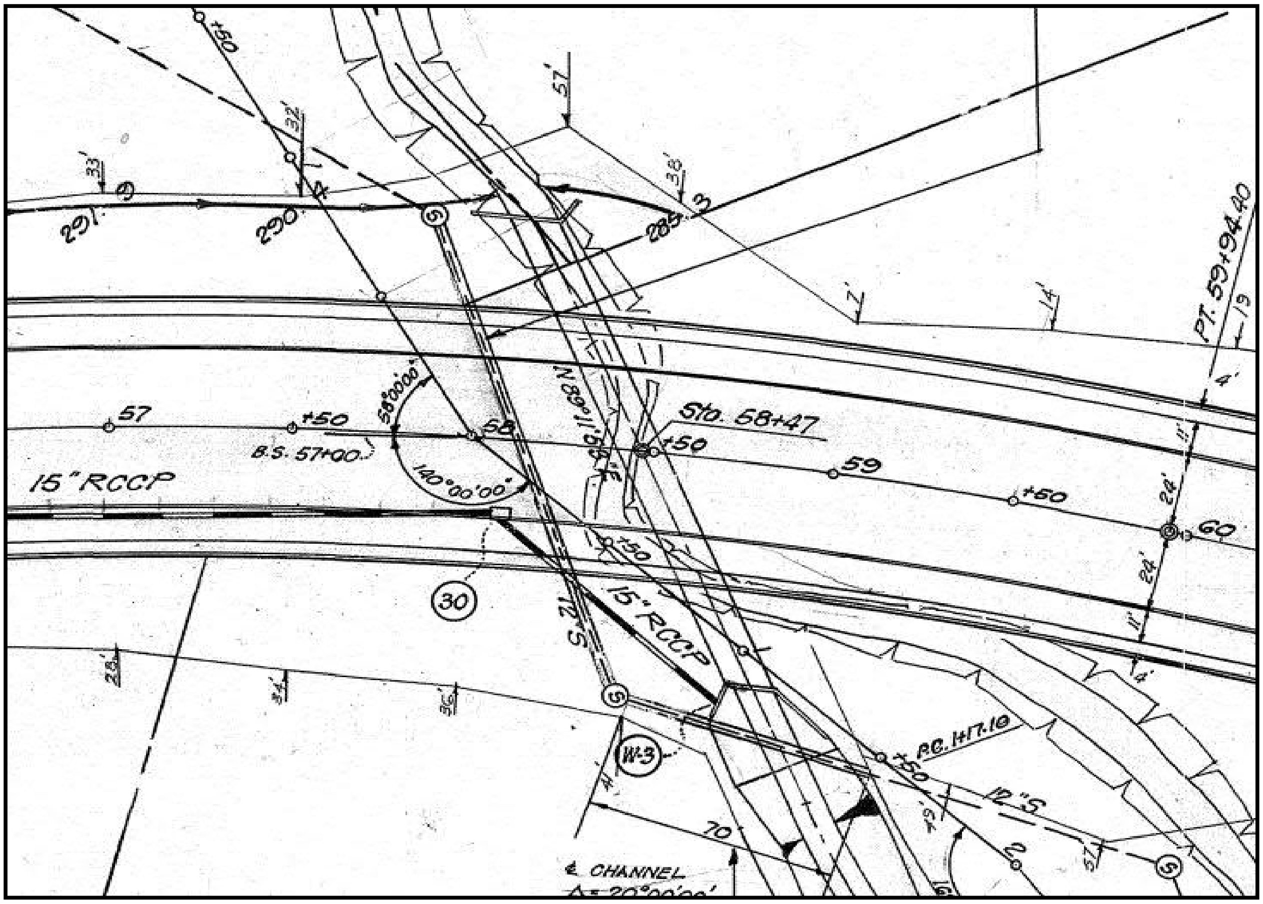

Figure 74 - Southern Culvert Crossing at Tributary 1

HYDRAULIC DATA
DRAINAGE AREA = .40 SQ. MI. = 254.5 ACRES
STORMWATER DISCHARGE C.F.S. = 852.0
TIDAL FLOW PER HOUR C.F.S. = 0
TOTAL MAXIMUM DISCHARGE C.F.S. = 852.0
MAXIMUM FLOW DEPTH AT H.W. FEET = 9.78 Inlet
8.00 Outlet
OPENING BY TALBOT
OPENING TO H.W. SQ. FT. = 85.0$
VELOCITY AT OUTLET FT. PER SEC. = 10.65'/s

Figure 75 - Tributary 1 Culvert Design Specifications

Figure 73 and Figure 75 provide insights into the hydrologic & hydraulic design of the culverts in 1963.

5.3.1 Hydrologic Design

Standard engineering practice applied today (and in the 1960's) calls for a three-step analysis for the design of a culvert.

1. The level of service (LOS) return period (Design frequency) must be determined
2. The hydrology of the watershed must be evaluated (Catchment or watershed basin area contributing flow to the culvert)
3. A flow rate for the design return period must be estimated (Peak basin discharge and design discharge as input for culvert sizing by hydraulic design procedures)

The Halpine Road Relocation plan set indicates that a peak discharge of 852 cfs was calculated for the Tributary 1 drainage area of 254.5 acres, and a peak discharge of 1031 cfs was calculated for Tributary 2 with a drainage area of 473 acres.

To assess the LOS frequency used for the 1963 Halpine Road relocation culvert design, the prevalent contemporary design methodologies were identified during the technical literature research. A 1962 technical report by Ven T. Chow[12] provides a comprehensive summary of various methods for the hydrologic determination of waterway areas for the design of small drainage structures.

Chow presents a summary of hydrologic design practices across 43 states dating back to 1852 and as of 1962. Among the 43 states, the most widely used methods were:

- The Talbot method: 58%
- The U.S. Geological Survey methods: 23 %
- The Rational Method: 13%
- Other methods: 6%

5.3.1.1 The Talbot Method

In 1876, Professor A. N. Talbot"' of the University of Illinois published[13] his well-known formula for determining the waterway area of culverts, which had since been very widely adopted in the United States.

$$Area\ of\ waterway\ in\ sq.\ ft. = C \times (Drainage\ Area\ in\ acres)^{3/4}$$

Where:

C = 0.31 for an average condition; 0.20 for rural sections; 0.25 for farm country; 0.30 for village with lawns and macadam streets; 0.65 for ordinary city streets;

[12] Chow, V. T., *Hydrologic Determination of Waterway Areas for the Design of Drainage Structures in Small Drainage Basins.* Eng. Exper. Station Bull. 462, University of Illinois (1962).
[13] Talbot, A. N., *The Determination of Water-Way for Bridges and Culverts,* Selected Papers of the Civil Engineers' Club, Technograph No. 2, University of Illinois, 1887-8, pp. 14-22.

and 0.75 for paved streets and built-up business blocks; It should be increased for steep side slopes, especially if the upper part of the valley has a much greater fall than the channel at the culvert.

Talbot's paper indicated that culverts generally should not be designed to flow full or with the headwater submerging the entrance more often than once every 25 years. On minor and lightly traveled roads, overtopping of the roadway once in a few years may be of no consequence if the embankment is protected. For heavily traveled roads and on railroads, the size of the culvert opening should be such that while submerging the entrance may occur on rare occasions, overtopping of the roadway will never occur.

5.3.1.2 The Rational Method

The rational formula was developed primarily for estimating rates of runoff from urban areas. The formula was first mentioned in 1889 and was derived by measurements of rainfall and of the flow in the sewers of Rochester, NY, during the period from 1877 to 1888. The rational formula is:

$$Q = CIA$$

Where:

Q = discharge in cubic feet per second

C = runoff coefficient depending on characteristics of the drainage basin

I = rainfall intensity in inches per hour

A = drainage area in acres

Sewerage engineers were interested in waterway area determination primarily for the purpose of designing storm sewers. Values of "C" commonly recommended for design purposes varied as a function of the location of the overland runoff from 0.05 (sandy soils) to 0.98 (asphalt or concrete paved areas). The prevalent/recommended "C" values were reported by a joint committee of the American Society of Civil Engineers and the Water Pollution Control Federation[14].

The technical report determined these values of "C" to be applicable for storms of 5-year to 10-year frequencies.

5.3.1.3 The U.S. Geological Survey methods

In the late 1940's, cooperative stream gaging programs were begun between state highway departments and the U.S. Geological Survey to collect runoff data from selected small rural watersheds. The expectation that the data and experiences accumulated would provide a basis for the development of improved practical methods involving a

[14] *Design and Construction of Sanitary and Storm Sewers*, ASCE Manuals of Engineering Practice No. 37 and WPCF Manual of Practice No. 9, 1960.

stepwise multiple regression technique for predicting flood flows for small ungauged rural watersheds. The USGS hydrologic methods were based on the data collected by the numerous weather stations located throughout the Country's watersheds. The USGS applied regionalization techniques using basin and climatic variables are used to translate the results for use on ungaged areas. By the 1960's only 20-25 years of records have been catalogued for use in watershed peak discharge statistical analysis such as performed using Bulletin 15[15], that recommended fitting the log-Pearson Type III distribution to annual observed peak flow data by the method of moments. However, the limited 20-year data period also limited design to flood frequencies of 10-25 years with reasonable degree of uncertainty (confidence limits).

5.3.1.4 Other methods

Some state highway departments had adopted the Bureau of Public Roads method (BPR) methodology which required computation of a topographic index (T), a rainfall index (P-index), and identification of the zone in which the watershed is located. Basically, the value of the 10-year flood was estimated and the magnitudes of other flood frequencies were found through use of curves related to the 10-year flood. States that used this method were Virginia, Maryland, Pennsylvania, New York, Arkansas, Vermont, and Michigan.

The synthetic unit hydrograph of Snyder (1938) is based on relationships found between three characteristics of a standard unit hydrograph and descriptors of basin morphology. The hydrograph characteristics are the effective rainfall duration, the peak direct runoff rate, and the basin lag time. A modified version of the Snyder Method was widely used by the US Army Corps of Engineers in the 1960's and by National Flood Insurance Program (NFIP) contractors for the preparation of Flood Insurance Studies in the early 70's.

The United States Department of Agriculture - Soil Conservation Service (SCS) developed in the 1960's a method for estimating peak flows for small farm-type basins (less than 2,000 acres and watershed slopes less than 30 percent) using the rainfall volume associated with various durations and frequencies as published in Technical Report #40[16].

The U.S. Soil Conservation Service proposed another method of hydrograph synthesis for developing design hydrographs. This method involved the following steps:

(1) Take a maximum probable 6-hour point rainfall amount for the appropriate geographic allocation of the structure.

[15] U.S. Water Resources Council Bulletin 15, *A Uniform Technique for Determining Flood Flow Frequencies*, 1967.
[16] U.S. Department of Commerce, Technical Paper No. 40, *Rainfall Frequency Atlas Of The United States*, for Durations from 30 Minutes to 24 Hours and Return Periods from I to 100 Years, May 1962.

(2) Modify the 6-hour point rainfall amount to account for size of drainage area above the structure in accordance with a given synthetic rainfall depth-area relationship.

(3) Develop a rainfall hyetograph for the modified 6-hour point rainfall in accordance with a given synthetic hyetograph distribution pattern.

(4) Determine the hydrologic soil-cover complex number of the drainage basin above the structure. The number shows the relative value of the hydrologic soil-cover complexes as direct runoff producers.

(5) Determine the direct runoff Q in inches by the following formula:

$$Q = \frac{\left(\frac{R - 200}{N + 2}\right)^2}{\left(\frac{R + 800}{N - 8}\right)}$$

Which later became:

$$Q = \frac{(R - 0.2S)^2}{R + 0.8S}$$

Where:

D = Direct runoff in inches

R = rainfall in inches

N = hydrologic soil-cover complex number (Same as CN)

S = Soil Storage = (1000/CN) -10

The higher the number, the greater the amount of direct runoff to be expected from a storm. The numbers for various land uses, crop treatments, hydrologic condition, and hydrologic soil groups were prepared using data from gaged drainage basins with known soils and cover.

The determination of the soil-cover complex number (Curve Number or CN) is done with reference to both soil cover and soil type. The soil cover as described from the hydrologic point of view, is given as either good / fair / or poor, depending on the infiltration capacity. A soil cover of high, medium, or low infiltration capacity is described as being of good, fair, or poor condition respectively. The soil types are classified on the basis of intake of water at the end of long-duration storms occurring after prior wetting and opportunity for swelling, and without the protective effects of vegetation. The major hydrologic soil groups are:

- Type A – (lowest runoff potential) includes deep sands with very little silt and clay; also deep, rapidly permeable loess.

- Type B – includes mostly sandy soils less deep than type A, and less deep or less aggregated than type A, but the group as a whole has above-average infiltration after thorough wetting.
- Type C – comprises shallow soils and soils containing considerable clay and colloid, though less than those of type D. This type has below-average infiltration after pre-saturation.
- Type D – (highest runoff potential) includes mostly clays of high swelling percent, but the type also includes some shallow soils with nearly impermeable sub horizons near the surface.

A classification of about 2,000 major soils of continental United States into the above four types was made available by the Service.

(6) Obtain the direct runoff from the previous step for uniform time intervals in the synthetic hyetograph

(7) Compute the time to peak (T) and peak discharge (q) of a triangular hydrograph for the direct runoff in each time interval of the hyetograph by the following equations:

$$Q_p = \frac{484AQ}{T_p}$$

Where:

Q_p = Peak flow rate in cubic feet per second

T_p = Time from beginning of direct runoff to peak in hours

A = Drainage area in square miles

Q = Direct runoff in inches

Peak discharge was related to drainage area, rainfall amounts for three types of rainfall time distributions (6, 12 and 24 hours), three categories of average watershed slopes, and watershed characteristics (land-use practices, hydrologic conditions, and hydrologic soil groups A, B, C, D). Charts were developed for easy application of the SCS method.

$$T_p = \frac{2}{3}T_c$$

Where:

T_p = Time from beginning of direct runoff to peak in hours

T_c = Time of concentration in hours = $5/3\ Lag$

And

$$Lag\ (hours) = \frac{L^{0.8} \times (S + 1)^{0.7}}{1900 \times Y^{0.5}}$$

Where:

L = Hydraulic length along flow path from furthest upstream point to outlet

Y = Watershed Slope along flow path in percentage

5.3.2 Hydrologic Design Criteria Summary

Analysis of the 1960's prevalent hydrologic analysis methodology indicates that a return design frequency of less than 30 years, but mostly 10 years, was typically used for the design of most culvert crossings. The concept of culvert design using larger (50- or 100-year) return frequencies was not yet fully established.

The use of a 50-year return frequency for culvert design did not become prevalent until the implementation of the Federal Interstate Roadway System Projects. The Interstate road program technical design guidelines indicated that the design of all culverts and bridges over streams shall be in accordance with the Standard Specifications for Highway Bridges of the American Association of State Highway Officials, 9th Edition 1965, to accommodate floods at least as great as that for a 50-year frequency or the greatest flood of record, whichever is greater, when the runoff was based on the 20-year projected land development expected in the watershed and with backwater limited to an amount which would not result in damage to upstream property or to the highway.

The concept of a 100-year return frequency for the establishment of floodplains did not become known or required until the late 1960's. The National Flood Insurance Program (NFIP) was created by the Congress of the United States in 1968 through the National Flood Insurance Act of 1968 (P.L. 90-448). The two-fold purposes of the NFIP were to share the risk of flood losses through flood insurance and to reduce flood damage by restricting floodplain development.

The program enabled property owners in participating communities to purchase insurance protection, administered by the government, against losses from flooding, and required flood insurance for all loans or lines of credit that are secured by existing buildings, manufactured homes, or buildings under construction, that were located in the Special Flood Hazard Area (SFHA) in a community that participates in the NFIP. The NFIP defined the floodplain as the area that would be flooded by a base flood, which is "the flood which has a one percent chance of being equaled or exceeded in any given year." In this sense, a base flood is synonymous with a 100-year flood and a floodplain is synonymous with a special flood hazard area.

Some additional technical literature on the design return period used by cities in their design of storm sewer systems and culverts was available from a February and March of 1967 survey that was taken of the storm drainage practices of thirty-two cities in the state of Wisconsin. These cities each had a 1960 population of 6000 or more and represented

41% of the State's total population. The frequency levels employed in design by the various reporting cities varied from one year to twenty-five years, with 70% reporting five to ten years and 20% using two to four years.

5.4 Original Design Level of Service Determination

The research summary of the historical technical methodology applied in the early 1960s strongly indicates that the Halpine Road relocation culverts were designed using a level of service associated with a 10-year return frequency. To make this determination this study recreated the culvert design calculation using data shown in section 5.3 Original Culvert Design Assessment, Figure 73 for the Tributary 2 culvert.

5.4.1 Waterway (Culvert) Area

Figure 73 indicates that a culvert area of 105 square feet was calculated using the Talbot formulation.

$$105 = C \times (473\,ac)^{3/4}$$

Solving for C yields:

C = 1.0

Assessment:

<u>The culvert's waterway area was indeed calculated using the Talbot's formulation.</u> The C value of 1.0 is within the technical literature findings that "C" should be increased (0.75 to 1.0) for steep side slopes, especially if the upper part of the valley has a much greater fall than the channel at the culvert, as is the case in the Tributary 2 drainage area.

5.4.2 Peak Discharge

Figure 73 indicates that a peak discharge of 1031 cubic feet second (cfs) was calculated for culvert sizing for a watershed area of 473 acres (0.74 square miles). A few methods will be assessed to recreate this value.

Using Rational Formula:

$$Q = CIA$$

$$I = Rainfall\ Intensity = 1.8\frac{in}{hr}$$

(From Bulletin TP-40) for a 10-year frequency

$$1031 = C \times 1.8 \times 473$$

Solving for C yields:

C = 1.2

(This value is beyond the technical literature range of 0.05 to 0.98)

Assessment:

<u>The Rational Method is not the source of the peak discharge calculation.</u>

This study also ruled out regression equation methods and the Modified Snyder's Method as these produced excessive 10-year storm event discharges. This study tested the use of the SCS formulation as follows:

Tributary 2 drainage Area: 473 acres = 0.74 Square Miles (per Figure 73)

Elevations: 446 feet (upstream most), 283 feet (culvert entrance)

CN = 84 (weighted, land cover has not changed significantly since the 1970's)

L = Hydraulic length along flow path from furthest upstream point to outlet (per Figure 76)

L = 7250 Feet

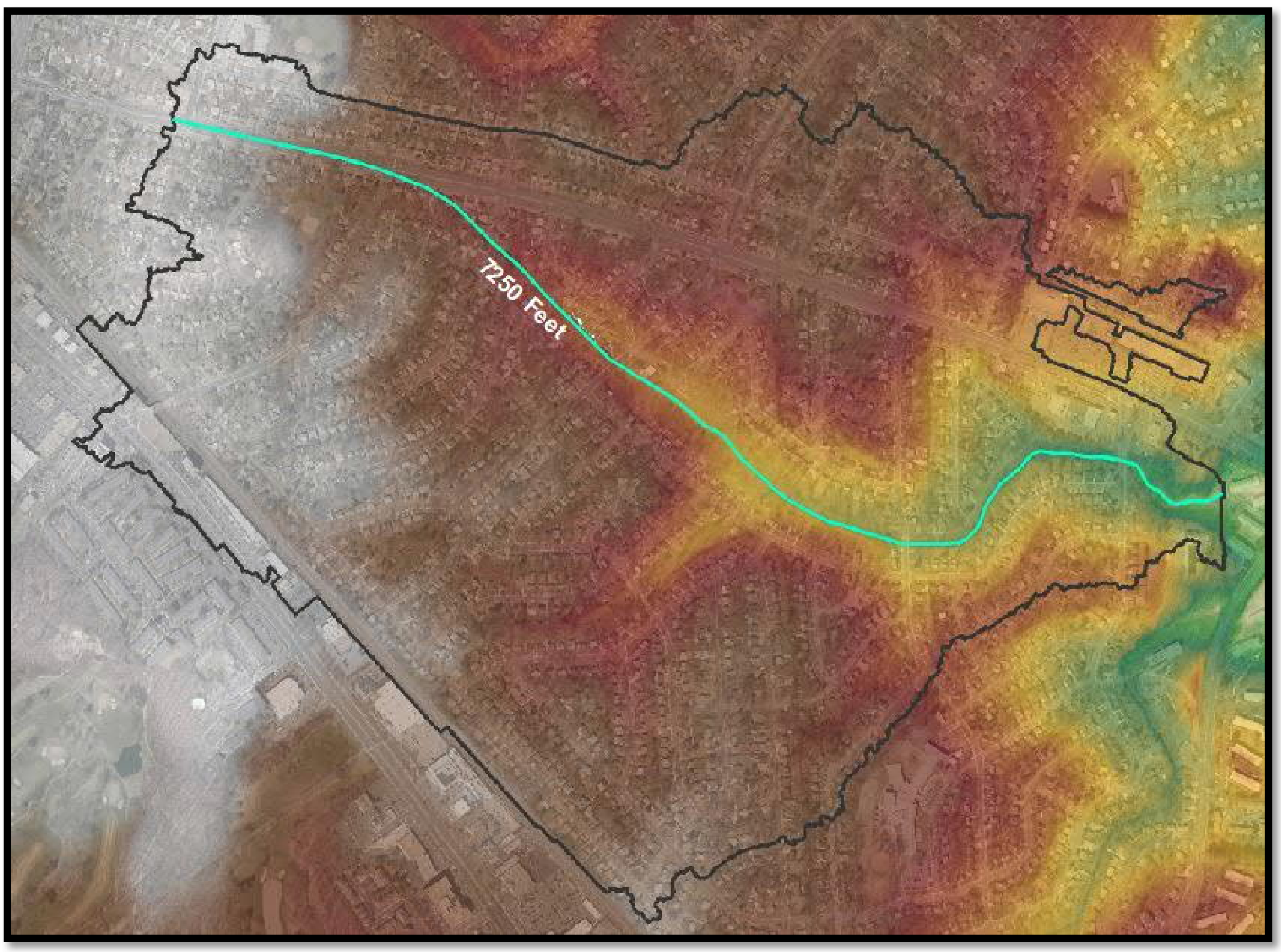

Figure 76 - Hydraulic length of Tributary 2

Rainfall: (Using the 1961 TP40 Rainfall Frequency Atlas of The United States for durations from 30-minutes to 24-hours and return periods from 1- to 100-years)

For the 10-year storm return frequency the following rainfall volumes are extracted from TP40 curves.

- 10-year 6-hour: 4.0 inches
- 10-year 12-hour: 4.5 inches
- 10-year 24-hour: 5.0 inches

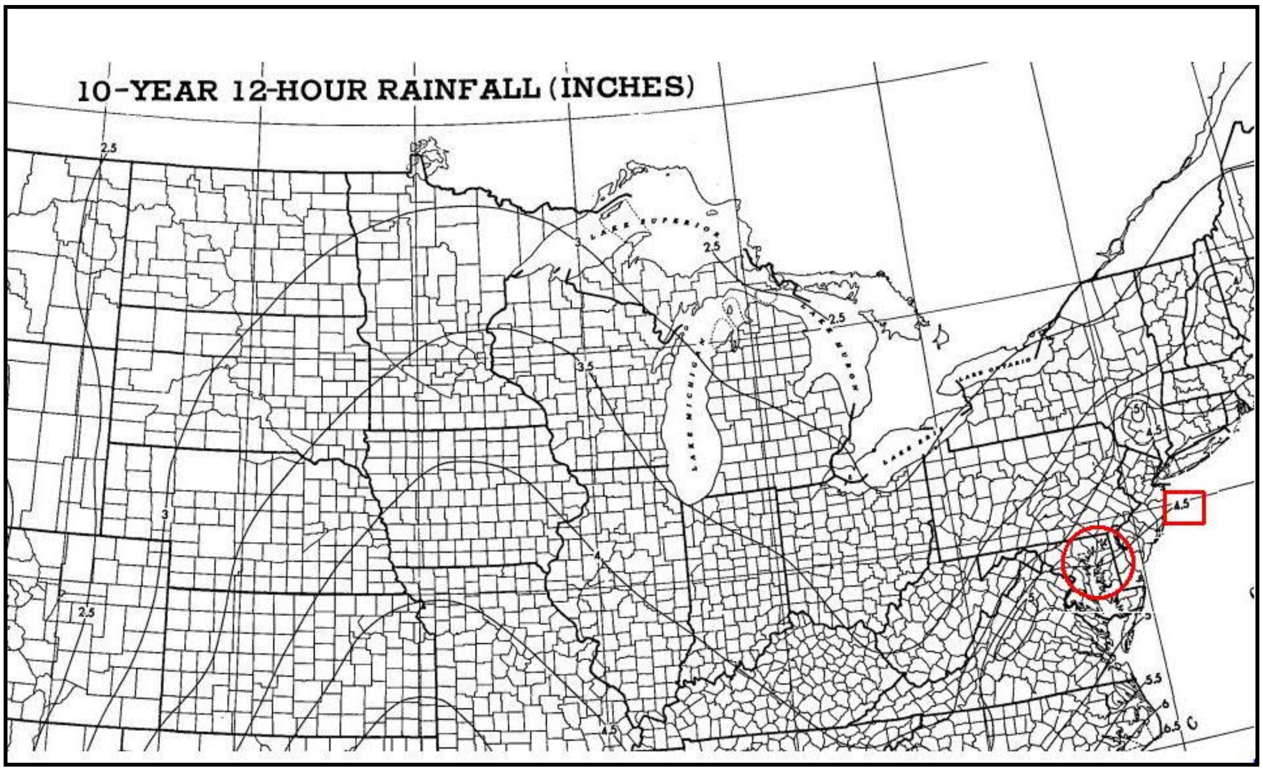

Figure 77 - 10-year 12-hour Rainfall Volume (Source: TP40)

$$S = \frac{1000}{84} - 10 = 1.9$$

$$Y = \frac{446 - 283}{7250} = 0.0225\frac{ft}{ft}\ or\ 2.25\%$$

$$Lag = \frac{7200^{0.8} \times (1.9 + 1)^{0.7}}{1900 \times 2.25^{0.5}} = 0.90$$

$$Q = \frac{\left(4.5 - (0.2 \times 1.9)\right)^2}{4.5 + (0.8 \times 1.9)} = 2.82\ inches$$

$$Q_p = \frac{484 \times 0.74 \times 2.82}{2/3 \times 5/3 \times 0.9} = 1{,}010\ cfs$$

<u>The application pf the SCS direct runoff and triangular hydrograph methodology results in a peak discharge of 1,010 cfs which closely matches the 1,031 cfs peak discharge in Figure 73.</u>

Whether this methodology or similar was used to calculate the 1031 cfs peak discharge, the use of 4.5 inches of rainfall associated with the 10-year, 12-hour rainfall storm event indicates that the designer intended to use the 10-year return frequency as the base for Level of Service design. This study will perform actual (2021) ICPR4 hydrodynamic simulation of the existing culvert flood-carrying level of service capacity to assess the validity and accuracy of this premise.

5.5 Hydraulic Design

5.5.1 Maximum Flow Depth (Prevalent Inlet vs Outlet Control Condition)

Figure 73 shows that maximum flow depths of 8.93 feet (Inlet) and 6 feet (Outlet) were calculated using hydraulic methods available in the 1960's.

In the 1960's, most municipalities adhered to the Bureau of Public Roads (BPR) methodology. The Bureau of Public Roads was established in the Department of Agriculture in 1893 to established guidelines for roadway culvert design based on local/regional characteristics.

The American Association of State Highway and Transportation Officials (AASHTO) also provided design criteria for culvert crossings and bridges. The AASHTO Standard Specifications for Highway Bridges is a technical publication available since the early 1920's. The compilation of these specifications began in 1921 with the organization of the Committee on Bridges and Structures of the American Association of State Highway Officials. The first edition of the Standard Specifications was published in 1931 and a revised edition of this book was generally published every 4 years including 1961, 1965 and 1969.

Beginning in 1964, the Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC) began research, training, and technical assistance in the application of hydrologic engineering methodology for waterways and culvert crossings. The HEC worked closely with the Office of Bridge Technology, National Highway Institute, Federal Highway Administration. Hydraulic Design of Highway Culverts, Third Edition currently provides the current methodology for culvert design. Series Number 5 (HDS 5) originally merged culvert design information contained in Hydraulic Engineering Circulars (HEC) 5, 10, and 13 with other related hydrologic-, storage routing-, and special culvert- design information. This third edition is the first major rewrite of HDS 5 since 1985, updating all previous information and adding new information on software solutions, aquatic organism passage, culvert assessment, and culvert repair and rehabilitation.

The headwater computation depends on the results of inlet control depth and outlet control depth computations. The larger depth of the two will govern and be used to compute the headwater elevation. If water can flow through and out of the culvert faster than it can enter, the culvert is under Inlet Control. Flow capacity is controlled at the entrance by the headwater depth, cross-sectional area, and type of inlet edge. In inlet control the flow passes through critical depth at the culvert entrance and is supercritical in the barrel.

If water can flow into the culvert faster than it can flow through and out, then it is under Outlet Control. Outlet control means that flow through the culvert is limited by friction between the flowing water and the culvert barrel. Full or partial flow can pass through the culvert and critical depth typically occurs at the culvert outlet.

Current FHWA HY8 culvert sizing software automates the design methods described in HDS No. 5, "Hydraulic Design of Highway Culverts", FHWA-NHI-12-029 and in HEC No.14, FHWA-NHI-06-086. There are several flow profiles possible and HY8 simulates Type A, B, C, and D conditions described in HDS-5. The HY8 culvert design tool was applied using the Figure 73 Tributary 2 hydraulic data to reproduce the original 1962 culvert design hydraulic conditions. Figure 78 through Figure 80 show the HY8 input data.

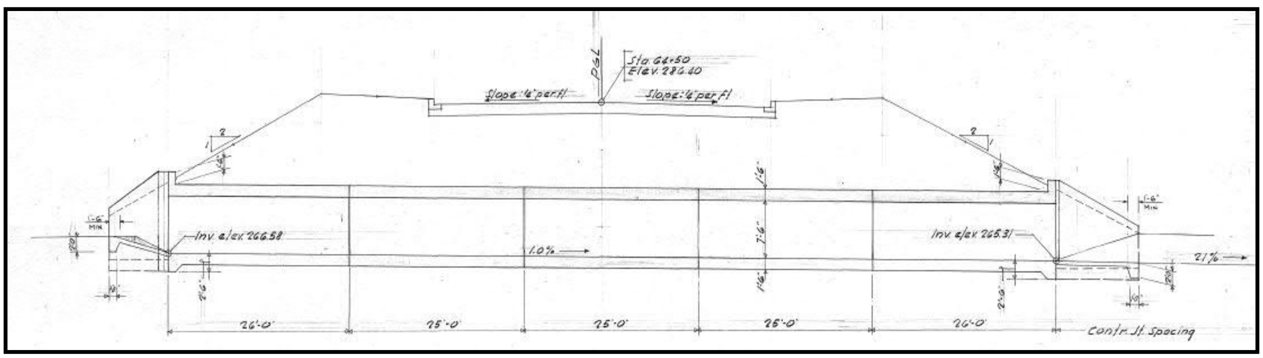

Figure 78 - Original Tributary 2 Culvert Profile (1962)

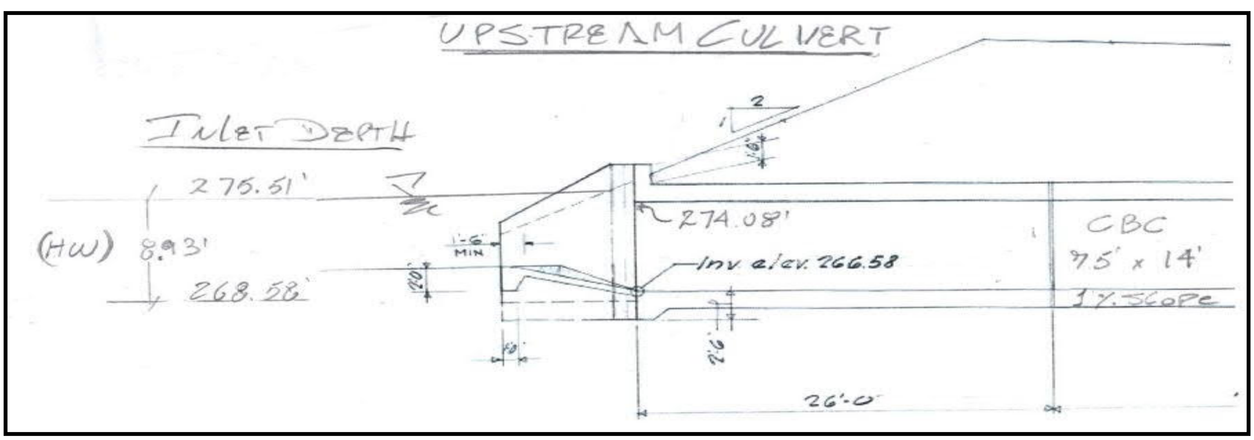

Figure 79 - Original Tributary 2 Upstream Culvert Data (1962)

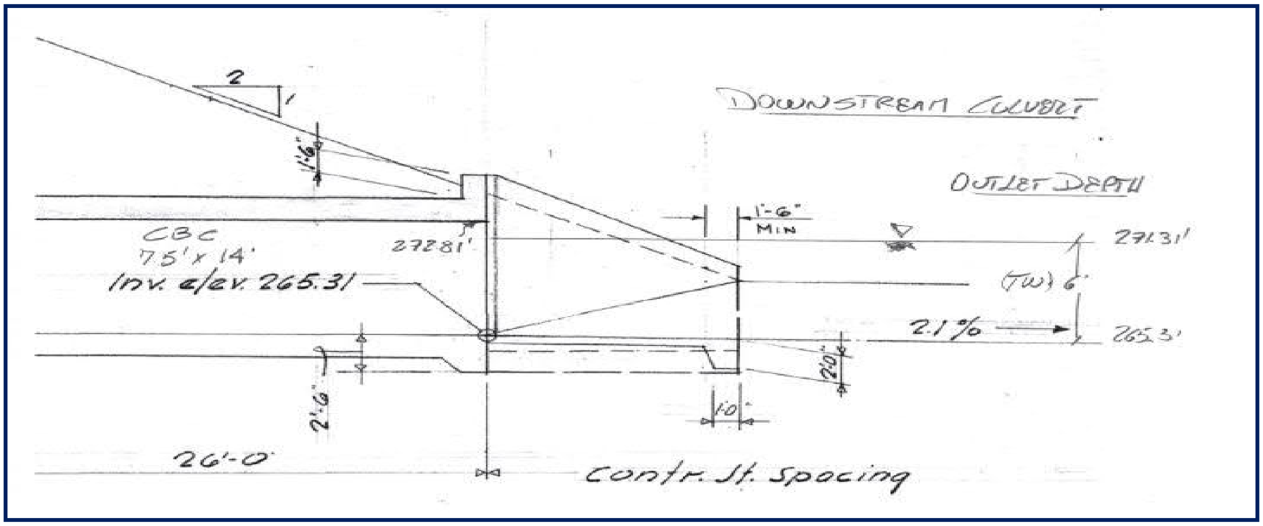

Figure 80 - Original Tributary 2 Downstream Culvert Data (1962)

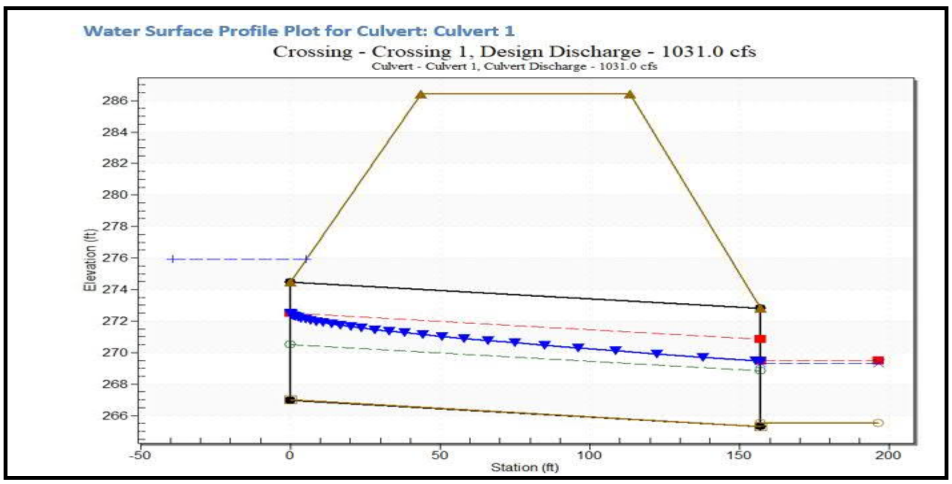

Figure 81 - HY8 Culvert Water Surface Performance Profile Plot

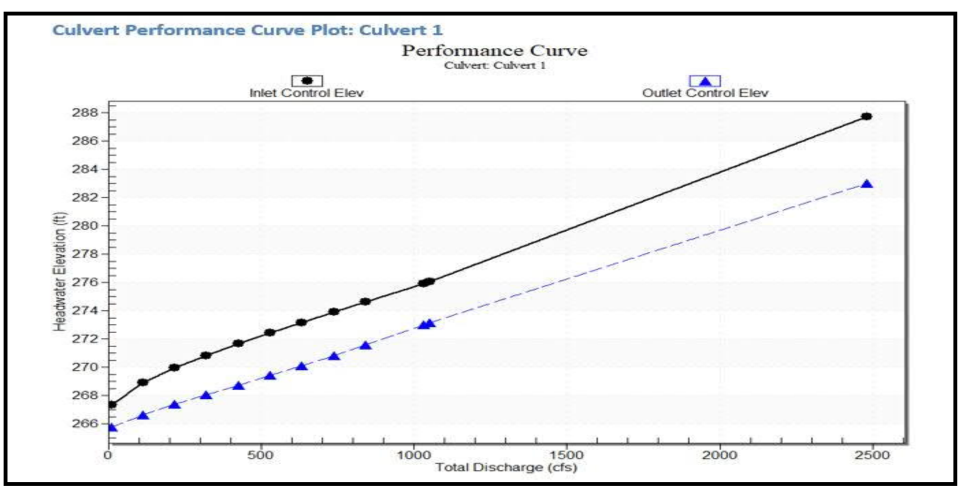

Figure 82 - HY8 Culvert Performance Rating

Culvert Crossing: Tributary 2 Culvert

Water Surface Profiles

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	Flow Type	Length Full (ft)	Length Free (ft)
10.00	10.00	267.36	0.37	0.0*	1-JS1t	0.00	157.00
114.00	114.00	268.91	1.92	0.0*	1-S2n	0.00	157.00
218.00	218.00	269.94	2.96	0.39	1-S2n	0.00	157.00
322.00	322.00	270.83	3.84	1.08	1-S2n	0.00	157.00
426.00	426.00	271.66	4.67	1.76	1-S2n	0.00	157.00
530.00	530.00	272.44	5.45	2.44	1-S2n	0.00	157.00
634.00	634.00	273.18	6.19	3.14	1-S2n	0.00	157.00
738.00	738.00	273.90	6.91	3.85	1-S2n	0.00	157.00
842.00	842.00	274.61	7.62	4.59	5-S2n	0.00	157.00
1031.00	1031.00	275.93	8.95	6.00	5-S2n	0.00	157.00
1050.00	1050.00	276.07	9.08	6.15	5-S2n	0.00	157.00

Figure 83 - HY8 Water Surface Profiles Table

Figure 83 shows that a headwater (HW) of 8.95 feet for inlet condition and 6.0 feet for outlet conditions was calculated using the original Tributary 2 culvert data. The HDS5 Flow Type Condition 5-S2n corresponds to inlet control governing with supercritical outlet flow.

These values are almost identical to the original 1962 maximum flow depth design of HW(Inlet) = 8.93 feet and HW(Outlet) = 6.0 Feet. <u>Therefore, it can be concluded that the original culvert design was based on an inlet headwater condition that did not overtop the road, and conveyed the peak discharge through the culvert with an outlet velocity representative of supercritical flow regime.</u>

5.5.2 Outlet Velocity

Figure 73 shows that an outlet velocity of 12.27 feet per second (fps) was calculated for the inlet control condition. This velocity is representative of supercritical flow regime mentioned above.

Outlet velocity is calculated by $Q = V \, x \, A$ for inlet control conditions where the culvert slope is not a factor. Where:

Q = Peak Discharge

A = culvert cross sectional area

From Figure 73:

Q = 1031 cfs

A = 105 square feet for a maximum 7.5 culvert depth

The maximum flow depth for outlet condition was calculated as 6 feet. Therefore, the partial area of flow is:

A = 6/7.5 = 0.8 or 80%

1031 = V x 0.80 (105) Solving for V

V = 1031/ 0.80 x 105

V = 12.27 fps

<u>The culvert's exit or outlet velocity was indeed calculated using the inlet condition supercritical outlet flow condition (no slope or length friction losses).</u>

5.5.3 1966 Culvert Enclosure Extension

No hydraulic data was included on the 1966 permit plans for the culvert extension (enclosure) and storm drain layout of the apartment complex; however, both permit sets were designed by Greenhorne & O'Mara Engineers. Because both plan sets were created within a four-year timeframe by the same engineering firm, it is reasonable that the same design methodology was used for both designs. The storm drain project entailed enclosing the tributary open channels with 5 feet by 10 feet concrete box culverts to redevelop the Bullis properties to make way for the construction of the Rock Creek Woods Apartment buildings on the subject property. These culverts connect with two previously constructed box culverts that run beneath Twinbrook Parkway (formerly Halpine Road), and later join at an outfall to the southeast of the property. Figure 84 shows the enclosure plans which were prepared by Greenhorne & Omara, the same engineers used for the original Halpine Road relocation culvert design.

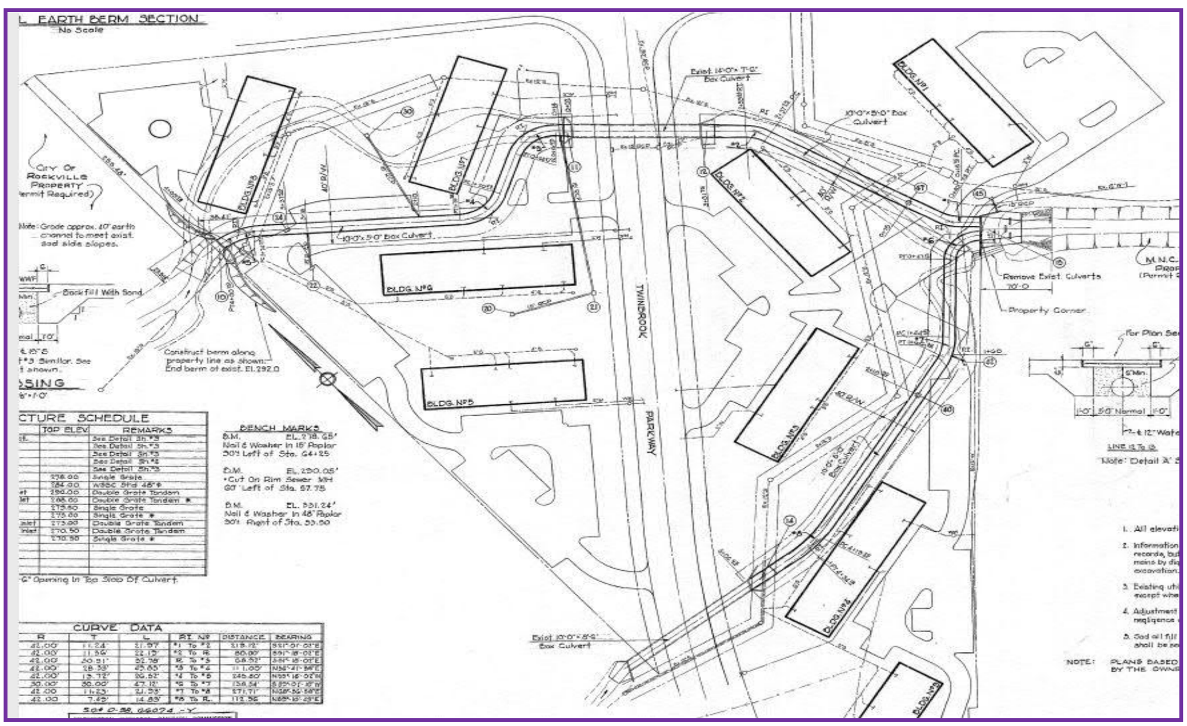

Figure 84 - Enclosure Plan for Twinbrook Tributaries 1 and 2 (Source: G&O, 1966)

There is a need to understand the selection of smaller 5 feet x 10 feet culverts to enclose the open channels upstream and downstream of the existing 7.5 feet x 14 feet culverts at Twinbrook Parkway. Unlike the original (1962) Halpine Road Relocation drawings, the 1966 tributary enclosure plans do not provide a basis of design information (i.e., hydrologic/hydraulic data). Therefore, an assessment rather than a reproduction of results of the enclosure plan drawings will be made to ascertain design intentions. The following figures show the plan and profile details for Tributary 2 west and east enclosure segments.

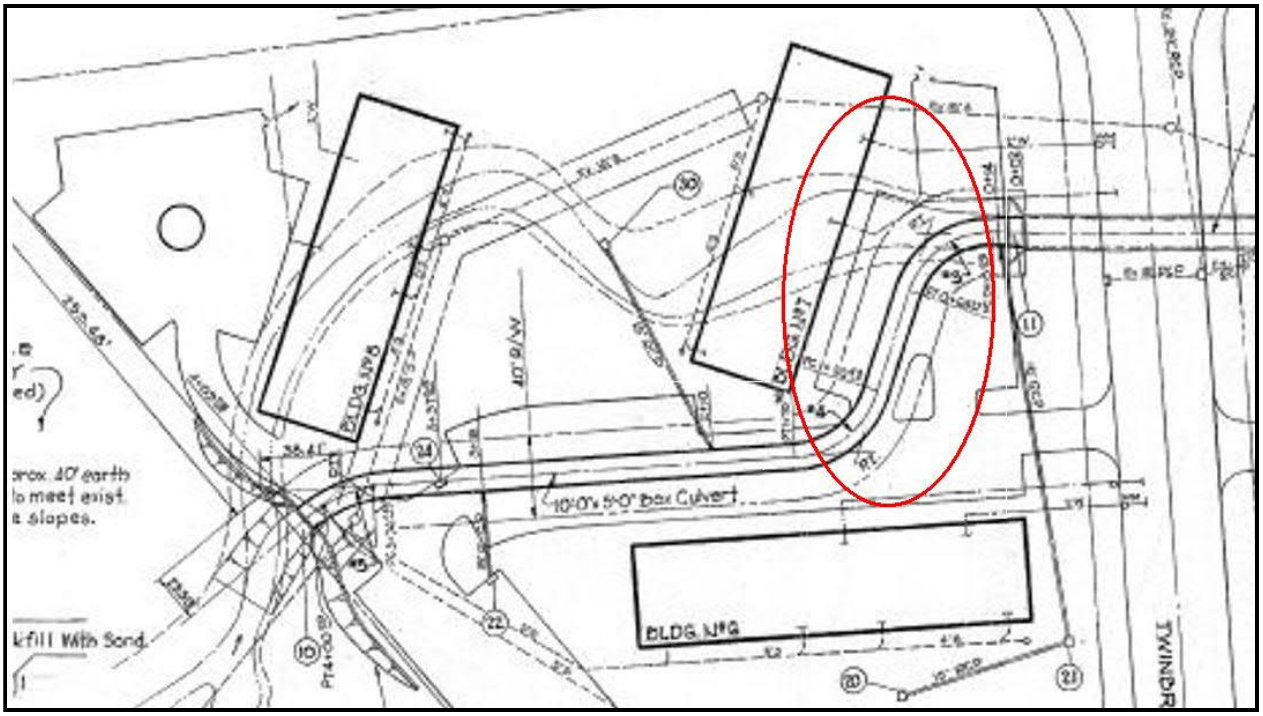

Figure 85 - Tributary 2 Culvert Enclosure Plan View of Western Segment. Note the change in horizontal alignment (circled).

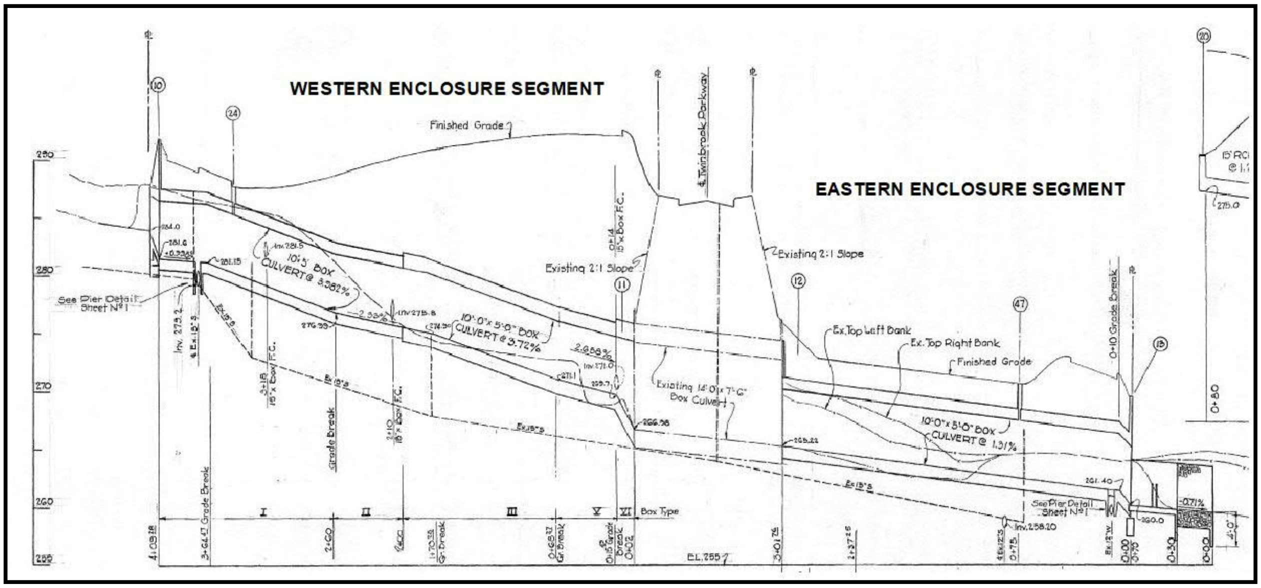

Figure 86 - Tributary 2 Culvert Enclosure Profile View of Western and Eastern Segments

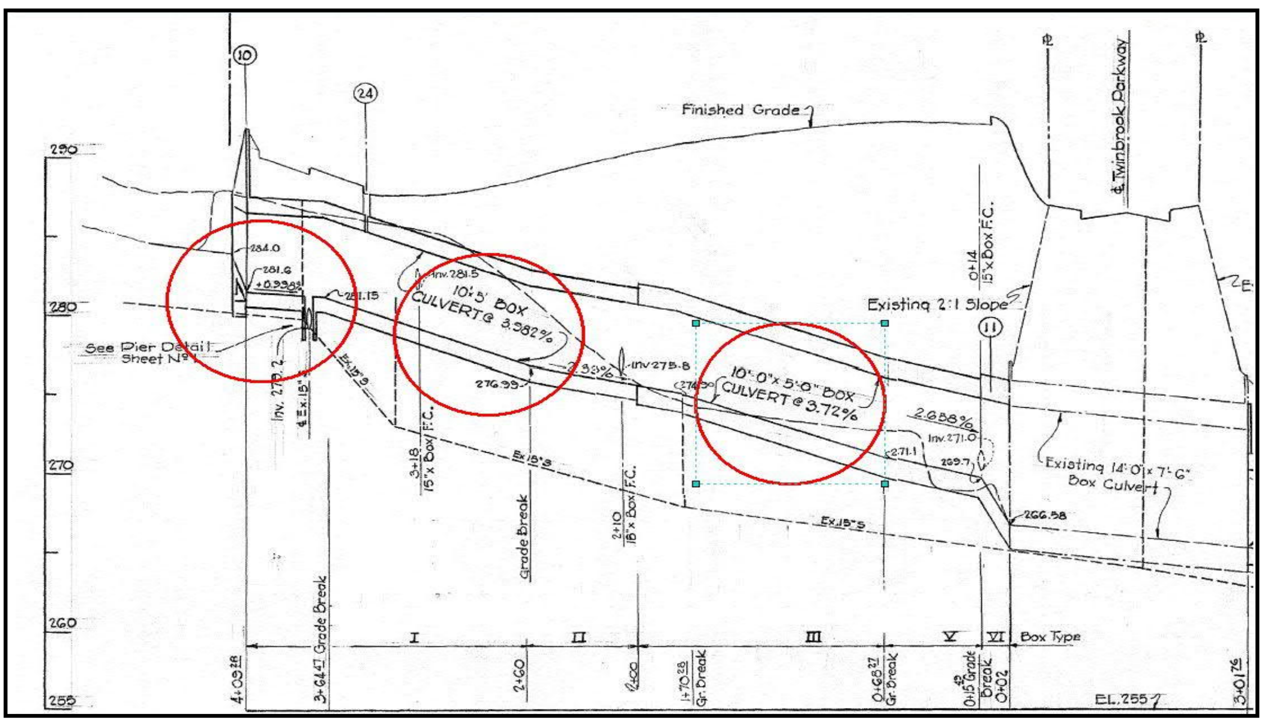

Figure 87 - Tributary 2 Culvert Enclosure Profile View of Western Segment. Note the changes/breaks in profile slope (circled).

Figure 87 shows that the western enclosure was designed as a “broken back” culvert. A broken-back culvert has one or more breaks in profile slope and is most often used in areas of high relief and steep topography. The usual broken-back configuration begins with a relatively flat slope followed by a steep slope and a very flat "runout" section. Figure 87 shows that three main culvert segments with slopes of 0.99%, 3.98% and 3.72% were designed for a culvert length of approximately 409 feet. Figure 85 also indicates that the horizontal enclosure alignment was designed with major offset angles for flow conveyance.

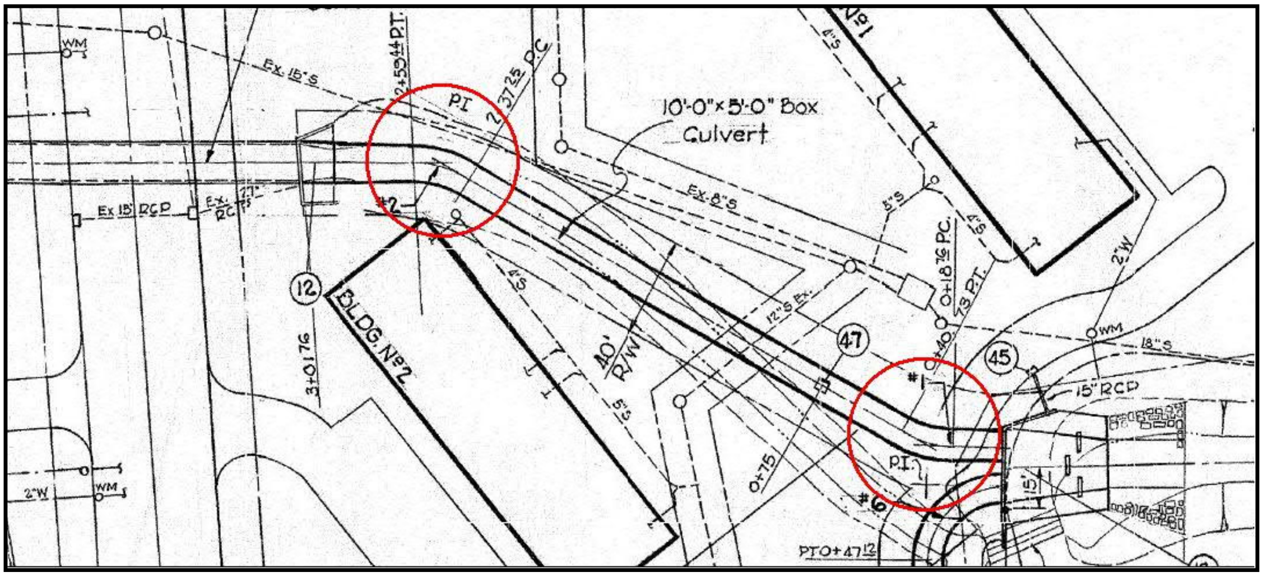

Figure 88 - Tributary 2 Culvert Enclosure Plan View of Eastern Segment

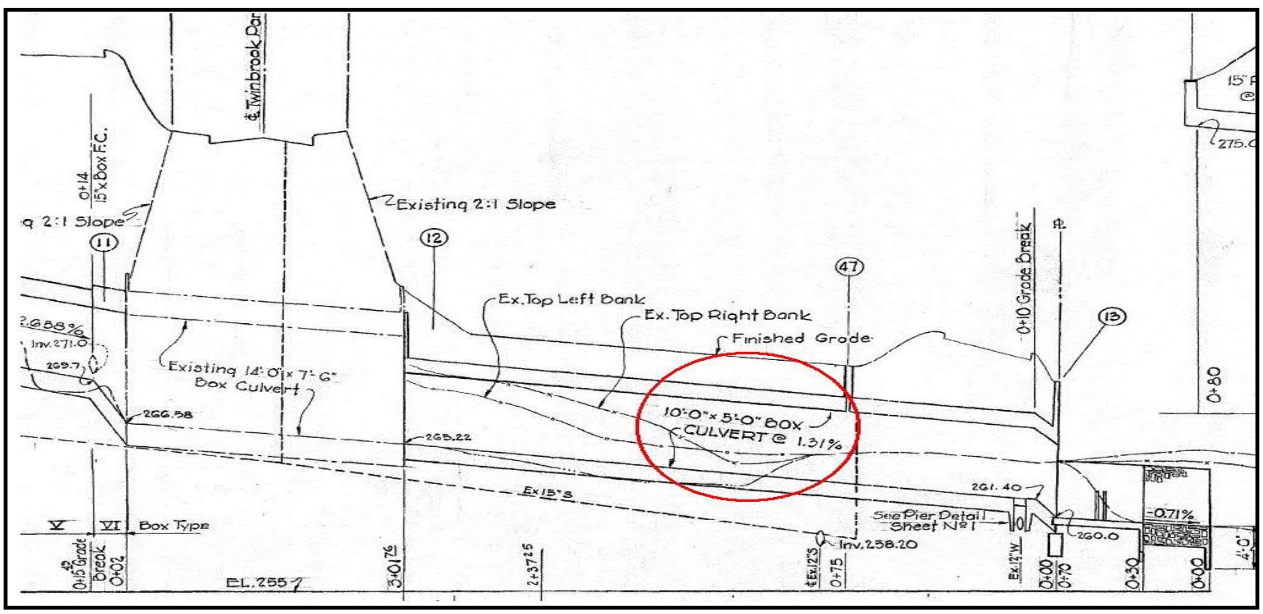

Figure 89 - Tributary 2 Culvert Enclosure Profile View of Eastern Segment

Figure 89 shows that the eastern enclosure segment was designed with a single slope of 1.31% with a culvert length of approximately 301 ft. Figure 88 indicates that the horizontal enclosure alignment was designed with two minor deflection angles for flow conveyance.

The vertical and horizontal alignment features exhibited in Figure 85 to Figure 89 are indicative of a complex hydraulic enclosure design with numerous hydraulic losses for flow regime and headwater calculations.

5.5.4 Tributary Area and Peak Discharge

As previously indicated, for typical municipal early 1960's applications, the culvert design discharge was frequently associated with the 10-year (10% annual chance) storm event. This study established that the 10-year return frequency was used for the calculation of a 1031 cfs peak discharge for a 473 acre (0.74 square mile) catchment area during the Halpine Road Relocation culvert design in 1963. The calculation of a peak discharge for the enclosure of Tributary 2, just a few years later by the same consultant, would have applied a similar drainage area as the enclosure length of 409 feet to Twinbrook Parkway did not significantly affect the contributory drainage area of 473 acres and the resulting 1031 cfs peak discharge calculation.

The enclosure did take into consideration the steepness length (3-4% average) of the 409 feet enclosure length as three different slopes were applied in the enclosure design. The use of multiple or segmental slopes in the enclosure is indicative that the designer considered a smaller (10 feet x 5 feet) concrete box culvert (CBC) at a steeper slope to pass the 1031 cfs peak at full flow through the proposed culvert enclosure (i.e, a culvert discharge increases with slope).

Using Manning's full flow equation for open channel enclosures using a box culvert:

$$Q = \frac{1.486}{n} A {R_h}^{2/3} S^{1/2}$$

Where:

Q = Flow rate passing through the box culvert in cubic feet per second

A = Cross-sectional area of flow normal to the flow direction in feet

S = Bottom slope of the culvert in feet/foot (dimensionless)

n = Manning's Roughness coefficient

R_h = Hydraulic radius = A/P

P = Wetted perimeter of the cross-sectional area of flow in feet

For:

N = 0.012

A = 10 x 5 = 50 sf (for 10-ft x 5-ft CBC)

P = 30 ft (for 10-ft x 5-ft CBC)

$R_h = A/P = 50/30 = 1.67$

$$S = \frac{(0.99\% \times 45) + (3.98\% \times 164) + (3.72\% \times 200)}{409}$$
$$= 3.52\%\ average\ over\ 409\ feet$$

$$Q = \frac{1.486}{0.012}(50)(1.67)^{2/3}(0.0352)^{1/2} = 1{,}641.3\ cfs$$

Assuming that the culvert would be partially full (50% conservatively) due to inlet/outlet conditions:

R_h (for 50% depth of flow) = 1.67 x 0.5 = 0.835

$$Q = \frac{1.486}{0.012}(50)(0.835)^{2/3}(0.0352)^{1/2} = 1{,}031.5\ cfs$$

The Manning's full and partial full flow calculations would have indicated to the designer that the project 1031 cfs peak discharge requirement for enclosure design could be met using a 10-ft x 5-ft CBC at an average slope of 3.5%.

5.5.5 Headwater

Culverts in the early 1960s and today are typically designed to pass the specified design discharge without creating an unacceptably high headwater depth. Thus, for an engineer to design a culvert successfully, the headwater depth for the design discharge must be reliably predicted. Knowledge of the headwater depth associated with a particular flow condition will indicate whether or not the culvert will pass the design flow safely without overtopping the embankment or violating applicable state or local regulations. The definition of an unacceptable headwater depth varies among municipalities, but typically, the maximum headwater elevation should be about 1 or 2 feet lower than the roadway shoulder elevation or embankment to minimize the potential for roadway flooding.

In the early 1960s there were few tools available for the calculation of complex culvert hydraulics such as found at the Twinbrook open channel enclosure. The inlet vs outlet condition analyses were relegated to manual (slide rule) equation calculations and the use of charts and nomographs. However, none of the available methodologies address the design of over 800 feet of open channel enclosure with a 10-foot by 5-foot concrete box culvert (CBC), mild to steep slopes, and meandering layouts.

It is possible that the designer would have selected a segmental calculation approach where the maximum headwater was calculated for the first 45 feet flat (0.99% slope) segment. The assumption would have been made that once the initial discharge entered the 10-ft x 5-ft CBC the next two steeper culvert segments (3.58 and 3.72%) would effectively convey the flow to the existing 14-foot x 7.5-foot CBC and beyond into the eastern enclosure, even at half full flow. Figure 90 shows the vintage 1963 Bureau of Public Works (BPW) Chart 15 “Head for Concrete Box Culverts Flowing Full, 0.012” that the designer would have applied to calculate the headwater at the entrance to the 10-ft x 5-ft CBC.

Chart Data:

Discharge = 1031 cfs

For a 10-ft x 5-ft CBC with A= 50 sf, K_e = 0.2 (Beveled headwall w/ 30^0 to 75^0 wingwalls)

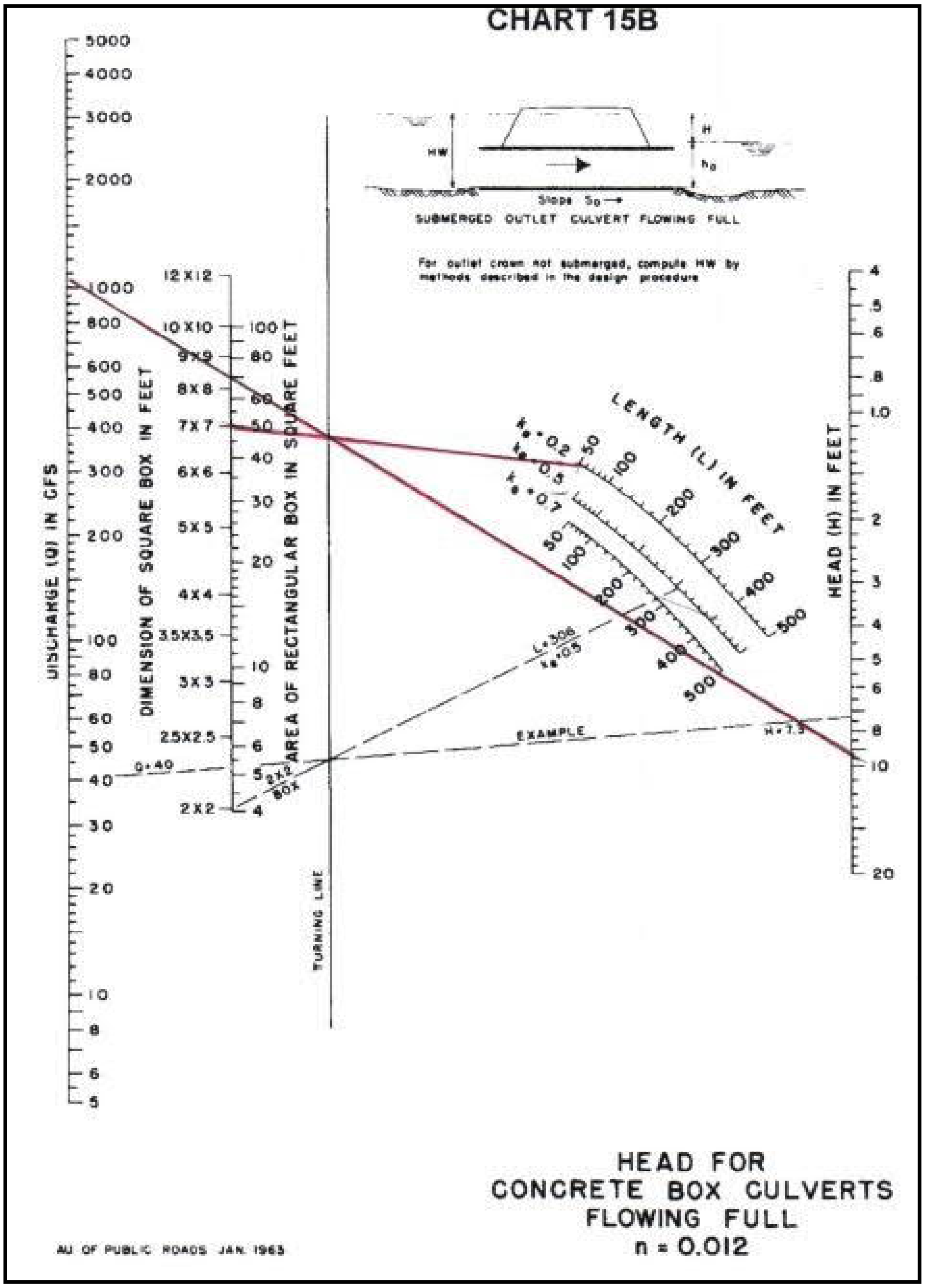

Figure 90 - Headwater for Full Flow Through 10-ft x 5-ft CBC

The Figure 90 chart indicates that a headwater of approximately 9 feet would occur for a full flow condition through the 10-foot x 5-foot CBC.

The Figure 90 results assumed full flow through the culvert as a function of conduit slope. As a cross check, the designer would have calculated the headwater for the proposed 10-foot x 5-foot CBC for inlet control conditions that do not depend on conduit characteristic other than waterway opening area. Figure 91 shows that using the BPR Chart 8B “Headwater Depth for Culverts with Inlet Control” with the same data yields a headwater of approximately 19.25 feet. (Hw/D= 3.85 or Hw = 3.85 x 5 = 19.25).

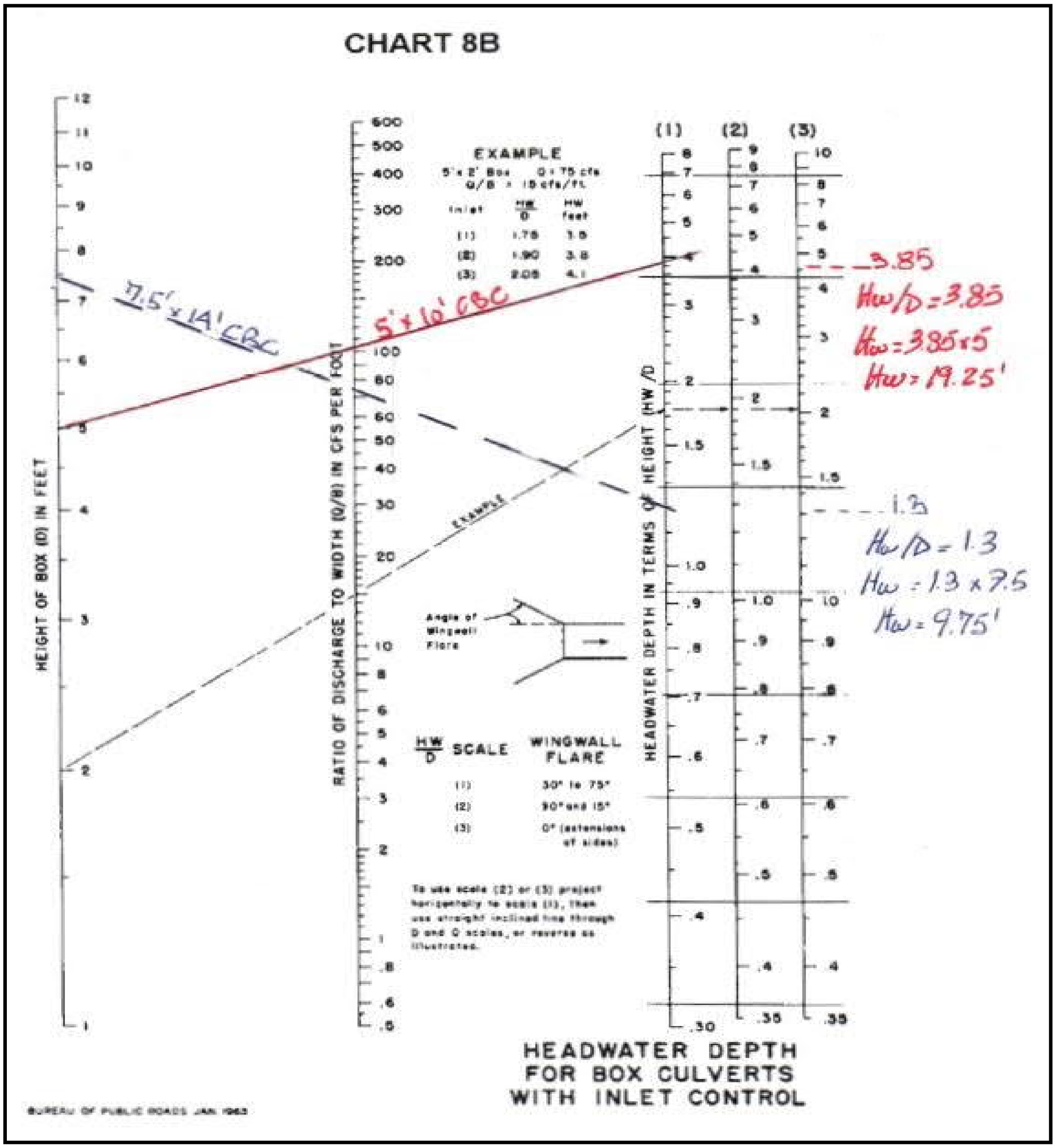

Figure 91 - Headwater for Inlet Control Through 10-ft x 5-ft CBC

Figure 91 also includes an alternate assessment of headwater with inlet control for a 14-ft x 7.5-ft CBC to match the existing culvert at Twinbrook Parkway. Chart 8B indicates a headwater of approximately 9.75 feet. (Hw/D= 1.3 or Hw = 1.3 x7.5 = 9.75).

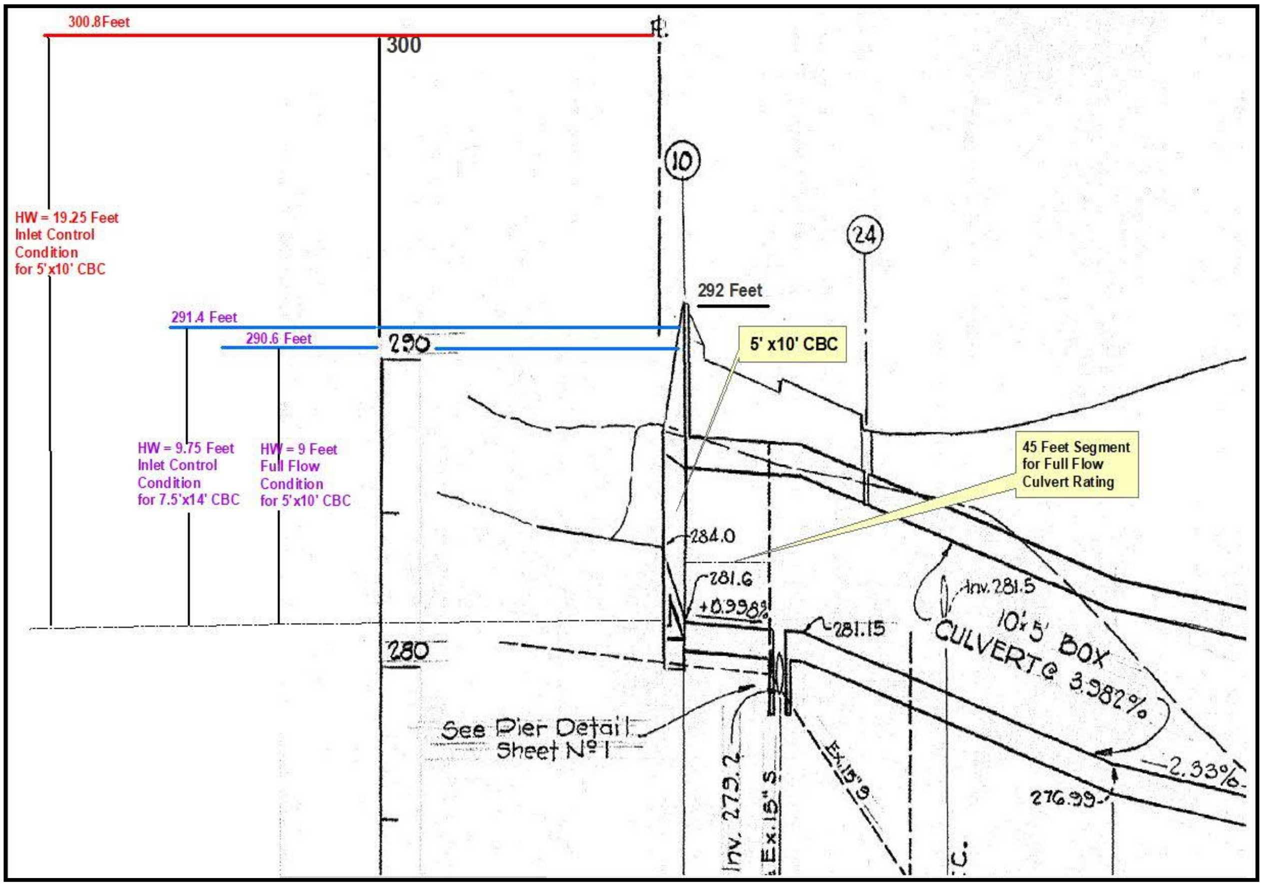

Figure 92 - Headwater Depth for Full Flow and Inlet Control for 10-ft x 5-ft & 14-ft x 7.5-ft Culverts

Figure 92 includes all the calculated headwater depths and elevations (NGVD) that could have been potentially considered during the Twinbrook open channel enclosure design, and the indicates the following:

- The 10-foot x 5-foot CBC would pass the 10-year 1031 cfs peak discharge for full flow through the first 45 feet culvert segment (Slope=0.99%) with a headwater depth of 9 feet equivalent to an elevation of 290.6 feet without overtopping the culvert embankment at 292.0 feet.
- The 10-foot x 5-foot CBC would pass the 10-year 1031 cfs peak discharge for inlet control (hydraulic opening = 50 sf) with a headwater depth of 19.25 feet equivalent to an elevation of 300.8 feet clearly overtopping the culvert embankment at 292.0 feet.
- An alternate 14-foot x 7.5-foot CBC (similar to the culvert size below Twinbrook Parkway) would pass the 10-year 1031 cfs peak discharge for inlet control

(hydraulic opening of 105 sf) with a headwater depth of 9.75 feet equivalent to an elevation of 291.4 feet without overtopping the culvert embankment at 292.0 feet.

5.5.6 Rating Assessment Findings

Using the limited culvert data available and applying culvert rating methods available in the 1960s, it can be surmised (not concluded) that the designer of the Twinbrook open channel enclosure selected the 10-foot x 5-foot CBC based on the ability of the culvert to pass the design peak discharge at full (or partial) full flow through the first enclosure segment. This would be premised on the ability of the remaining culvert segments (at steeper slopes) to convey the inflow discharge to the outfall through the 14-foot x 7.5-foot CBC at Twinbrook Parkway and through the eastern conveyance enclosure (Also 10-foot x 5-foot CBC).

The inflow control analysis indicates that although the 10-foot x 5-foot CBC could pass most storm events at full (or partial full) flow without overtopping the culvert embankment, it would not do so for storm events that produced an inlet control condition (such as storm events with increased rainfall intensity) where the hydraulic opening is not able to keep pace with the incoming peak. The inlet control analysis for the 10-foot x 5-foot CBC with a potential headwater of 19.25 feet (Elevation 300.8 feet) clearly shows exceedance of the maximum available head of 10.4 feet (292-281.6).

The alternative culvert inlet control condition analysis using a larger culvert size indicates that if the designer had selected a similar culvert size such as the one located at the Twinbrook Parkway (14-foot x 7.5-foot), a calculated inlet control headwater depth of 9.75 feet (Elevation 291.4 feet) would not overtop the embankment at 292.0 feet.

5.5.7 Original Culvert Design Assessment Using Current Rating Tools

The HY8 Federal Highway Administration (FHWA) culvert rating tool was not available in the 1960s. However, this study has also applied the HY8 culvert rating tool to reproduce and verify the previous Nomograph rating approach results for the western segment of the enclosure The HY8 rating tool was applied for the broken-back culvert design segments with culvert slopes of 0.99%, 3.58% and 3.72%. The following figures display the HY8 analysis results.

Culvert Crossing: Tributary 2 Culvert

Water Surface Profiles

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	Flow Type	Length Full (ft)	Length Free (ft)
10.00	10.00	282.07	0.52	0.0*	1-S2n	0.00	394.00
114.00	114.00	283.96	2.65	0.0*	1-	0.00	394.00
218.00	218.00	285.33	4.08	0.0*	1-	0.00	394.00
322.00	322.00	286.57	5.36	0.0*	5-	0.00	394.00
426.00	426.00	287.82	6.73	0.0*	5-	0.00	394.00
530.00	530.00	289.20	8.35	0.15	5-	0.00	394.00
634.00	634.00	290.78	10.30	1.56	5-	0.00	394.00
738.00	723.44	292.34	11.25	2.24	5-	0.00	394.00
842.00	761.07	293.04	11.88	2.68	5-	0.00	394.00
1031.00	810.75	294.04	12.43	6.49	5-	0.00	394.00
1050.00	815.07	294.12	12.90	3.41	5-	0.00	394.00

Figure 93 - HY8 Culvert Water Surface Profiles Table

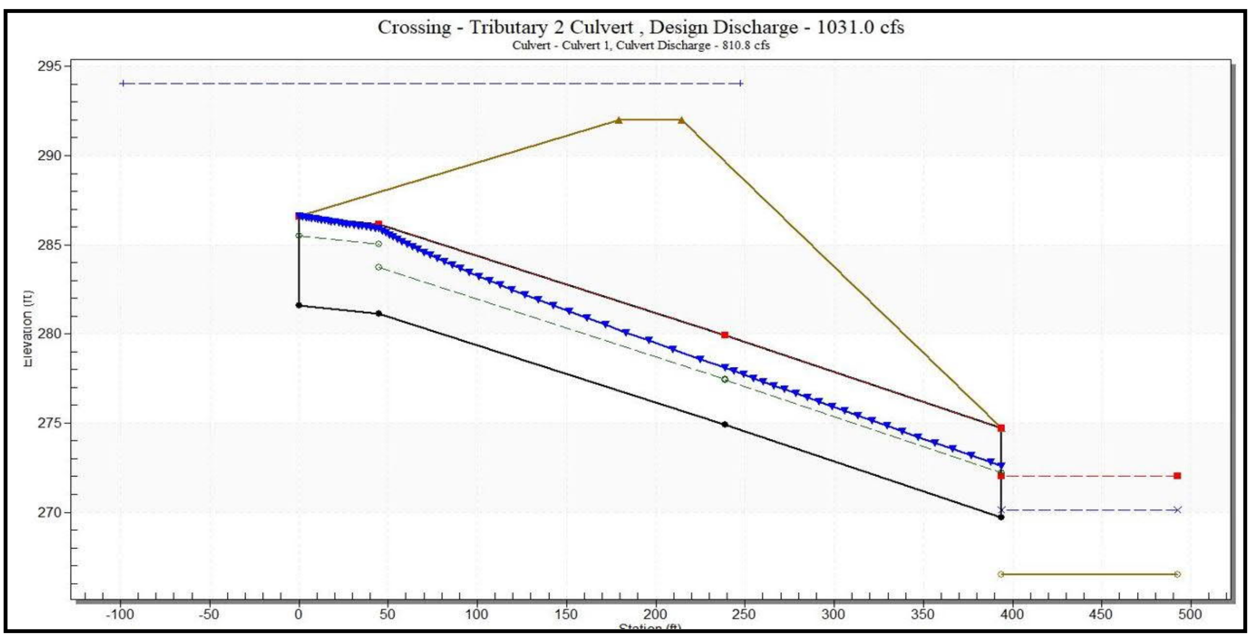

Figure 94 - HY8 Culvert Performance Rating

Figure 93 indicates that using today’s rating tools a headwater of 294.04 feet is calculated for the 1963 broken back western segment culvert design. This water surface stage is 2 feet above the maximum allowed stage of 292 feet.

5.6 Contemporary Methodology for Long Culvert Hydraulic Analysis

The FHWA HY8 culvert rating tool is one of many software programs used to perform typical highway culvert crossings ratings that do not include complex hydraulic features such as sudden drops in invert elevations and horizontal alignment changes.

It was previously stated that the vertical and horizontal alignment features of the culvert contribute to numerous hydraulic losses associated with flow regime and the headwater. Additionally, Figure 95 shows that a velocity dissipation pad and checks (weirs) were incorporated to the downstream (eastern segment) culvert conveyance in the 1966 design. This could also potentially result in additional constraint to flow discharge.

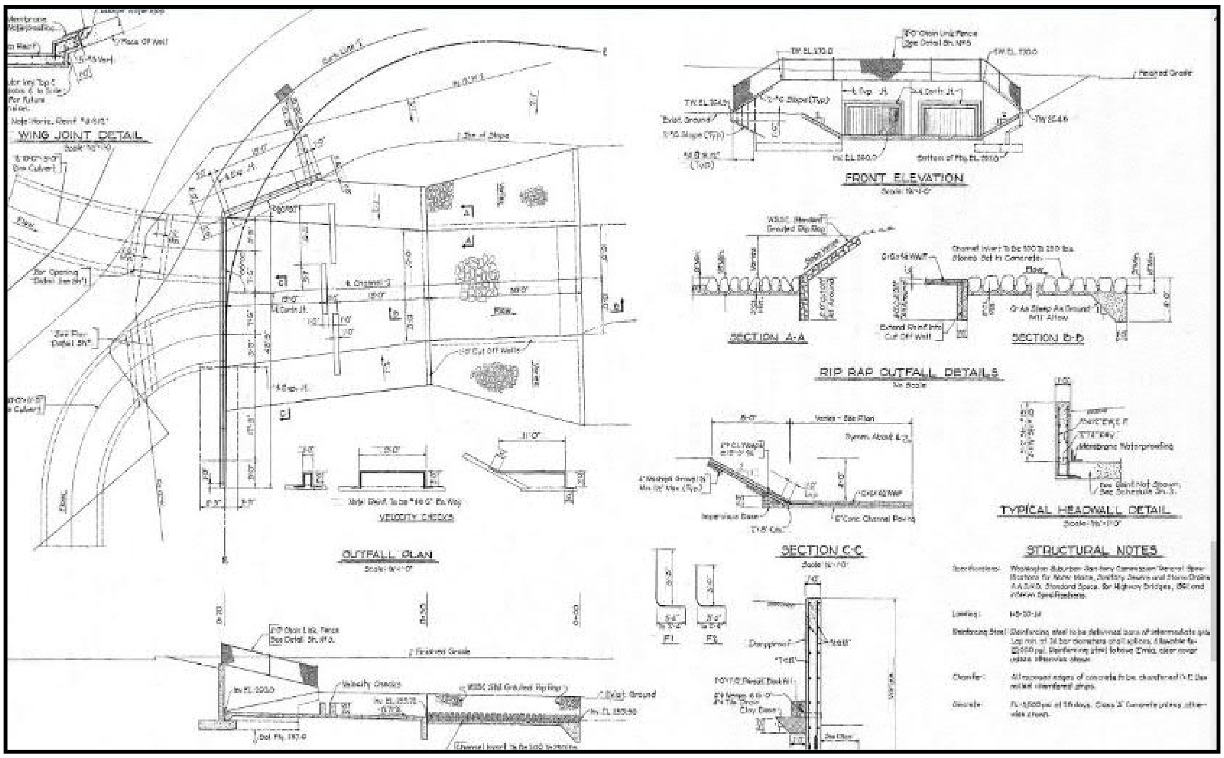

Figure 95 - Outlet Velocity Control Design Features

Although a broken back design can be performed for culverts of limited length, it is at best an approximation of real hydraulic behavior. This is especially true for extra-long culverts.

The various culvert horizontal alignment changes and abrupt vertical invert drops generate significant head losses throughout the long culvert enclosure alignment that are very difficult to analyze with conventional culvert inlet/outlet control condition software. Long culverts have been analyzed using the Corps of Engineers HEC-RAS culvert design software, private domain software including StormCAD, Culvert Master, etc. with various degrees of success. A recent long culvert rating methodology comparative study performed by Shrinivas Kaulgud, P.E., CFM[17] provides insight for the culvert overtopping analysis of Tributary 2.

The Kaulgud comparison indicates that steady state HEC-RAS with Lid option for pressurized pipes underestimated the floodplain and water surface elevations for long culverts and storm drainpipes. Alternative long culvert/pipe capacity analysis considered for comparison included CulvertMaster+HEC-RAS or StormCAD+HEC-RAS combinations that produce moderate levels of accuracy. If high accuracy is desired the EPA SWMM/ XP-SWMM or similar hydrodynamic programs with robust hydrologic routing methods and with the ability to simulate complex hydraulic schemes should be considered. These include models that have the capacity to analyze detailed overland and storm drain networks, are run in unsteady mode and can seamlessly capture overland flow and underground pipe flow in 1D, 2D, a hybrid or combination of 1D/2D modes.

The Interconnected Pound Routing Model Version 4 (ICPR4) falls in this category, and thus Mercado Consultants and WRMA utilized ICPR4 to analyze the Twinbrook Tributary 2 culvert overtopping event on September 1, 2021. Further details on the methodology applied with ICPR4 is provided in the following section.

[17] *A Comparison of Different Methods*, Shrinivas Kaulgud, P.E., CFM, Cheryl Hannan, P.E., CFM, LEED AP, October 12, 2017

6.0 Hydrologic and Hydraulic Study

6.1 Rainfall Data Analysis

Post flood event hydrology procedures require the reconstruction of the event to determine what really happened, what factors contributed and what failed. This is especially important in the context of climate change, which will increase the magnitude of an event and the frequency of extremes. The analysis consists of the application of an established water resources engineering methodology immediately after the occurrence of the event using the best available hydrometeorological records in the vicinity of the project flood event site. The analysis can then move forward in performing an evaluation or study of the event.

The Hydrometeorological data acquired includes records of rainfall precipitation from nearby climatologic stations such as airports, fire stations, public and personal accredited networks, etc. Preference is given to automated stations and meteorological observatories with long term records that record rainfall precipitation at the lowest intervals (typically 5 minutes). Such information will provide rainfall event information regarding magnitude, duration, and intensity. The hydrometeorological and hydrological analyses include the following.

6.1.1 Storm Genesis

The understanding of the storm event requires an analysis of the storm's beginning. The natural phenomena must be clearly identified and characterized in its relationship to the flood event at the project site. It is necessary to determine if the single storm or a combination of events caused the culvert overtopping.

Meteorology defines four different conditions that produce flash flooding rains (synoptic, frontal, mesohigh, and western). This analysis considers each of these conditions to categorize the heavy rainfall event of September 1, 2021.

The storm event that caused the culvert overtopping failure at Twinbrook was related to Hurricane Ida. Hurricane Ida was a deadly and destructive Category 4 Atlantic hurricane that became the second-most damaging and intense hurricane to make landfall in the U.S. state of Louisiana on record (behind Hurricane Katrina in 2005).

Hurricane Ida originated from a tropical wave in the Caribbean Sea on August 23, 2021. The tropical wave developed into a tropical depression on August 26, 2021, which then became Tropical Storm Ida later that day, near Grand Cayman. Ida intensified into a hurricane on August 27, 2021, just before moving over western Cuba. The hurricane underwent rapid intensification over the Gulf of Mexico and reached its peak intensity as a strong Category 4 Hurricane while approaching the northern Gulf Coast. Ida had maximum sustained winds of 150 mph (240 km/h) and a minimum central pressure of 929 millibars (27.4 inHg). Hurricane Ida made landfall near Port Fourchon, Louisiana on August 29, 2021.

Hurricane Ida weakened steadily over land and was downgraded to a tropical depression on August 30, 2021. As the system moved through the Northeastern United States it transitioned into a post-tropical cyclone and combined with a frontal zone to unleash unprecedented rainfall across the region. The remnants of the storm produced unexpectedly severe damage in the Northeastern United States on September 1 and 2, 2021.

6.1.2 Spatio-Temporal Distribution

Figure 96 is a Radar Loop scene of the storm captured from a video by the City of Rockville engineering staff. The staff used the RadarScope App to follow the storm event in near real time.

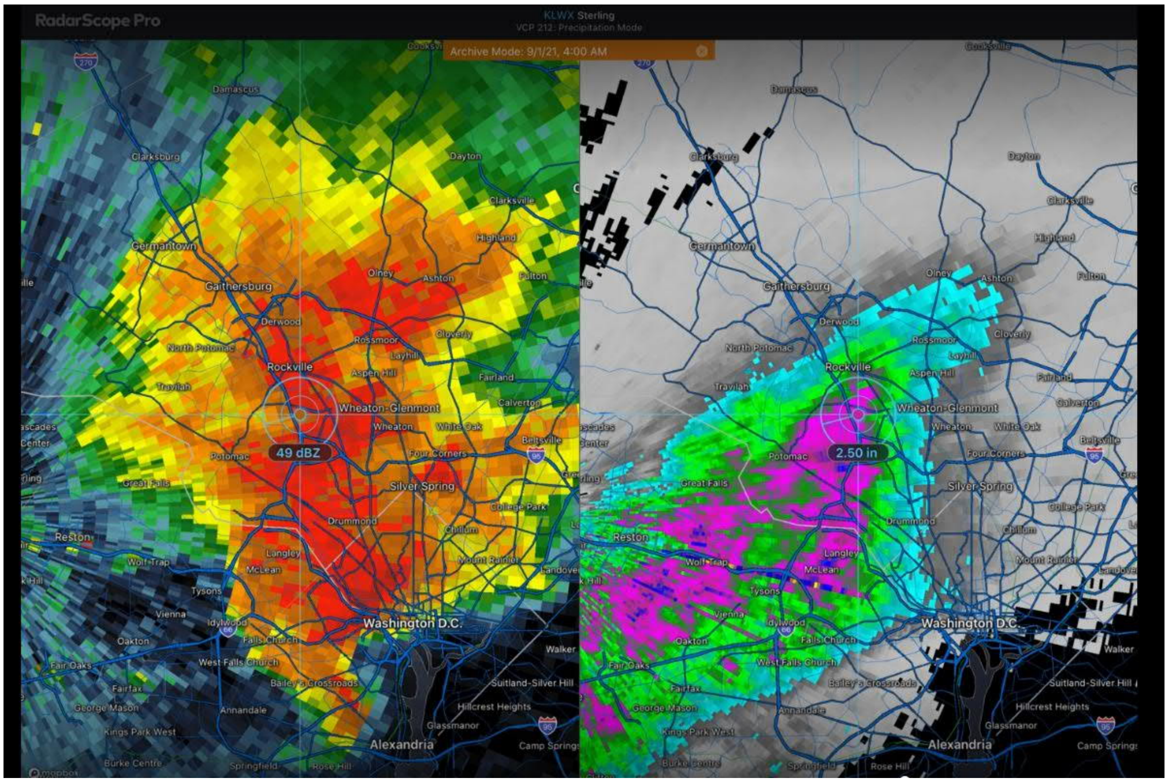

Figure 96 - NWS Sterling Radar Loop Scene.

RadarScope can access the Next Generation Weather Radar (NEXRAD) weather data. The NEXRAD system is a network of 160 high-resolution S-band Doppler weather radars jointly operated by the National Weather Service (NWS), the Federal Aviation Administration (FAA), and the U.S. Air Force. The NEXRAD system detects precipitation and wind. Processed data can map precipitation patterns and movement. NWS provides access to archived NEXRAD Level-II data and Level-III products.

The App was able to tap into the nearest NEXRAD NWS station at Sterling, VA and capture a video that shows the typical reflectivity and provides an estimate of the rainfall precipitation. The advantage of Doppler radar is to detect where the precipitation is falling and determine the movement of particles within the storm.

Figure 96 indicates that the front was moving from SW to NE and between the hours of 2:00 am to 4:00 am, a frontal cell with intense (red) rainfall precipitation was directly above the City of Rockville and the Twinbrook area in particular. The RadarScope App indicated that approximately 2.4 inches of rainfall fell within 34 minutes at or near the Twinbrook site, which corresponds to a 200-to-250-year frequency of occurrence (average recurrence interval per NOAA 14 Atlas).

The mixed subtropical depression/frontal zone storm event falls within the “synoptic” category. These events are the result of an intense synoptic scale cyclone or frontal system. Synoptic events normally develop in association with a quasi-stationary or slow-moving front, usually oriented from southwest to northeast, moving through at approximately 500mb and with heavy rains occurring in the warm sector ahead of the front.

6.1.3 Precipitation Data Sources

Hydrometeorological data acquired for rainfall/runoff analysis include records of rainfall precipitation from nearby climatologic stations such as airports, fire stations, recreation centers, and personal accredited networks. Preference is given to automated stations and meteorological observatories with long term, time series records that document rainfall precipitation at daily, hourly, and sub-hourly intervals. This data statistically defines the storm events using historical records. Likewise, NWS NEXRAD Doppler data is necessary to provide local information of the rainfall event regarding its magnitude, duration, and intensity. Less used but equally as important is the availability of personally acquired weather station data such as available from the Weather Underground network.

There are many ways to estimate precipitation intensity and accumulation and at ungauged sites such as the Tributary 2 culvert overtopping site. These include radar-based point observations, and automated weather station sites with digital recording and telemetry. There are also many ways to spatially distribute or average the precipitation measurements from various gage sources including the Thiessen and Isohyetal Methods. This analysis will compare the precipitation from the nearest rain gage, with other regional gaging sources obtained from established public sources within a 25-mile radius, and from local (1-3 miles) gaging sources.

This study investigated the available public and private weather stations near the project site to perform the post-storm event hydrologic analysis. A rainfall gage operating on behalf of the City of Rockville was located within 0.2 miles of the project site. The City of Rockville installed the rain gauge, located on the roof of the Twinbrook Community Recreation Center, in June 2021. The gage records and sends information that is used to calibrate the City’s sanitary sewer model. The rain gauge equipment is composed of a Trimble Transponder and a Texas Electronics rain gauge. Figure 97 depicts the rainfall gage.

Figure 97 - Twinbrook Recreational Center Rain Gage.

The Texas Electronics, Inc. TR-525I Rainfall Sensor with 6" diameter funnel is a remote tipping bucket style rain gauge that measures liquid precipitation. The Rain Gauge is a freestanding receptacle for measuring precipitation. Rain is collected and then funneled to a mechanical device through an opening at the top of the device to a tipping bucket (the bucket tips one time for each 0.01” of rain that falls). As the tipping bucket collects water, it fills to the point that it tips over causing a momentary closure of a switch to incrementally measure rainfall accumulation. This measurement action empties the bucket in preparation for additional measurement. Water discharged by the tipping bucket passes out of the rain gauge with no need for emptying. The approximate combined cost of the Trimble Transponder and Texas Electronics rain gauge hardware/software is over $2,000.

The Trimble Telog 32 series recorder has a wireless data transfer capability. Using cellular technology enables unmanned monitoring of remote sites as well as instant updates and alarm notifications. The Telog RG-32A uses a low power, LTE/Cat 1 cellular communication modem certified on multiple cellular systems. This ensures maximum coverage, reliability of service, and alignment with cellular carrier’s technology roadmaps. The Telog RG-32A can be configured to call its server application on a schedule (e.g., once per day; every four hours, etc.) and/or on alarm (e.g., in response to a major rainfall event). Data may be stored in the recorder at user defined intervals (e.g., five minutes, one minute, etc.). TR-525I has a measurement accuracy of +/- 1% for a 0-2 in/hr. rainfall accumulation range**.** Figure 98 shows the Trimble Telog 32 series recorder.

Figure 98 - Trimble Telog 32 Series Recorder.

The location of the gage on the roof of the Recreation center is ideal as there are no trees or branches blocking the rainfall. The City of Rockville employs a consultant for regular maintenance and operation of the equipment.

As previously indicated, the City of Rockville obtained a radar Loop of the storm event (via the RadarScope App) that fell on the project site on the night of September 1, 2021. The App updates the total rainfall with each "sweep" of the radar, which is typically every 4-6 minutes. The radar estimated cumulative rainfall in inches is directly reported with the timestamped scan. The City of Rockville staff acquired the Radar loop data to get a sense of what sort of storm event occurred and the estimated rainfall intensity that fell during the storm.

Although the City staff-acquired Radar Loop was intended as a broad, high-level look at the storm, this study will use it in the analysis as a comparative tool to the rainfall intensity and accumulated rainfall volume obtained by the Twinbrook Community Recreation Center (TCRC) Station. Figure 99 shows the rainfall volume accumulation for the Twinbrook Community Recreation Center Station and the Radar Loop estimates from 2:24 am to 3:50 am on 09/01/2021.

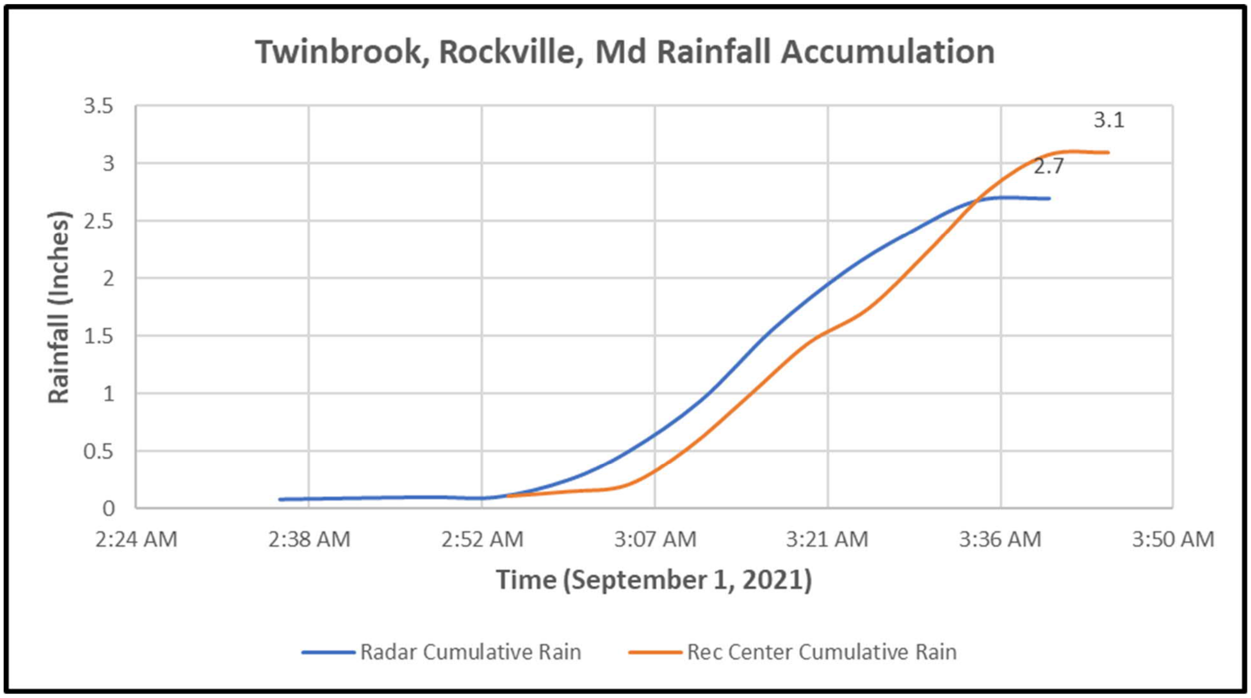

Figure 99 - Twinbrook Community Recreation Center Station and the Radar Loop Accumulated Rainfall Volumes.

The Radar rainfall is updated with each sweep of the radar (typically every 4-6 minutes) and reported near instantly while the TCRC Tipping Bucket station is a mechanical device and takes longer to be recorder by the data logger for reporting (typically at 3-5 minutes). This effect explains the lagging nature of the TCRC rainfall volume accumulation depicted in Figure 99.

The 0.4-inch difference in total volume accumulation at the flattening of the curve (around 3:40 am to 4:00 am) can be explained by the volume accumulation accounting technique. The Radar loop digitally calculates the accumulation measurement based on the 4-6 minutes sweep. The tipping bucket is a mechanical analog teeter-totter type device and the bucket tips one time for each 0.01” of rain that falls. The data logger reports at 5-minute increments or higher and therefore total accumulated volume reported tends to be slightly larger.

6.1.4 Regional Rainfall Weather Station Sources

Initially, as part of the hydrometeorological data search, this study requested the official real time NEXRAD rainfall precipitation records for the project site from the NWS. However, the Covid-19 pandemic restrictions has significantly impacted the processing of NWS records requests and was not received by the study publish date.

In the absence of NWS records, this study utilized regional public source weather stations and the local Weather Underground Network to compare and collect data from the surrounding area.

Figure 100 shows the location of five established regional public source weather stations located within twenty-five miles of the project site. This study obtained daily total precipitation recorded from these sources for the period before and after the storm event for comparative analysis.

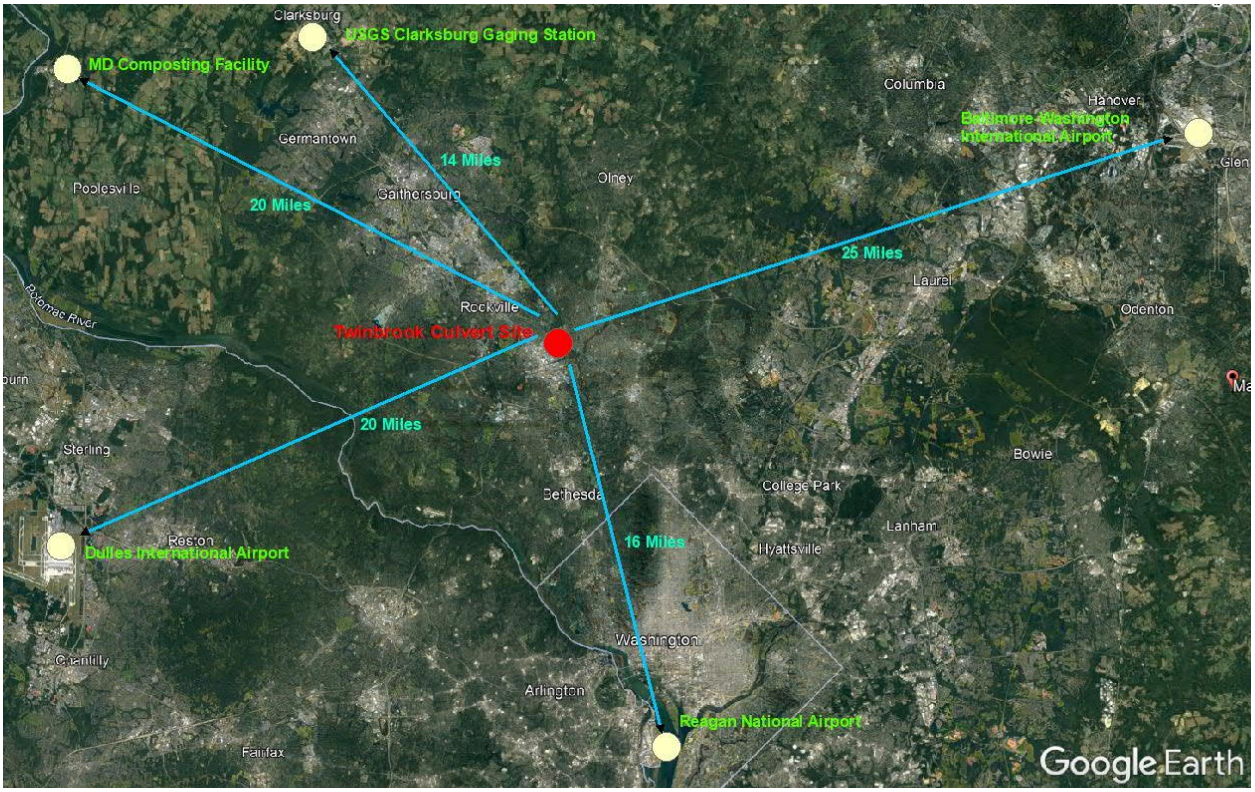

Figure 100 - Regional Public Source Weather Gaging Stations Relative to the Project Site.

1. **USGS 391407077174001 Tenmile Creek Precipitation Gage at Clarksburg, Maryland (14 Miles Northwest of Project Site)**

Location: Latitude 39°13'24.2" N, longitude -77°18'44.1" W, Elevation 426.05 feet above NAVD88. Located in Montgomery County, MD, on left bank 1.3 mi upstream from Little Seneca Lake, and 2 miles north of Boyds, MD. This gage is located 14 miles from the project site.

Period Of Record: June 2013 to current year (9 years)

Figure 101 and Figure 102 show that daily total precipitation for September 1, 2021, was 1.79 (provisional). Little rainfall was recorded at the Clarksburg Gage for the August 20 to 31, 2021 period.

Daily Sum Precipitation, total, inches		
DATE	Aug 2021	Sep 2021
1		1.79^P
2		0.00^P
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		
20		
21		
22		
23		
24		
25	0.19^P	
26	0.00^P	
27	0.03^P	
28	0.20^P	
29	0.00^P	
30	0.16^P	
31	0.01^P	
COUNT	7	2
MAX	0.20	1.79
MIN	0.00	0.00

Figure 101 - USGS Weather Station Tenmile Creek @ Clarksburg – Daily Precipitation Totals.

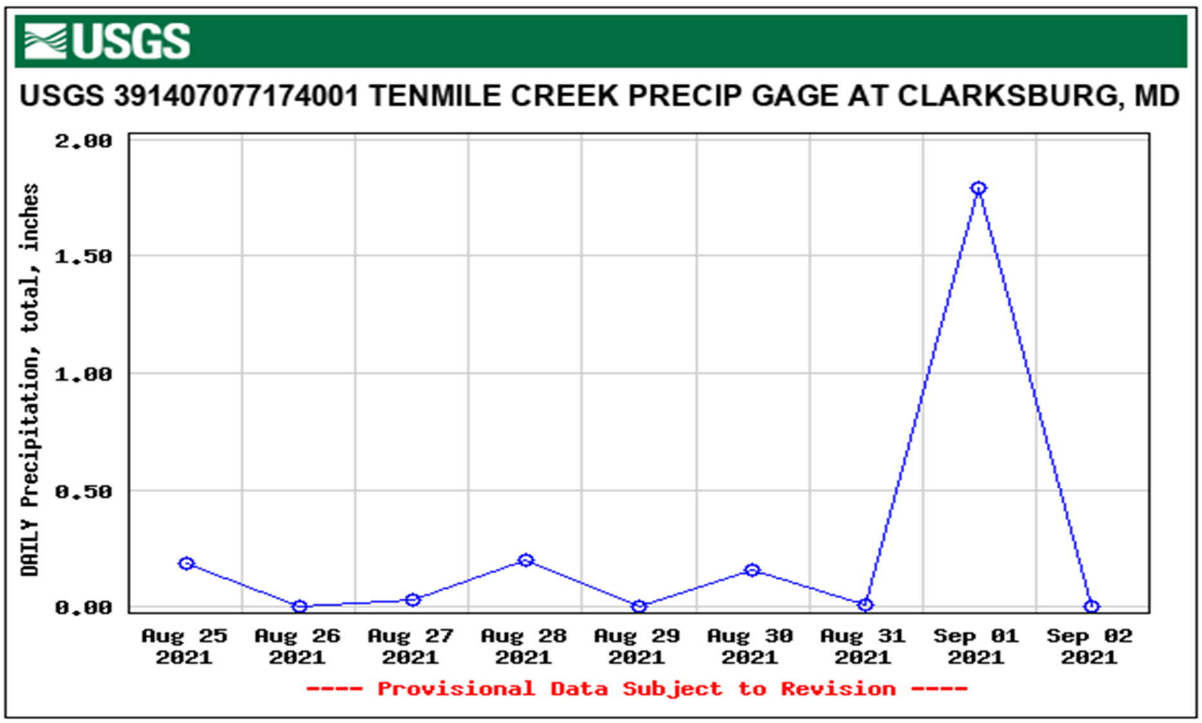

Figure 102 - USGS Weather Station Tenmile Creek @ Clarksburg – Plot of Daily Precipitation Totals.

2. **Department of Environmental Protection Dickerson Weather Station at Montgomery County, MD Composting Facility (20 Miles Northwest of Project Site)**

*Location***:** Latitude 39°11'52.4" N, longitude -77°27'04.03" W, Elevation 390 Feet above NAVD88. The weather station is located adjacent to the Montgomery County Composting Facility trailer.

Equipment: The station consists of a 30-foot (Standard Height) aluminum tower and a trailer. This equipment is on the top of the tower to collect weather data: An anemometer to collect wind speed, A wind vane to collect wind direction, A thermistor to collect ambient temperature, A Dew Cell to record dew point temperature/relative humidity. A tipping bucket rain gauge is on the ground near the trailer for collecting rainfall. The gage collects precipitation at 15-minute intervals.

Period Of Record: Since 2002 (20 years)

Figure 103 indicates that little rainfall occurs in the vicinity of the Dickerson gage for the previous five days of the overtopping event on 09/01/2020 (Period from 08/26/2020 to 09/01/2020). Figure 104 shows that the Dickerson gage registers a maximum rainfall intensity of 0.63 inches per hour on 09/01/2021 between 4:00 am and 5:00 am. Figure 105 indicates that the Dickerson gage documented a maximum daily rainfall volume of 2.28 inches on 09/01/2021.

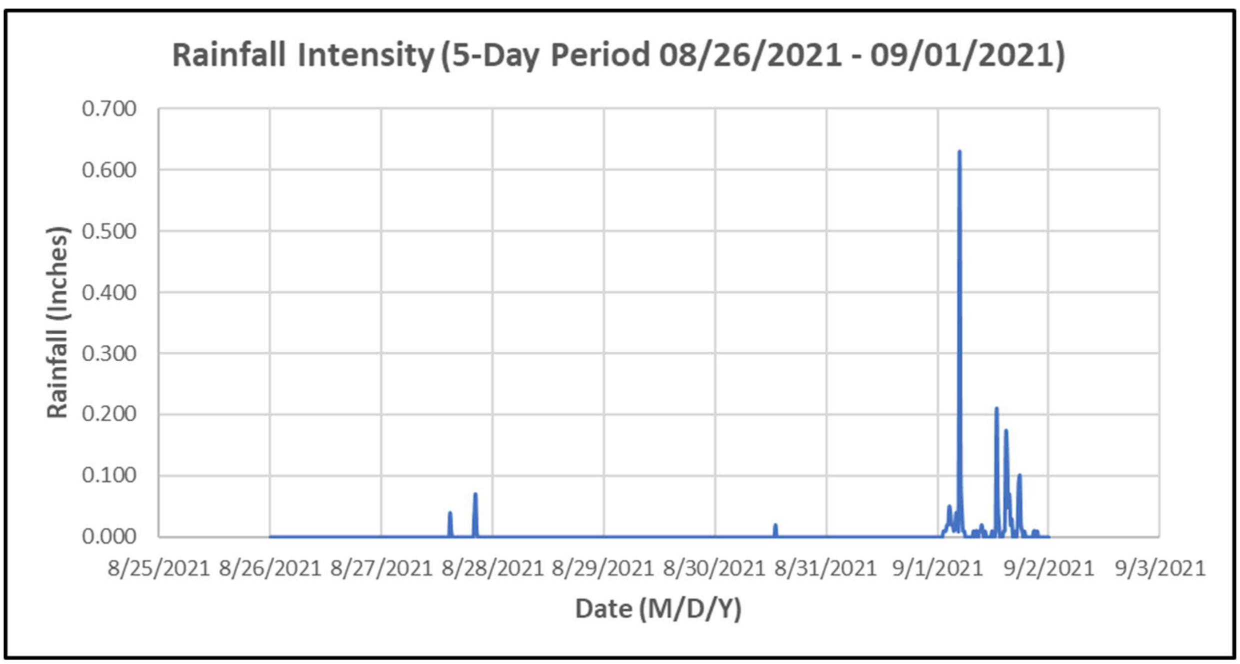

Figure 103 - Five Day Rainfall Intensity (08/26/2021 to 09/01/2021)

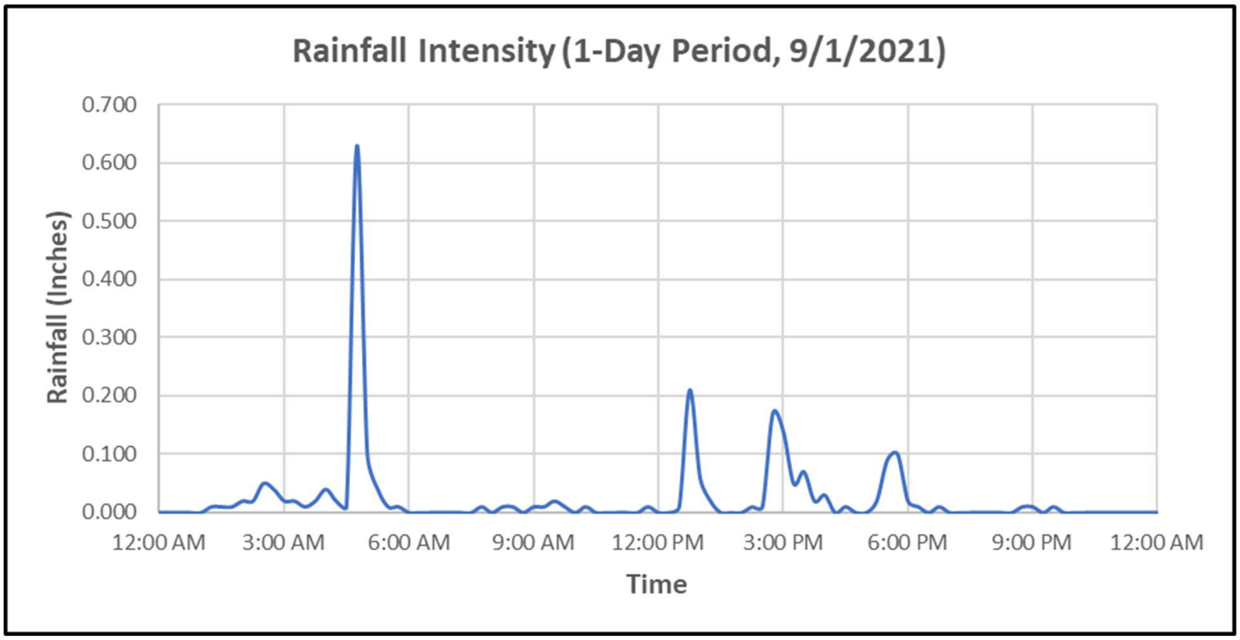

Figure 104 - One Day Rainfall Intensity (12:00 am 09/01/2021 to 12:00 am 09/02/2021).

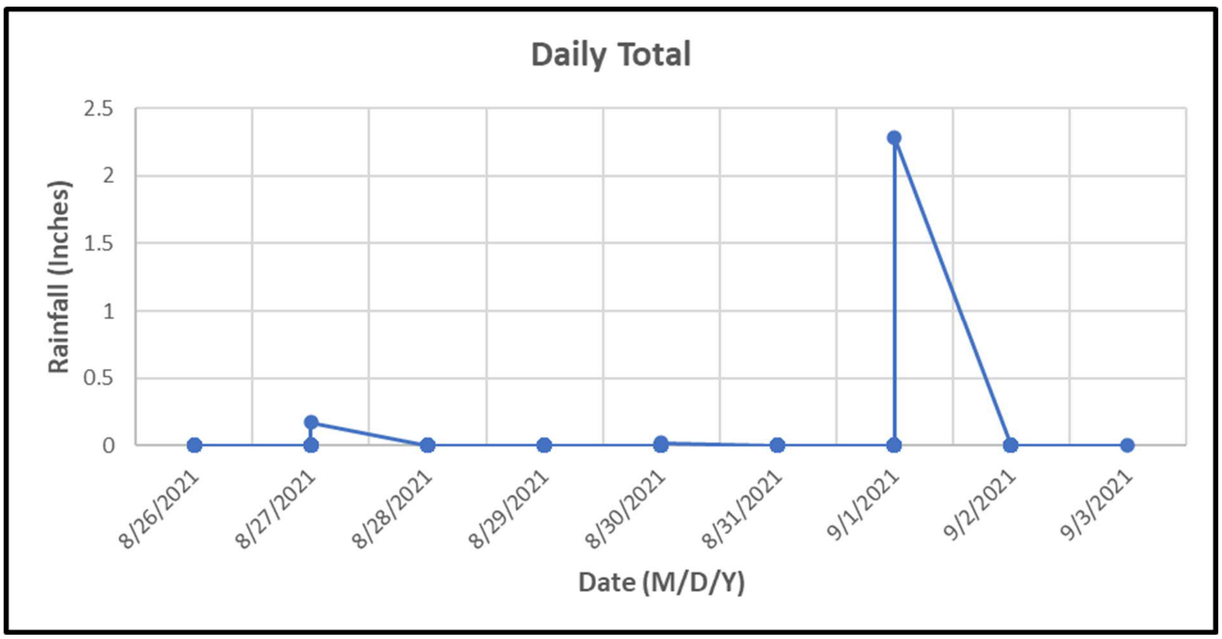

Figure 105 - Dickerson One Day Cumulative Rainfall Total.

3. **Washington Dulles International Airport Weather Station (20 Miles West of Project Site)**

Location: Latitude 38.9349° N, Longitude -77.4473° W, Dulles, Virginia, Elevation 290.0 feet above NAVD88.

4. **Washington Reagan National Airport Weather Station (16 Miles South of Project Site)**

Location: Latitude 38.8472° N, -77.0345° W, Arlington, Virginia, Elevation 10 feet above NAVD88.

Period Of Record: Since 01/09/1936 (86 years)

5. **Baltimore Washington International Airport Weather Station (25 Miles Northeast of Project Site)**

Location: Latitude 39.1733° N, -76.684° W, Maryland, Elevation 155.8 feet above NAVD88.

Period Of Record: Since 01/07/1939 (81years)

Figure 106 shows the daily airport rainfall volumes for 08/20/2021 to 09/05/2021 period. Table 2 includes the total daily rainfall volumes for all regional weather stations.

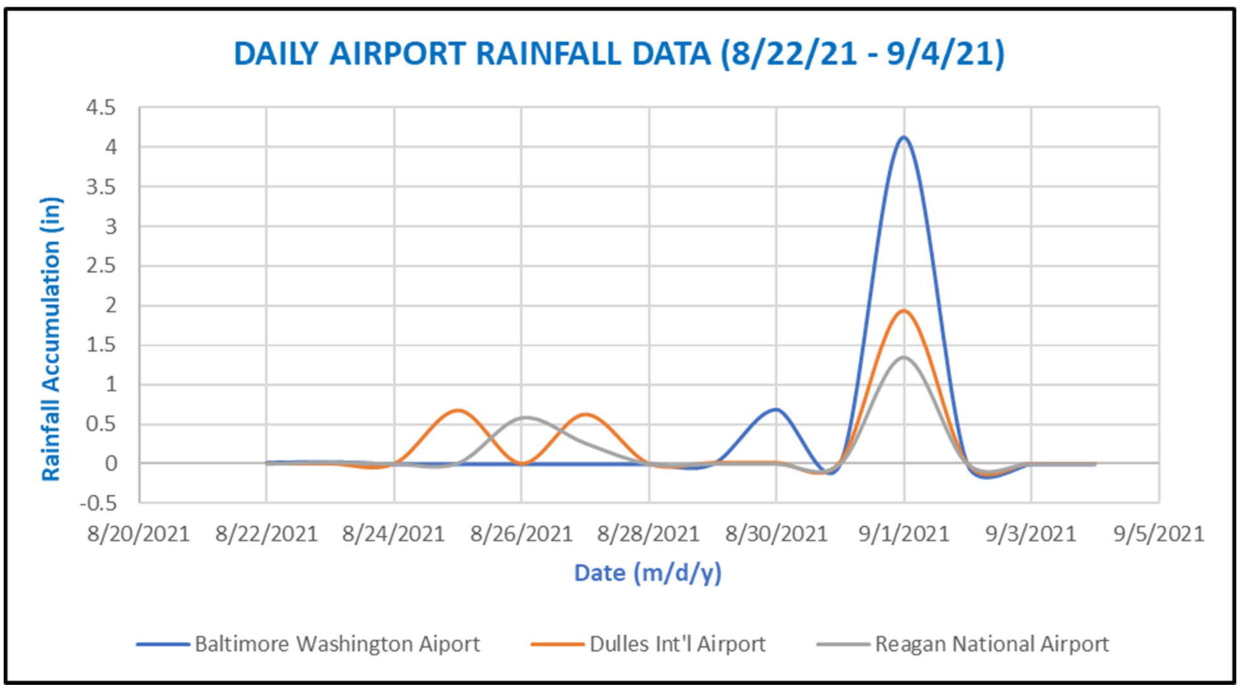

Figure 106 - Daily Airport Rainfall Volumes for 08/20/2021 to 09/05/2021.

Table 2 - Total Rainfall Volumes for Regional Weather Station

Weather Rainfall Station ID	Regional Weather Rainfall Station Name	Total Daily Rainfall Volume for 09/01/2021 (Inches)
1	USGS 391407077174001 Tenmile Creek Precipitation Gage at Clarksburg, Maryland	1.79
2	Department of Environmental Protection, Dickerson Weather Station at Montgomery County, MD Composting Facility	2.28
3	Washington Dulles International Airport Weather Station	1.93
4	Washington Reagan National Airport Weather Station	1.34
5	Baltimore Washington International Airport Weather Station	4.13

Figure 101**,** Figure 105**,** Figure 106 and Table 2 indicate that total daily rainfall volumes varying from 1.34 to 4.13 inches were experienced on 09/01/2021 by regional gaging sites within 25 miles from the culvert failure site. These volumes align with the direction (SW to NE) and intensity of the frontal storm. The Baltimore Washington International and Washington Reagan National Airport gages recorded the maximum (4.13 inches), and minimum (1.34 inches) one day volumes, respectively.

All gages recorded minimum rainfall for the five days prior to the storm event. This is significant as the prevalent Antecedent Moisture Condition (AMC) is an indicator of the catchment regarding soil moisture storage before the storm and has an important impact on the volume of runoff.

The apparent AMC 2 condition means that a significant storm with high rainfall intensity would have quickly used any available soil storage and become excess runoff almost instantly, such as the 0.6 inches per hour recorded at the Dickerson gage (see Figure 103).

6.1.5 Local Rainfall Weather Station Sources

Weather Underground is a global network and community of people collecting data from weather stations at a personal, hyperlocal level. The network includes over 250,000 personal weather stations worldwide. Figure 107 shows some of the many network weather stations in the City of Rockville area.

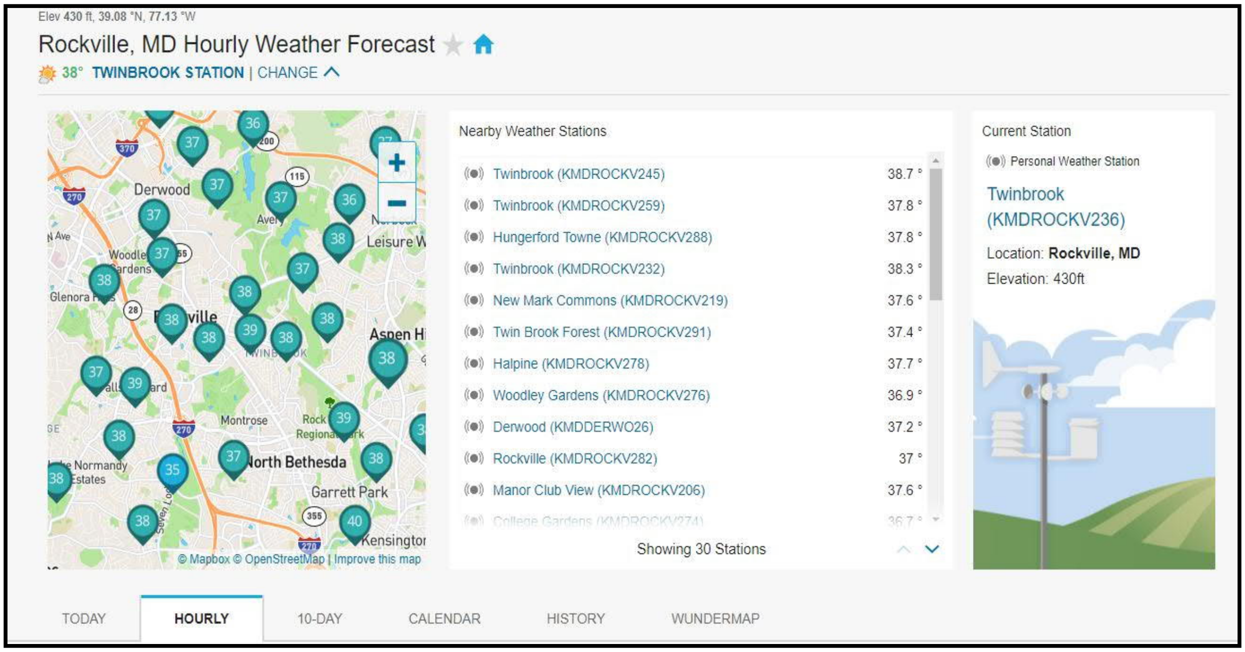

Figure 107 - Weather Underground Stations Near Rockville, Maryland.

Figure 108 shows the location of the project site, the Twinbrook Community Recreation Center gage and four gaging sites of the Weather Underground network selected for analysis. All stations are within 3.0 miles from the project site.

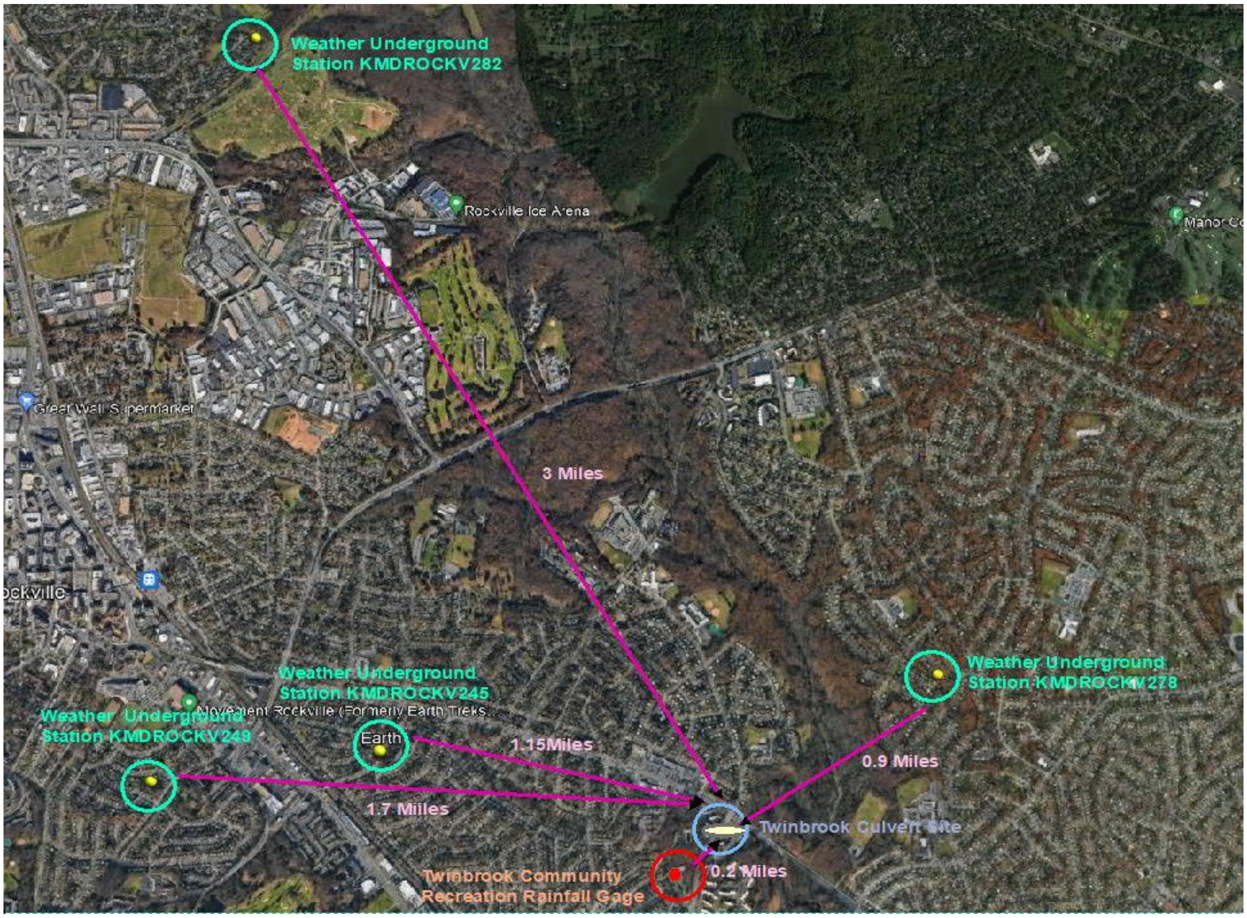

Figure 108 - Twinbrook Community Center and Weather Underground Rainfall Stations.

1. **Weather Station ID: KMDROCKV278 (0.9 Miles Northeast of Project Site)**

Location: City of Rockville, MD, Latitude 39.077° N, / Longitude -77.101° W, Elevation:338 Feet Above NAVD88

Period of Record: Over 5 years

*Hardware***:** Logia 5-in-1, Software**:** weatherlink.com 1.10

Figure 109 shows that station KMDROCKV278, located 0.9 miles from the Tributary 2 culvert, registered a rainfall volume accumulation of 2.34 inches on 09/01/2021 between 2:00 am and 4:00 am. This period is especially important as it includes the onset of the culvert overtopping and flooding of the nearby Rock Creek Woods buildings and parking lot area.

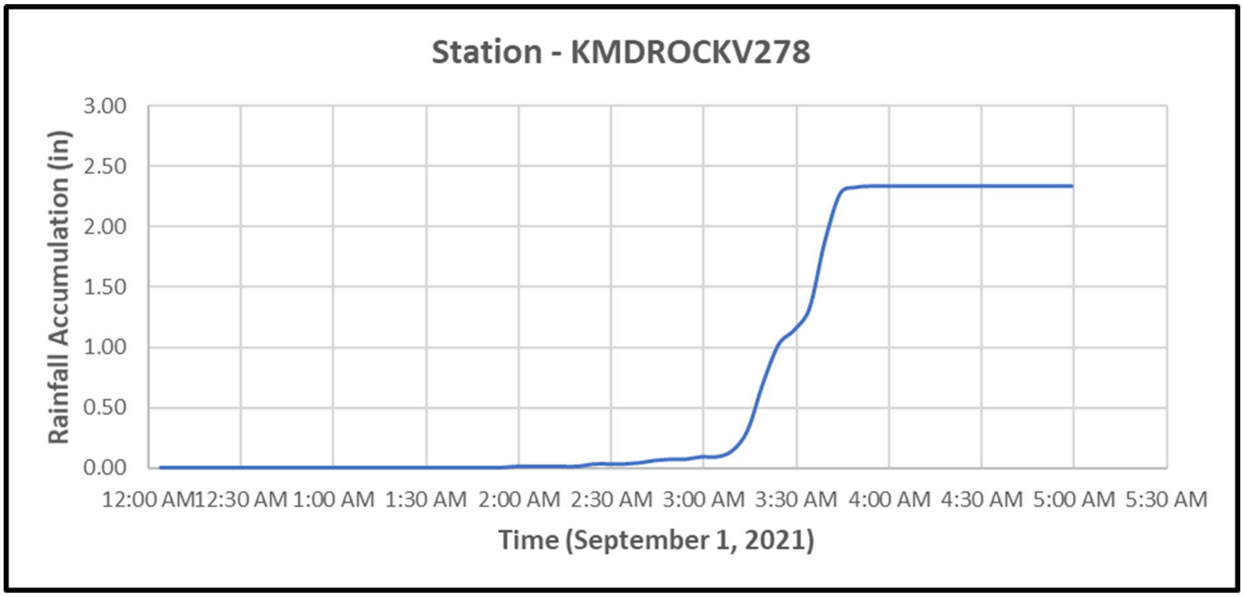

Figure 109 - Station KMDROCKV278 Rainfall Volume Accumulation.

2. Weather Station ID: KMDROCKV282 (3.0 Miles Northwest of Project Site)

Location: City of Rockville, Md, Latitude: 39.112° N, / Longitude -77.141° W, Elevation: 459 Feet Above NAVD88

Period of Record: Over 5 years

Hardware: Davis Vantage Vue (Wireless), Software: weatherlink.com 1.10

Figure 110 depicts rainfall accumulation at station KMDROCKV282. This station is located 3.0 miles away from the culvert overtopping location and registered a rainfall volume accumulation of 2.41 inches between 2:00 am and 4:00 am.

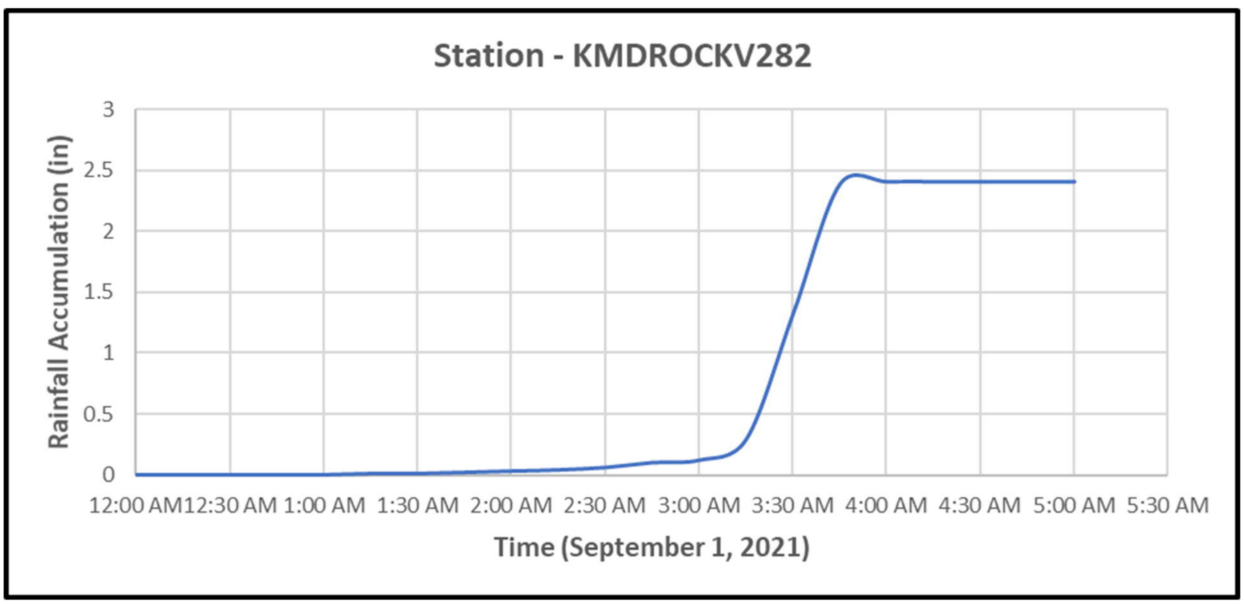

Figure 110 - Station KMDROCKV282 Rainfall Volume Accumulation.

3. **Weather Station ID: KMDROCKV249 (1.7 Miles West of Project Site)**

Location: City of Rockville, Md, Latitude 39.075° N, Longitude -77.145° W, Elevation: 423 Feet Above NAVD

Period of Record: Over 5 years

Hardware: Ambient Weather WS-2902

Figure 111 shows the rainfall accumulation at station KMDROCKV249. This station, situated 1.7 miles from the Tributary 2 culvert, registered a rainfall volume accumulation of 2.34 inches between 02:00 and 04:00 am.

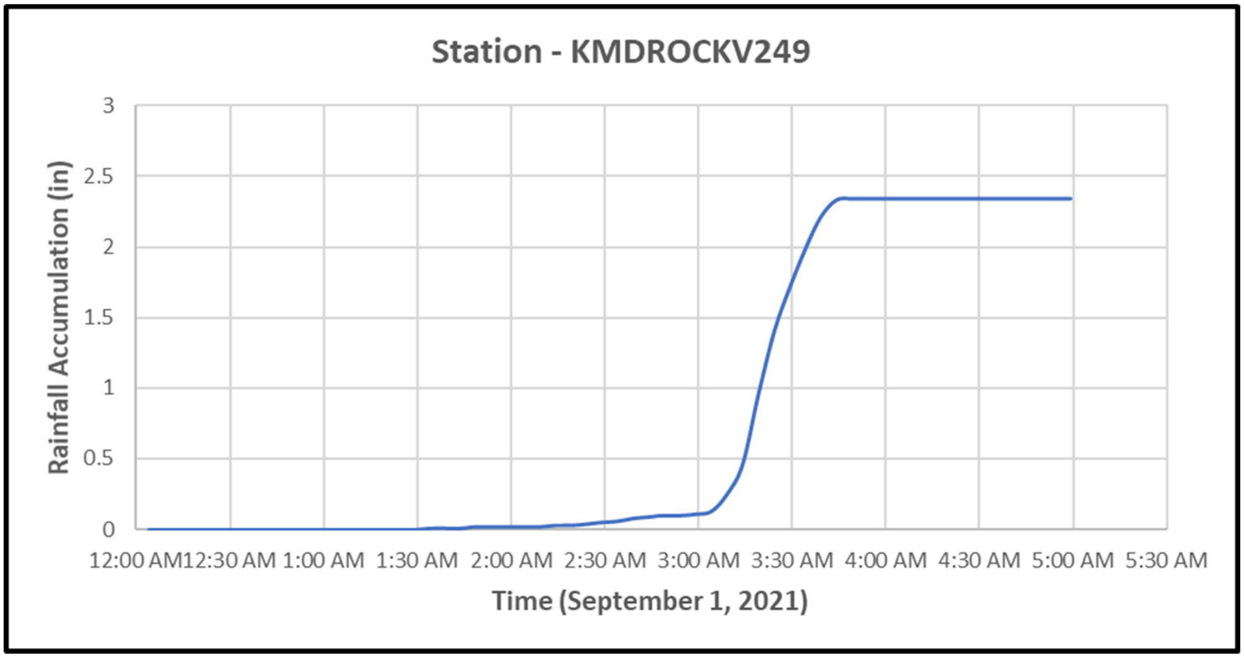

Figure 111 - Station KMDROCKV249 Rainfall Volume Accumulation.

4. **Weather Station ID: KMDROCKV245 (1.15 Miles West of Project Site)**

Location: City of Rockville, Md, Latitude:39.074° N, / Longitude -77.129° W, Elevation:410 Feet Above NAVD

Period of Record: Over 5 years

Hardware: Ambient Weather WS-2902

Software: AMBWeatherV3.0.3

Figure 112 shows rainfall accumulation at station at KMDROCKV245. This station, located 1.15 miles from the culvert overtopping location, documented a rainfall volume accumulation of 3.94 inches between 2:00 am and 4:00 am.

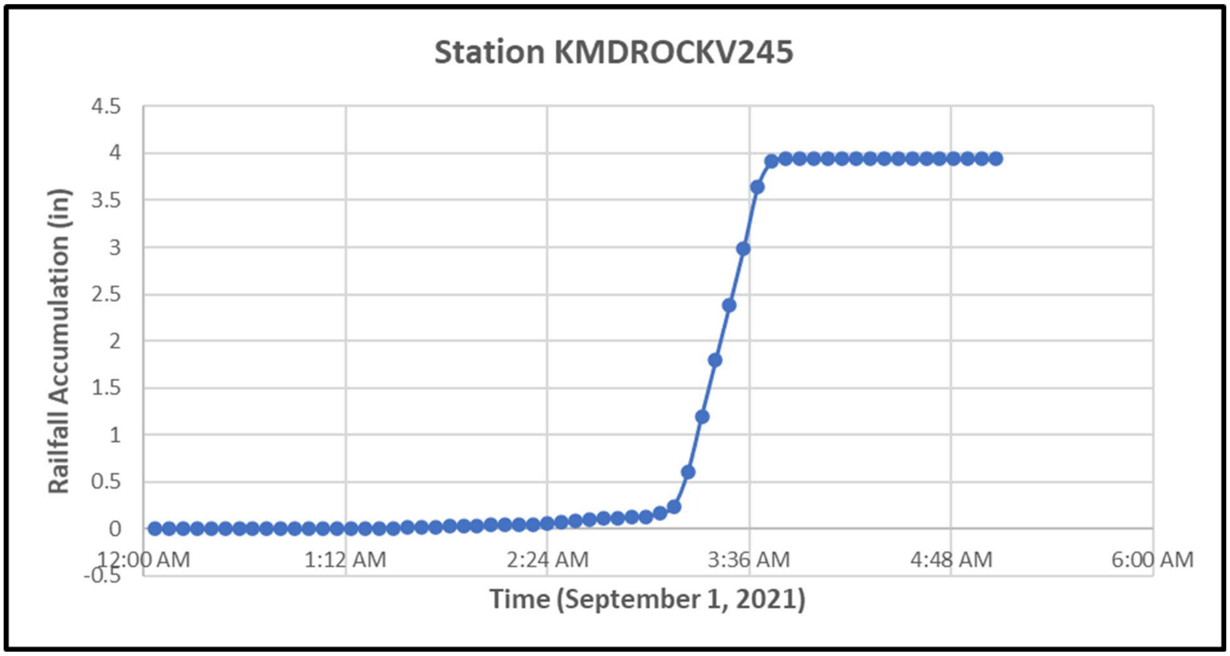

Figure 112 - Station KMDROCKV245 Rainfall Volume Accumulation.

5. Weather Station ID: Twinbrook Community Recreation Center

Location: City of Rockville, Md, Latitude:39.071° N, / Longitude -77.119° W, Elevation:337 Feet Above NAVD

Period of Record: Six months

Hardware: The Texas Electronics, Inc. (TR-525I Rainfall Sensor)

Software: Trimble Telog 32 series recorder with wireless data transfer capability.

Figure 113 is a plot of the rainfall accumulation and rainfall intensity between 2:00 am and 4:00 am on September 1, 2021. Table 3 shows the cumulative rainfall volume data depicted in Figure 113. This data indicates the following key occurrences:

- The gage recorded a total volume of precipitation of 3.10 inches for the 02:00 am to 04:00 am (2-hour) period.
- Most of the rainfall (3.01 inches) was recorded for the 2:45 and 3:45 am (1-hour) period.
- The highest precipitation intensities of 3.87 to 6.36 in/hr. (average 5.12 in/hr.), resulting in 2.56 inches of total accumulation, occurred during the 3:05 to 3:35 am (30-minute) period.

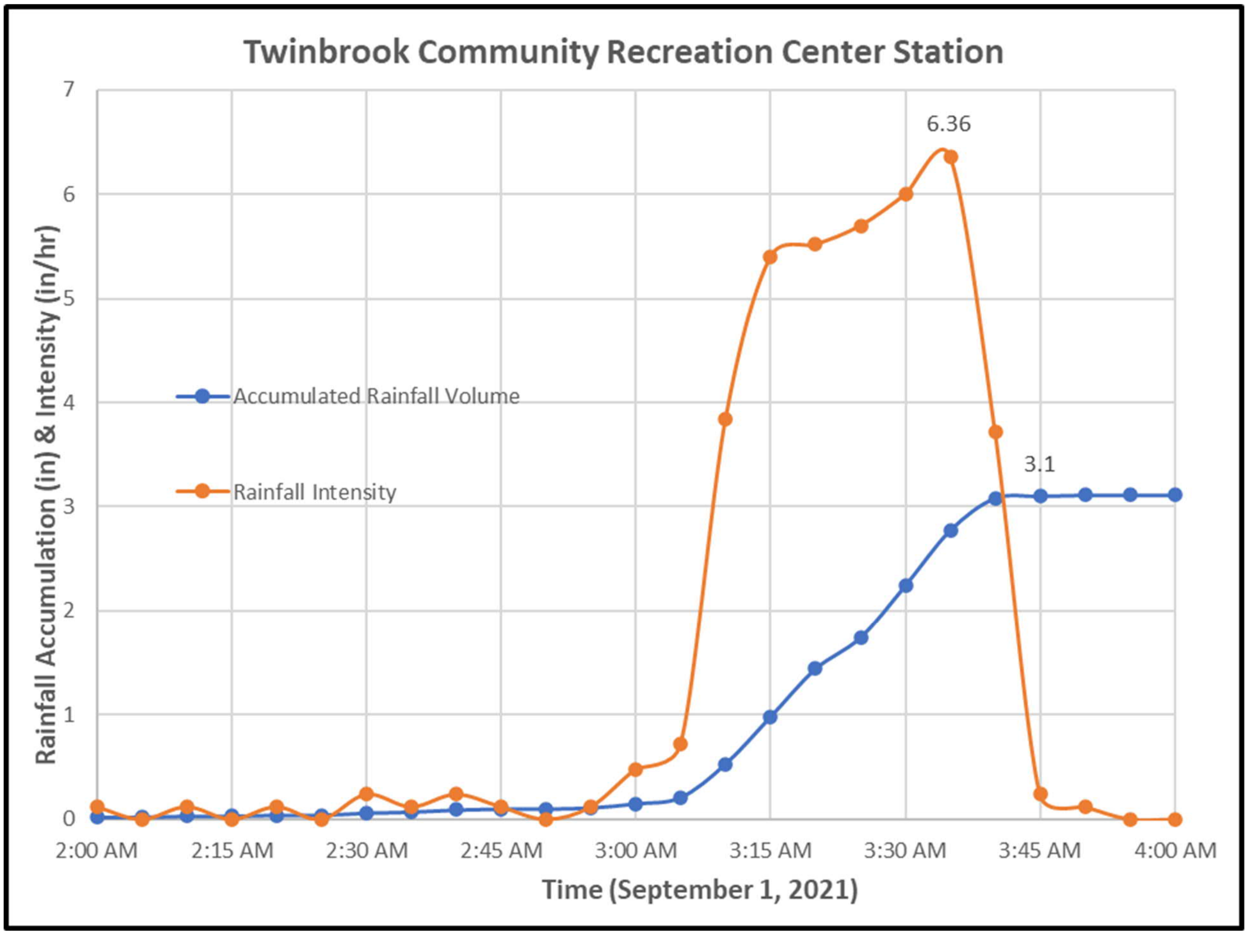

Figure 113 - Twinbrook Community Recreation Center Station - Rainfall Accumulation and Rainfall Intensity.

Table 3 - Twinbrook Community Recreation Center Station Rainfall Precipitation Records

Rainfall Precipitation Records (September 1, 2021)	
Time	**Cumulative Rainfall (Inches)**
02:00 am	0.02
02:05 am	0.02
02:10 am	0.03
02:15 am	0.03
02:20 am	0.04
02:25 am	0.04
02:30 am	0.06
02:35 am	0.07
02:40 am	0.09
02:45 am	0.10

02:50 am	0.10
02:55 am	0.11
03:00 am	0.15
03:05 am	0.21
03:10 am	0.53
03:15 am	0.98
03:20 am	1.44
03:25 am	1.74
03:30 am	2.24
03:35 am	2.77
03:40 am	3.08
03:45 am	3.10
04:00 am	3.10

Figure 109 through Figure 113 show a similar rainfall volume accumulation pattern for all gages between the 2:00 am and 4:00 am on September 1, 2021. Table 4 shows accumulated rainfall volumes for all local weather stations and indicates that all the local rainfall gages recorded similar rainfall accumulation (2 to 4 inches) with the Twinbrook Community Recreation Center Station on the higher end of the range.

Figure 113 also indicates that the highest rainfall intensity of 6.36 in/hr. occurred between 3:30 am and 3:45 am, approximately 10 minutes before the reported Tributary 2 culvert overtopping. This rainfall intensity is also identical to the 6.3 in/hr. rainfall intensity recorded at the Montgomery County Composting facility MDE/Dickerson Gage (Figure 103).

Table 4 - Total Rainfall Volumes for Local Weather Station.

Weather Rainfall Station ID	Local Weather Rainfall Station Name	Total Daily Rainfall Volume for 09/01/2021 (Inches)
6	Weather Station ID: KMDROCKV278	2.34
7	Weather Station ID: KMDROCKV282	2.41
8	Weather Station ID: - KMDROCKV249	2.34
9	Weather Station ID: KMDROCKV245	3.94
10	Twinbrook Community Recreation Center	3.10

6.1.6 Summary of Rain Data Sources Comparison Analysis

The comparison of the local (0.2 miles from project site) Twinbrook Community Recreation Center Station rainfall data intensity and accumulation with regional and local rainfall station data sources resulted in the following determinations:

6.1.6.1 Regional Gages

Nearly all the regional gages (25 miles or less from the project site) reported smaller (1.34 to 2.28 inches) total rainfall volumes than reported by the Twinbrook Community Recreation Center Station for 09/01/2021. The exception was the Baltimore Washington International Airport that reported 4.13 inches. However, this gage is located the furthest (25 miles) from the project site.

The MDE Dickerson Station gage at the Montgomery County Composting facility recorded a total volume of 2.28 inches for 09/01/2021, but also recorded a maximum rainfall intensity (at 15 minutes interval) of 6.3 in/hr., matching the rainfall intensity of the Twinbrook Community Recreation Center Station gage.

6.1.6.2 Local Gages

All local rainfall gages (within 3 miles) recorder similar (2 to 4 inches) rainfall accumulation volumes and align closely with the Twinbrook Community Recreation Center Station volume of 3.1 inches. The spatial and temporal distribution of rainfall intensity is remarkably similar for all stations.

The Twinbrook Community Recreation Center Station rainfall volume also aligns with the RadarScope App Loop data from the 2:00 am to 4:00 am timestamp period.

These observations indicate that the Twinbrook Community Recreation Center Station rainfall data obtained for the Hurricane Ida event on 09/01/2021 is adequate for post-flooding assessment of the culvert overtopping event for the following reasons:

1. The station is located within 0.2 miles of the project site.
2. The stations data logger recorded at 5 minutes increment (adequate for the overtopping analysis that occurred between 3:30 am and 4:00 am.
3. The total rainfall volume and intensity is consistent with similar data from nearby local gages and from unofficial (App-obtained) Radar Loop data at the time of the event.

Therefore, this study utilized the Twinbrook Community Recreation Center Station rainfall data recorded from 08/26/2021 to 09/02/2021 for a detailed 1 and 2-dimensional hydrological and hydraulic/hydrodynamic analysis of the culvert overtopping. This period accounts for the rainfall during the culvert overtopping, and for the 5-day antecedent rainfall occurring prior to the storm event.

6.1.7 Establishment of Return Period for the Rainfall of the Hurricane Ida Storm Event

As part of the hydrologic analysis process, engineers examen return periods of precipitation to determine the time interval in which an event of a given magnitude can reoccur or be surpassed on average and over time. This is accomplished through probability distribution functions that must be analytically fitted to data, where the distribution that fits the best is selected. These results can then be used to evaluate the magnitude of events with different exceedance probabilities or return periods.

The U.S. Department of Commerce National Oceanic and Atmospheric Administration (NOAA), and the NWS have performed rainfall intensity, duration, and frequency for the continental United States[18]. The NOAA Atlas 14, Volume 2, Version 3.0 contains precipitation frequency estimates with associated confidence limits for the United States. Maryland also includes additional information such as temporal distributions and seasonality. The Atlas is the official documentation of precipitation frequency estimates and associated information for the United States. The Atlas supersedes precipitation frequency estimates contained in Technical Paper No. 40 “Rainfall frequency atlas of the United States for durations from 30 minutes to 24 hours and return periods from 1 to 100 years” (Hershfield, 1961), NWS HYDRO-35 “Five- to 60-minute precipitation frequency for the eastern and central United States” (Frederick et al., 1977) and Technical Paper No. 49 “Two- to ten-day precipitation for return periods of 2 to 100 years in the contiguous United States” (Miller et al., 1964). The updates are based on more recent and extended data sets, currently accepted statistical approaches, and improved spatial interpolation and mapping techniques.

NOAA Atlas 14 provides precipitation frequency estimates for durations of 5-minutes through 60-days at average recurrence intervals of 1-year through 1,000-years. It provides the estimates and associated bounds of 90% confidence intervals at 30-arc seconds resolution. The Atlas also includes information on temporal distributions for heavy precipitation amounts for selected durations and seasonal information for annual maximum data used in the frequency analysis.

The NOAA Atlas 14 sources of precipitation data are the daily, hourly, and n-minute measurements of precipitation from the National Weather Service (NWS) Cooperative Observer Program’s (COOP) daily and hourly stations. Records length vary from 95 years for the N-minute dataset, to 101 years from the hourly dataset and 126 years for the daily dataset. The technical approach used in the NOAA Atlas 14 rainfall IDF determinations follows the regional frequency analysis using the method of L-moments. The method of

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[18] The applicable IDF information for the project site is located within the NOAA technical report *NOAA Atlas 14, Precipitation-Frequency Atlas, of the United States Volume 2 Version 3.0: Delaware, District of Columbia, Illinois, Indiana, Kentucky, Maryland, New Jersey, North Carolina, Ohio, Pennsylvania, South Carolina, Tennessee, Virginia, West Virginia Geoffrey M. Bonnin, Deborah Martin, Bingzhang Lin, Tye Parzybok, Michael Yekta, David Riley, U.S. Department of Commerce National Oceanic and Atmospheric Administration National Weather Service Silver Spring, Maryland, 2004 revised 2006.*

L-moments (or linear combinations of probability weighted moments) provides great utility in choosing the most appropriate probability distribution to describe the precipitation frequency estimates. The method provides tools for estimating the shape of the distribution and the uncertainty associated with the estimates, as well as tools for assessing whether the data are likely to belong to a homogeneous region (e.g., climatic regime).

The NOAA Atlas 14 data precipitation frequency (PF) estimates are based on frequency analysis of partial duration series (PDS) and are presented in both table and graphical formats. The table numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Figure 114 shows the NOAA Atlas 14 query screen for Twinbrook, Maryland, Latitude: 39.0747°, Longitude: -77.1210°. Figure 115 shows resulting query curves of the relationship between precipitation depth by duration of the event and the frequency of occurrence (return probability) of the storm event at the selected location.

Table 5 shows the NOAA Atlas 14 Table with point precipitation estimates for the Twinbrook project area used to generate the DD & DF curves of Figure 115**.** Interpolation of the point precipitation frequency estimates shown in Table 5 results in the following determinations:

- The accumulation of 3.10 inches of rainfall in a 2-hour period corresponds to a return frequency of occurrence of approximately 46 years.
- The accumulation of 3.01 inches of rainfall in a 1-hour period corresponds to a return frequency of occurrence of approximately 84 years.
- The accumulation of 2.56 inches of rainfall in a 30-minute period, during the most intense rainfall precipitation with an average of 5.12 in/hr. corresponds to a return frequency of occurrence of approximately 327 years.

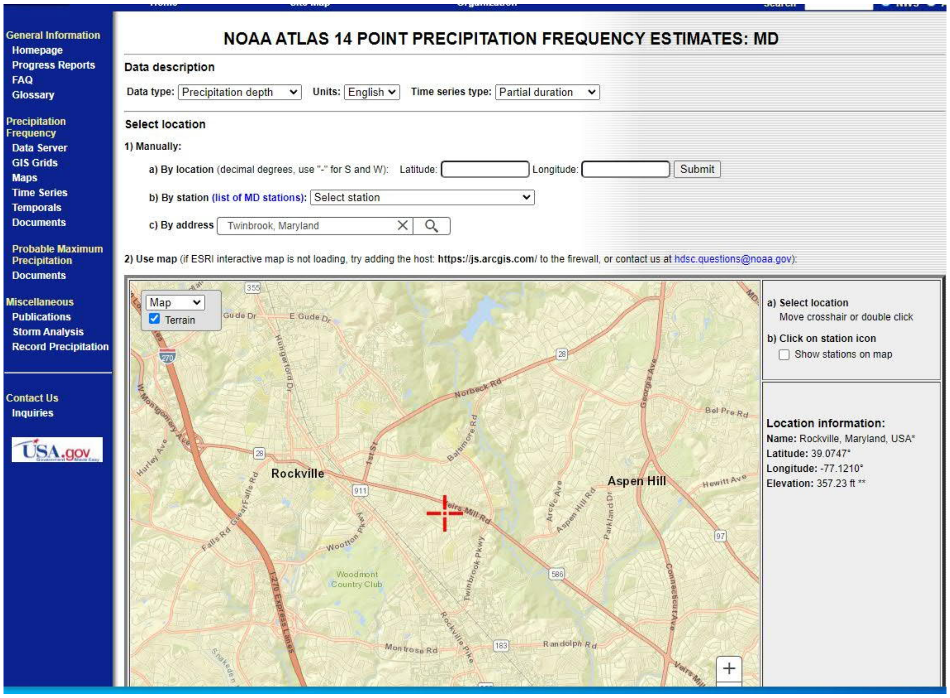


Figure 114 - NOAA Atlas 14 Query Screen.

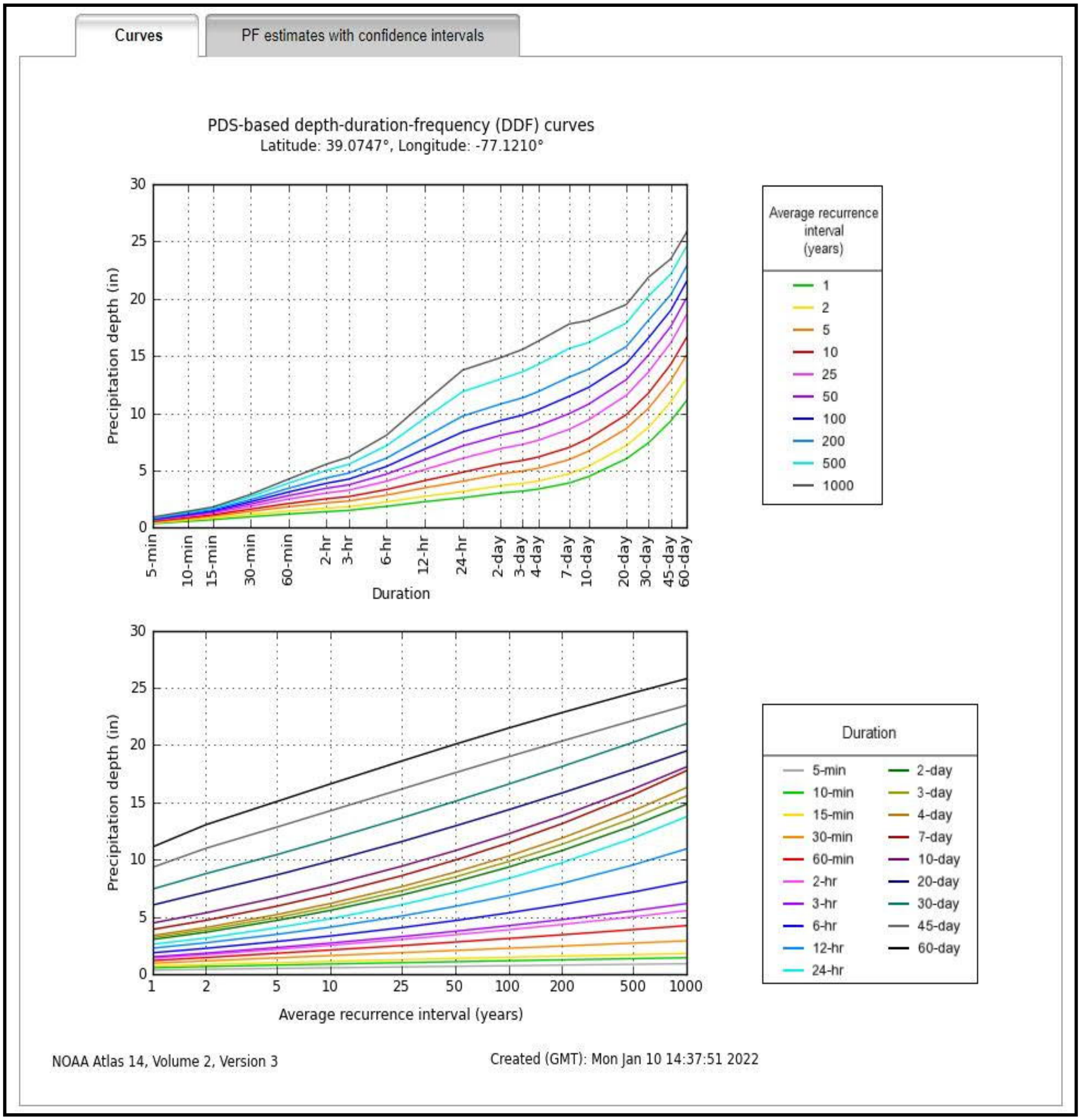


Figure 115 - NOAA Atlas 14 PDS-Based Depth-Duration-Frequency (DDF) Curves.

Table 5 - NOAA Atlas 14 Point Precipitation Frequency (PF) Estimates.

POINT PRECIPITATION FREQUENCY (PF) ESTIMATES

WITH 90% CONFIDENCE INTERVALS AND SUPPLEMENTARY INFORMATION
NOAA Atlas 14, Volume 2, Version 3

PF tabular | PF graphical | Supplementary information | Print page

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**0.346** (0.314-0.382)	**0.415** (0.376-0.456)	**0.493** (0.447-0.544)	**0.552** (0.498-0.608)	**0.625** (0.560-0.689)	**0.680** (0.606-0.750)	**0.734** (0.651-0.812)	**0.787** (0.692-0.873)	**0.854** (0.744-0.953)	**0.907** (0.784-1.02)
10-min	**0.553** (0.501-0.610)	**0.663** (0.601-0.730)	**0.790** (0.715-0.871)	**0.882** (0.796-0.972)	**0.996** (0.892-1.10)	**1.08** (0.965-1.20)	**1.17** (1.03-1.29)	**1.25** (1.10-1.38)	**1.35** (1.18-1.51)	**1.43** (1.23-1.60)
15-min	**0.691** (0.627-0.763)	**0.834** (0.755-0.917)	**1.00** (0.905-1.10)	**1.12** (1.01-1.23)	**1.26** (1.13-1.39)	**1.37** (1.22-1.51)	**1.48** (1.31-1.63)	**1.57** (1.39-1.75)	**1.70** (1.48-1.90)	**1.79** (1.55-2.01)
30-min	**0.948** (0.859-1.05)	**1.15** (1.04-1.27)	**1.42** (1.29-1.57)	**1.62** (1.46-1.78)	**1.87** (1.68-2.06)	**2.07** (1.84-2.28)	**2.26** (2.00-2.50)	**2.45** (2.16-2.72)	**2.71** (2.36-3.02)	**2.90** (2.51-3.25)
60-min	**1.18** (1.07-1.30)	**1.45** (1.31-1.59)	**1.82** (1.65-2.01)	**2.11** (1.90-2.32)	**2.49** (2.23-2.75)	**2.80** (2.49-3.09)	**3.11** (2.76-3.44)	**3.44** (3.02-3.81)	**3.88** (3.38-4.33)	**4.24** (3.66-4.75)
2-hr	**1.40** (1.27-1.54)	**1.70** (1.55-1.88)	**2.16** (1.96-2.38)	**2.52** (2.27-2.77)	**3.02** (2.71-3.32)	**3.44** (3.06-3.78)	**3.87** (3.43-4.27)	**4.34** (3.81-4.79)	**5.00** (4.34-5.56)	**5.54** (4.76-6.20)
3-hr	**1.50** (1.37-1.67)	**1.83** (1.66-2.02)	**2.32** (2.10-2.56)	**2.71** (2.44-2.99)	**3.27** (2.92-3.61)	**3.74** (3.32-4.12)	**4.23** (3.73-4.68)	**4.76** (4.15-5.28)	**5.53** (4.76-6.16)	**6.16** (5.24-6.91)
6-hr	**1.86** (1.69-2.06)	**2.25** (2.04-2.50)	**2.84** (2.57-3.15)	**3.33** (3.00-3.69)	**4.05** (3.62-4.49)	**4.67** (4.13-5.17)	**5.34** (4.68-5.92)	**6.08** (5.27-6.76)	**7.16** (6.11-8.02)	**8.08** (6.79-9.09)
12-hr	**2.26** (2.04-2.55)	**2.73** (2.46-3.07)	**3.47** (3.11-3.90)	**4.10** (3.66-4.60)	**5.07** (4.47-5.67)	**5.91** (5.17-6.62)	**6.86** (5.92-7.70)	**7.92** (6.74-8.91)	**9.53** (7.95-10.8)	**10.9** (8.96-12.4)
24-hr	**2.60** (2.38-2.89)	**3.15** (2.88-3.49)	**4.04** (3.69-4.48)	**4.83** (4.39-5.34)	**6.05** (5.45-6.64)	**7.13** (6.37-7.80)	**8.35** (7.38-9.10)	**9.74** (8.51-10.6)	**11.9** (10.2-12.9)	**13.7** (11.6-14.9)
2-day	**3.02** (2.76-3.34)	**3.66** (3.34-4.05)	**4.68** (4.27-5.19)	**5.57** (5.06-6.15)	**6.90** (6.22-7.59)	**8.06** (7.22-8.85)	**9.35** (8.31-10.2)	**10.8** (9.50-11.8)	**13.0** (11.2-14.2)	**14.8** (12.7-16.3)
3-day	**3.19** (2.92-3.53)	**3.86** (3.53-4.28)	**4.94** (4.51-5.47)	**5.87** (5.34-6.48)	**7.26** (6.56-7.99)	**8.48** (7.61-9.31)	**9.83** (8.74-10.8)	**11.3** (9.99-12.4)	**13.6** (11.8-14.9)	**15.6** (13.3-17.1)
4-day	**3.37** (3.08-3.72)	**4.07** (3.73-4.51)	**5.20** (4.75-5.75)	**6.17** (5.62-6.81)	**7.63** (6.90-8.40)	**8.90** (7.99-9.78)	**10.3** (9.18-11.3)	**11.9** (10.5-13.0)	**14.3** (12.4-15.6)	**16.3** (14.0-17.9)
7-day	**3.90** (3.59-4.28)	**4.69** (4.32-5.16)	**5.93** (5.45-6.51)	**6.99** (6.41-7.66)	**8.58** (7.81-9.37)	**9.95** (8.99-10.9)	**11.5** (10.3-12.5)	**13.1** (11.7-14.3)	**15.6** (13.7-17.1)	**17.8** (15.4-19.5)
10-day	**4.45** (4.10-4.88)	**5.35** (4.93-5.86)	**6.68** (6.14-7.31)	**7.79** (7.14-8.52)	**9.41** (8.59-10.3)	**10.8** (9.77-11.7)	**12.2** (11.0-13.3)	**13.8** (12.4-15.1)	**16.2** (14.3-17.6)	**18.1** (15.8-19.8)
20-day	**6.02** (5.60-6.52)	**7.17** (6.65-7.75)	**8.66** (8.03-9.36)	**9.87** (9.15-10.7)	**11.6** (10.7-12.5)	**12.9** (11.9-13.9)	**14.3** (13.2-15.5)	**15.8** (14.4-17.0)	**17.9** (16.2-19.3)	**19.5** (17.5-21.1)
30-day	**7.42** (6.92-7.96)	**8.78** (8.19-9.42)	**10.4** (9.73-11.2)	**11.8** (11.0-12.6)	**13.6** (12.6-14.6)	**15.1** (14.0-16.2)	**16.6** (15.3-17.8)	**18.1** (16.6-19.4)	**20.2** (18.5-21.7)	**21.9** (19.8-23.5)
45-day	**9.32** (8.75-9.93)	**11.0** (10.3-11.7)	**12.8** (12.1-13.7)	**14.3** (13.4-15.2)	**16.2** (15.1-17.2)	**17.6** (16.4-18.7)	**19.0** (17.7-20.2)	**20.4** (18.9-21.7)	**22.1** (20.5-23.6)	**23.5** (21.7-25.1)
60-day	**11.1** (10.5-11.8)	**13.1** (12.3-13.8)	**15.1** (14.2-16.0)	**16.6** (15.6-17.6)	**18.6** (17.5-19.7)	**20.1** (18.8-21.3)	**21.5** (20.1-22.8)	**22.8** (21.3-24.2)	**24.6** (22.8-26.1)	**25.8** (23.9-27.4)

6.2 H&H Modeling Methodology

6.2.1 Software & Model Selection

In review of available H&H modeling platforms, the Streamline Technologies - Interconnected Channel and Pond Routing Model (ICPR) Version 4 was selected as the software to develop the Rock Crest - Twinbrook model.

ICPR4 has become well established across the eastern United States and has been used for an array of purposes, including the modeling of pond systems with groundwater effects, urban stormwater infrastructure, and FEMA regulatory floodplains. The software has undergone several updates since its release over 30 years ago, with the latest iteration released in 2014. The current Version 4 of the program is capable of modeling 1-dimensional (1D) conduit flow while fully integrating 2-dimensional (2D) surface/groundwater flow.

The ICPR4 model for the Rock Crest - Twinbrook drainage areas was developed using both a 1D and a hybrid 1D/2D approach; the purpose of each, respectively, being to calculate peak flows/stages at the Rock Crest & Twinbrook culverts and to evaluate the floodplains produced by Hurricane Ida and categorical storms.

The 1D-only model is a traditional H&H model in the sense that surface runoff generation takes place over delineated drainage basins, with subsequent overland flows described through statistically-derived equations. However, ICPR4 is somewhat more sophisticated in the calculation of rainfall excess than with most models. This aspect is discussed later in this report (see the section on 6.2.3.4 Volume Methods).

The hybrid 1D/2D model introduces what is known as a “rain on mesh” component to the 1D model, specifically over the regions of the two natural channels of Tributaries 1 and 2, the land areas over the culverts, and the downstream stretch of open channel ultimately discharging to Rock Creek. See Figure 116 for a visual reference of the 2D model zone (symbolized in yellow), the irregular boundary of which follows the edges of delineated 1D sub-catchments. Note, the areas outside of the 2D zone but within the total drainage area (symbolized in black) are modeled in 1D.

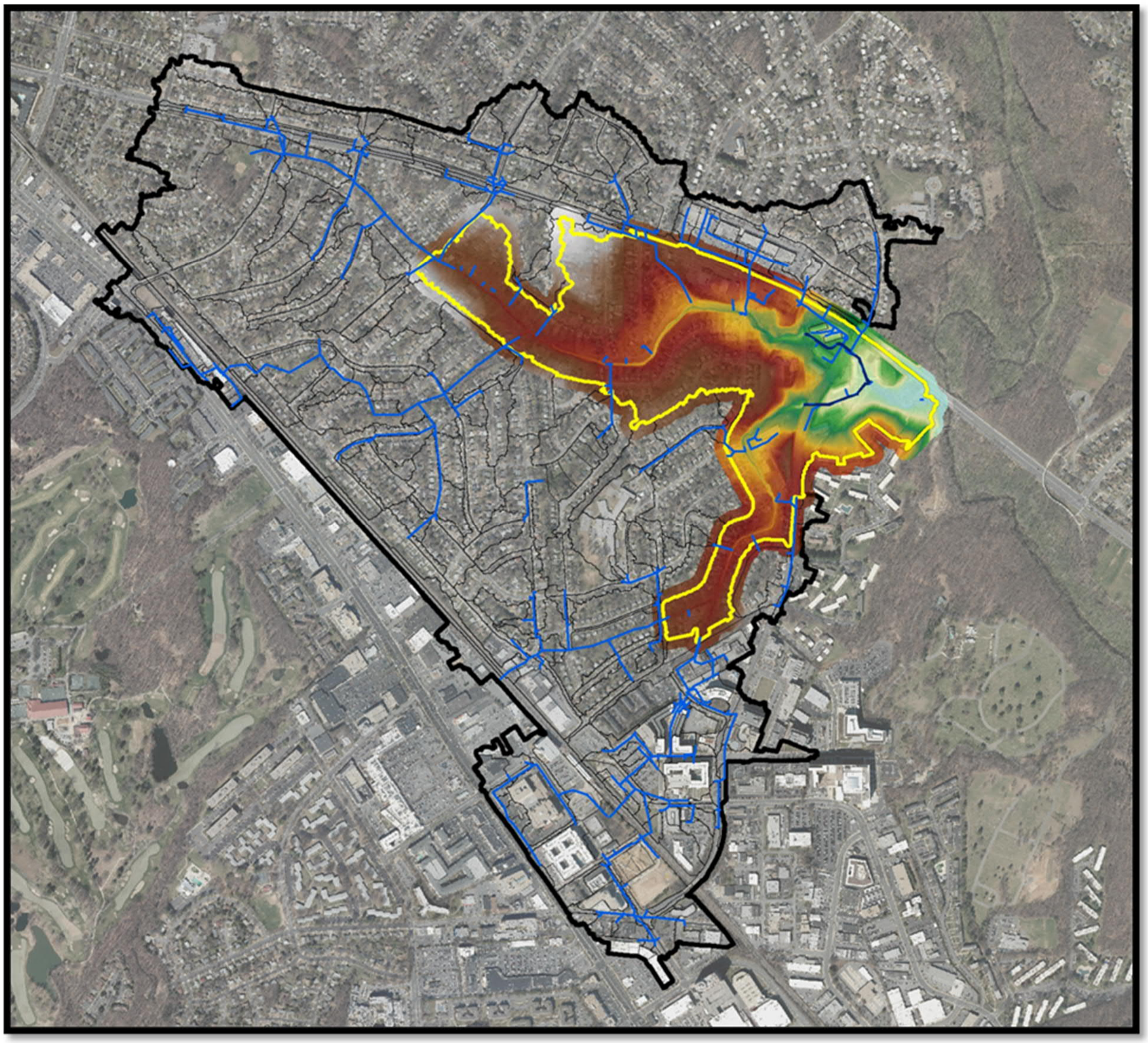


Figure 116 - Map showing the 1D and 2D model zones. Blue lines indicate the storm drain system within the model zones.

The rain on mesh component essentially takes the 1D approach to a minutely detailed degree. Across the aforementioned regions, previously delineated drainage basins are instead replaced with a grid (in ICPR4's case a "honeycomb") of many thousands of smaller surface basins, on which rainfall excess is calculated. Then, overland flow is routed between each honeycomb cell via an overlaying mesh of interconnected links and nodes. These overland flows are calculated directly through forms the well-known St. Venant equations, allowing the model to produce dynamic runoff hydrographs as opposed to the watershed-based synthetic runoff hydrographs of a 1D model. With the mesh coupled to the 1D hydraulic network, the 2D model can also capture any surcharge flooding produced by storm drain backflow onto the surface.

6.2.2 Model Input Data

6.2.2.1 Hydraulic Network

A GIS database referred to as MC-StormNET, managed by Montgomery County (MC), and produced for and by engineers, was used as the initial basis for the physical representation of existing storm drain pipes and structures in the Rockville, MD area. The dataset included information related to pipe geometry, invert elevations, materials, and structural information such as type, and rim/grate elevation.

A comprehensive review of the data was conducted prior to importing the assets into ICPR4. The review consisted of an overall assessment of the source data and an identification of any missing information or information inconsistent with known field conditions.

Where possible, assets were geographically and typologically verified for accuracy via aerial imagery, Google Earth, or field investigations by engineers/surveyors.

Throughout the QA/QC process, any gaps or errors found in the hydraulic data (invert elevations, physical size/shape, etc.) were ultimately corrected by referencing survey data, supplemental sources of information such as as-builts, or interpolation/assumption by the engineers' judgement.

In specific regard to missing invert elevations, inverts were interpolated by initially assuming positive pipe slope and flowline depth of 5 feet from the surface, cover permitting. The interpolation was further refined via adopting a design constraint that the pipe slope produces a full flow velocity of at least 2.5 feet per second (fps) and ideally less than 15 fps, with application of the Manning equation. In special cases, governed by terrain elevations, velocities higher than 15 fps but no greater than 25 fps were permitted. This range of flow velocities is consistent with hydraulic design standards throughout Maryland.

Of the reviewed assets, nearly 600 storm drain pipes were retained for ICPR4 modeling. Some pipes were added manually per the methodology noted above. In total, the ICPR4 model consisted of 614 pipes, serving approximately 30 individual storm drain networks. The complete modeled storm drain system can be seen in Figure 116 (blue lines). Generally, roof drainage and pipes less than 15-inches diameter were not modeled. In select locations such small pipes were retained for the accurate capture of drainage flows. The majority of the pipes were of less than 30-inches diameter or equivalent size. In fact, 66% of the pipes were between 15- and 24-inches diameter. Figure 117 below displays the full distribution of pipe sizes captured in the model.

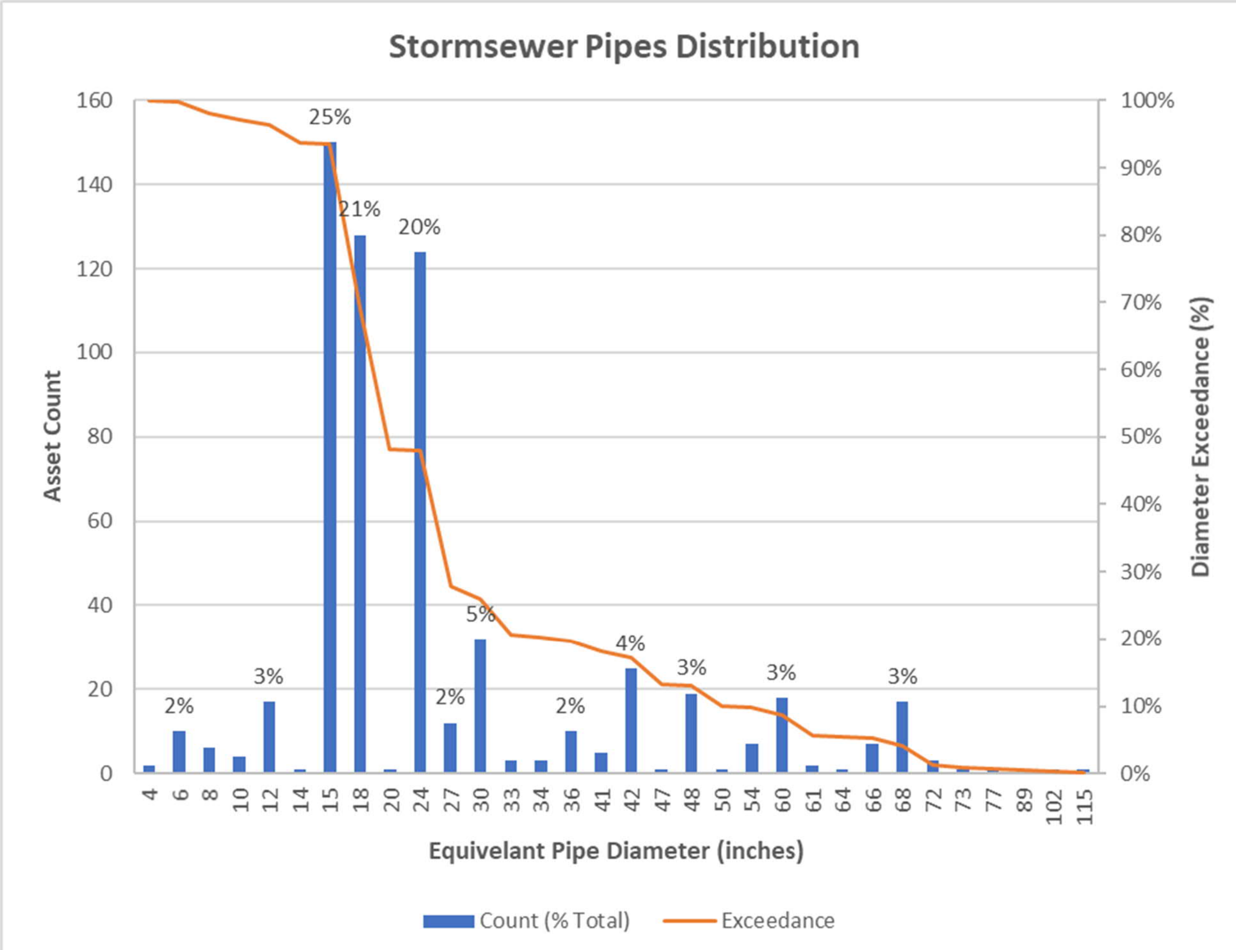


Figure 117 - ICPR Model Pipe Diameter Distribution

6.2.2.2 Topography

A Digital Elevation Model (DEM) is required to model overland surface flows in ICPR4. WRMA was able to acquire a bare earth DEM raster[19] of the project area derived from 2018 MNCPPC Light Detection and Ranging (LiDAR) point cloud data for Montgomery and Prince Georges Counties, MD. Figure 118 below displays an image of the DEM symbolized to highlight Tributaries 1 and 2 discharging into the box culverts.

[19] Montgomery County Planning, https://montgomeryplanning.org/tools/gis-and-mapping/data-downloads/

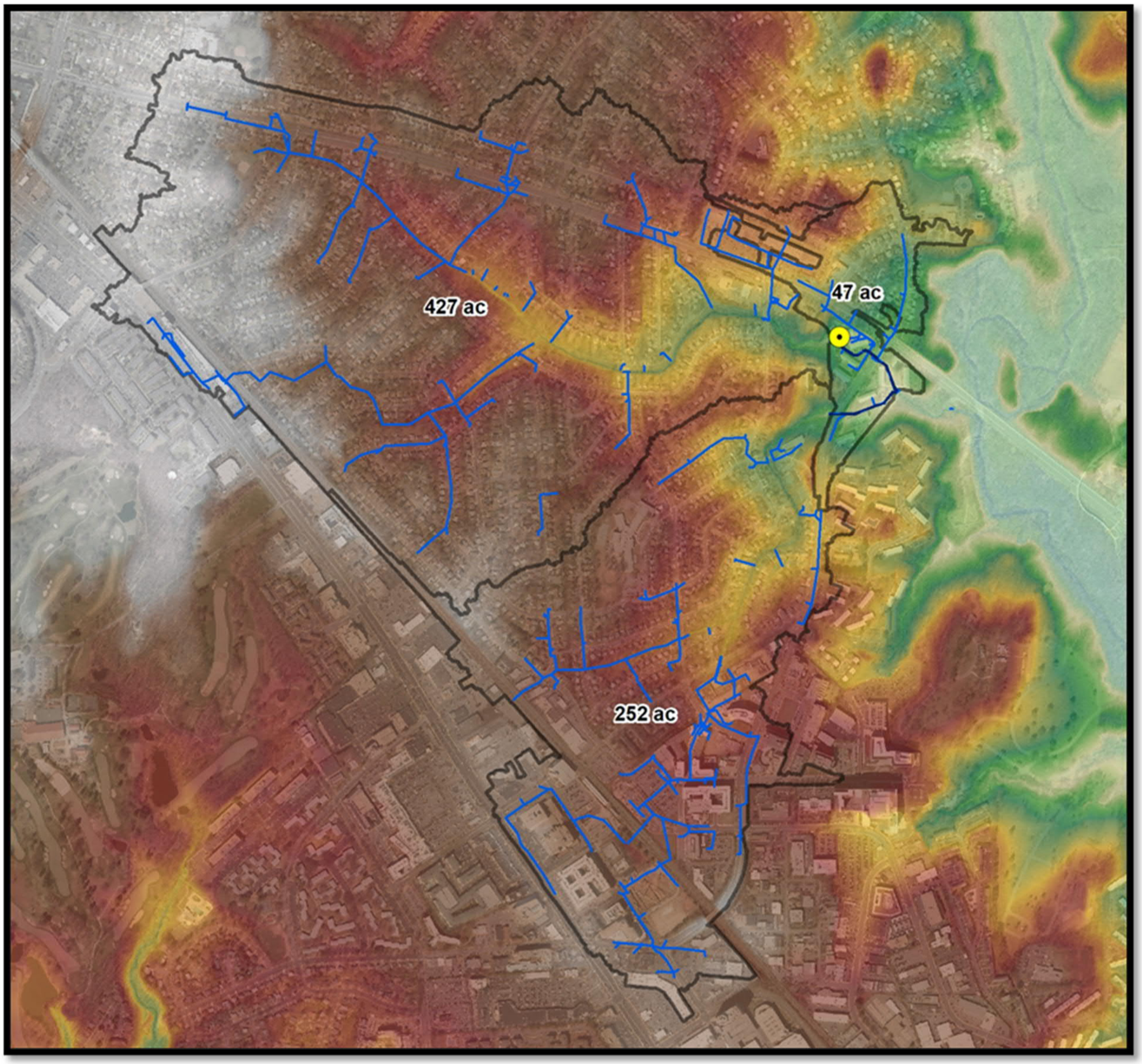


Figure 118 – LiDAR DEM for Montgomery and Prince Georges Counties, MD

This DEM represents the best topographic dataset available for the Rockville area at the time of writing, with a rasterized resolution of 2 feet x 2 feet; suitable for 2D modeling.

The bare earth DEM was previously verified to have a vertical accuracy of ±0.298 feet (±3.6 inches), well within the limits defined by the National Standards for Spatial Data Accuracy (NSSDA). However, LiDAR point shots can be obstructed by dense vegetation or standing water, making point returns along the thalweg of streams particularly vulnerable to vertical error. For that reason, the DEM was further reviewed against surveyed cross-sections taken by Mercado Consultants. The DEM was found to be sufficiently close [for H&H modeling] to the surveyed elevations, generally within inches but in a few areas up to a foot difference. For example, Figure 119 below shows a composite cross-section taken along Tributary 2 (referred to as "Trib-A" for modeling purposes). In review, it was noted that the LiDAR appeared less accurate surrounding the entrances and exits of culvert crossings throughout the Tributaries, likely due to higher water depths at those locations.

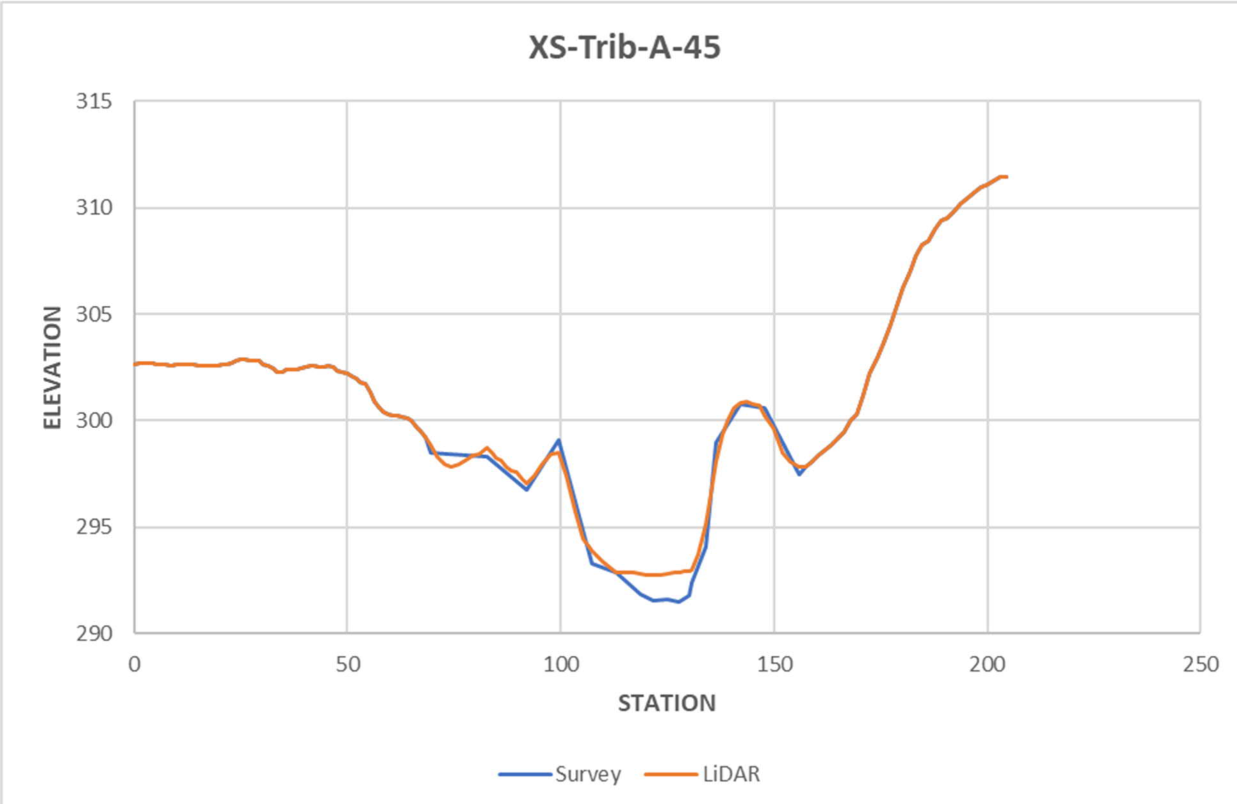


Figure 119 - Example Cross-Section of Tributary 2

Given the nature and intent of this modeling effort, the LiDAR was supplemented with surveyed data by Mercado Consultants.

The 1D H&H model is composed of 126 stream cross-sections, 50 of which include surveyed points at locations critical for the 1D hydraulic modeling effort. The remaining sections were generated solely from the LiDAR at locations sufficient to capture changes in stream geometry, and at locations in-between to maintain a segmental distance of no greater than 200 feet. As for the 2D model, which computes flows directly at the surface, the LiDAR DEM was combined (overwritten) with a number of AutoCAD Civil3D generated surfaces derived from detailed topographic survey by Mercado Consultants, as depicted in Figure 120.

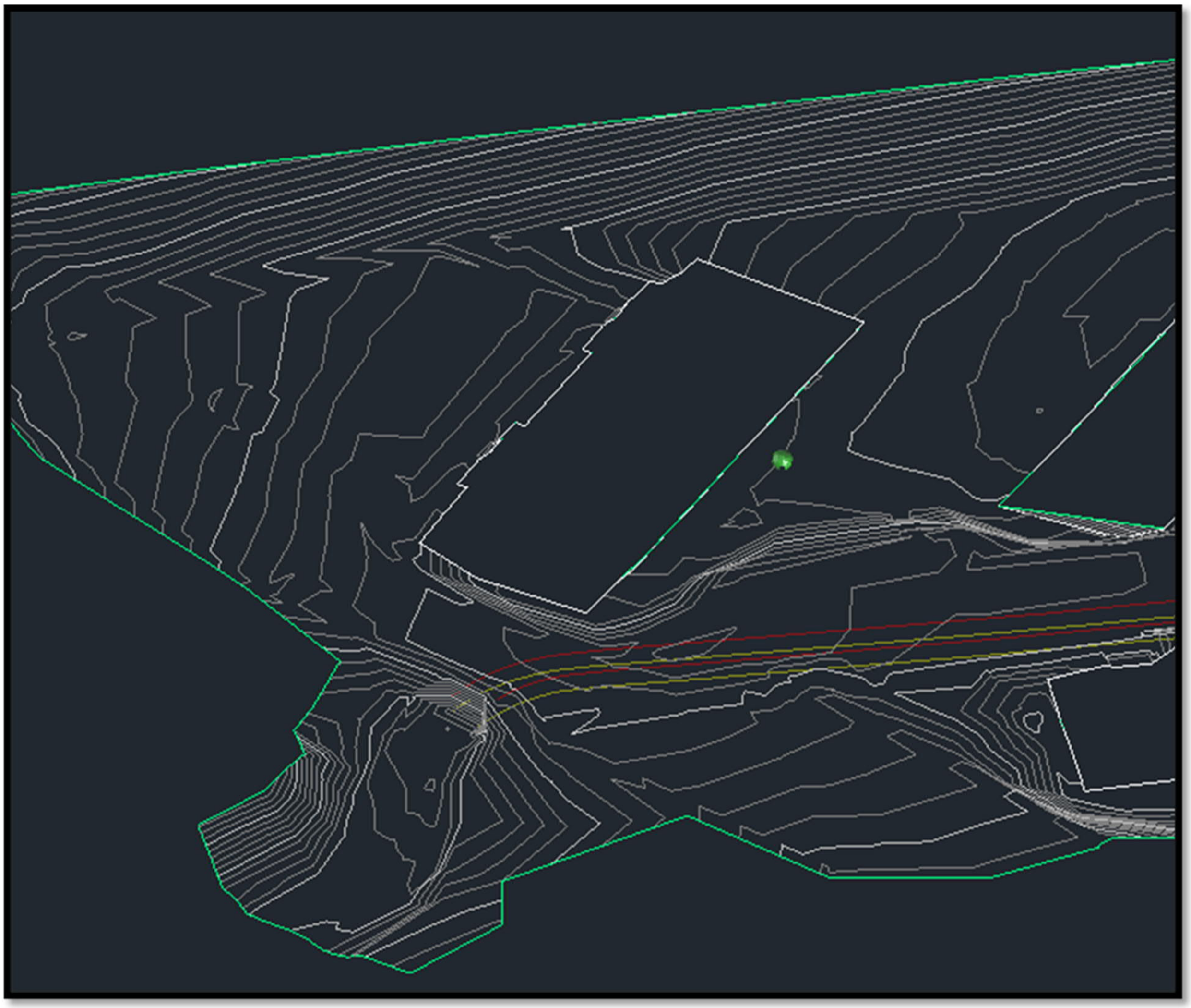


Figure 120 - Snapshot of survey surface contours displayed in AutoCAD Civil3D

The survey surfaces were limited to critical areas such as the affected residential properties, culvert crossings and pedestrian bridges along the connected streams. Figure 121 displays the exact domain of each surveyed surface (symbolized in black) incorporated into the composite DEM of the 2D model.

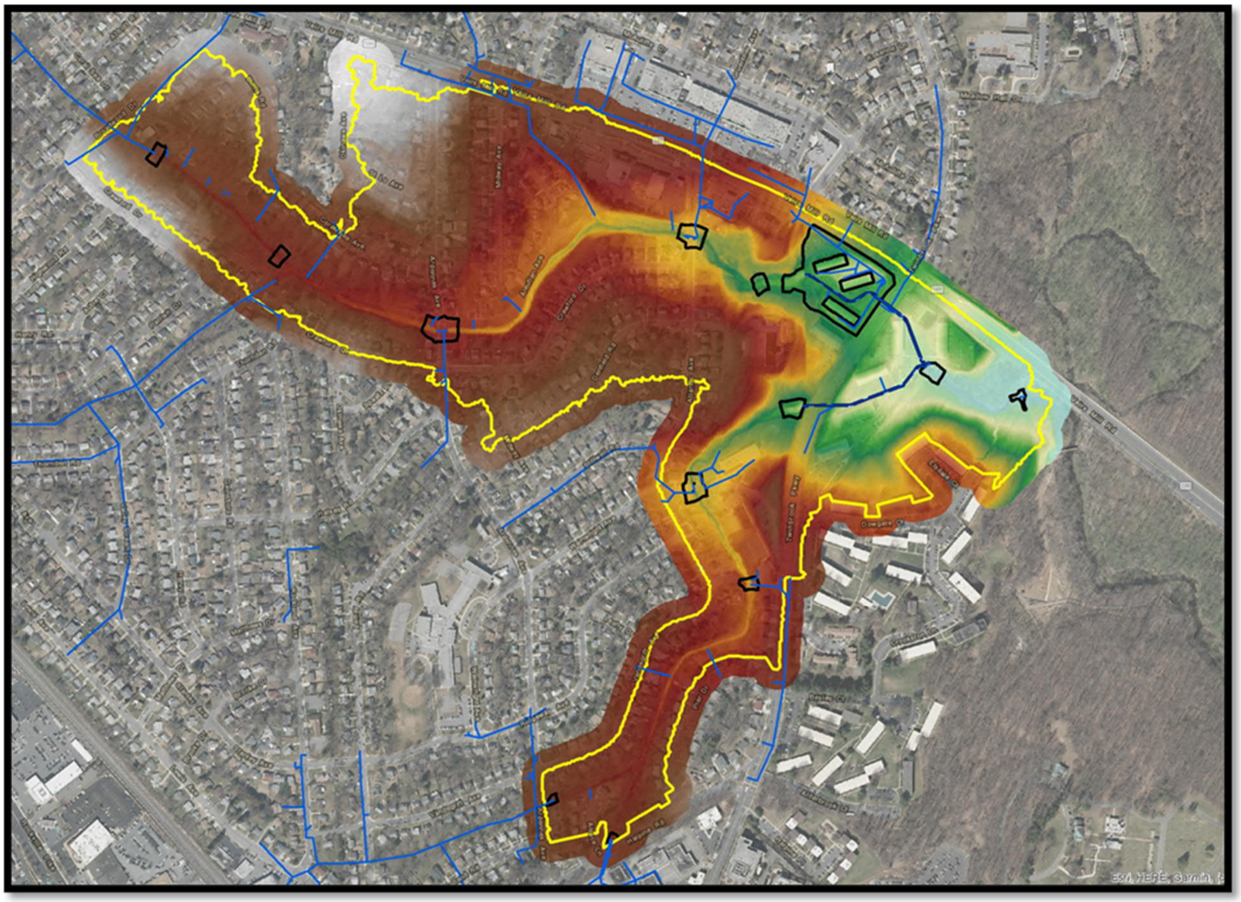


Figure 121 - Map showing the domains of surveyed surfaces within the 2D zone (outlined in black)

6.2.2.3 Soils

Soil characteristics for the Rockville, Maryland area were attained from the Web Soil Survey published by the Natural Resources Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS), an agency of the U.S. Department of Agriculture.

Under the Unified Soil Classification System (USCS), soils are classified within three primary buckets: coarse grained, fined grained, and highly organic. These classes are further sub-divided for other descriptive qualities, all having a two-letter designation (or a hybrid designation). As shown in Figure 122, there are 3 classified soil groups across the Rock Crest and Twinbrook drainage areas (some areas in gray are unclassified), with most of the soil being classified as fine grained – inorganic silts or clays.[20]

[20] The liquid limit is the fractional water content at which a soil begins to exhibit liquid qualities.

1. ML – inorganic silts (liquid limit 50% or less)
2. MH – inorganic, elastic silts (liquid limit greater than 50%)
3. CL – inorganic clays (liquid limit 50% or less)

It should be noted here that throughout multiple field investigations, standing water and/or minor flow was observed in thalwegs of the tributary streams. As such, a baseflow of 0.25 cubic feet per second (cfs) was assumed at each major outfall (serving very large storm drain networks).

In hydrology, Hydrologic Soil Groups (HSGs) are a categorical method for engineers to describe the degree to which water can infiltrate downward through a soil. There are 4 HSGs, A through D, which can be summarized as progressively impeding the downward movement of water as you proceed alphabetically. There are also dual designations in which certain wet soils have the letter D assigned (e.g., A/D, B/D, C/D). This is due to the presence of a water table within 2 feet (60 cm) of the surface, providing a marginal infiltration and storage capacity. Following the USCS classes, the project drainage area has 3 HSGs with its boundaries.

1. B – moderate infiltration rate
2. C/D – low infiltration rate if well drained, nearly impervious if not
3. D – very low infiltration rate

Silts and clays tend to provide poor infiltration capacities and therefore higher levels of runoff during storms.

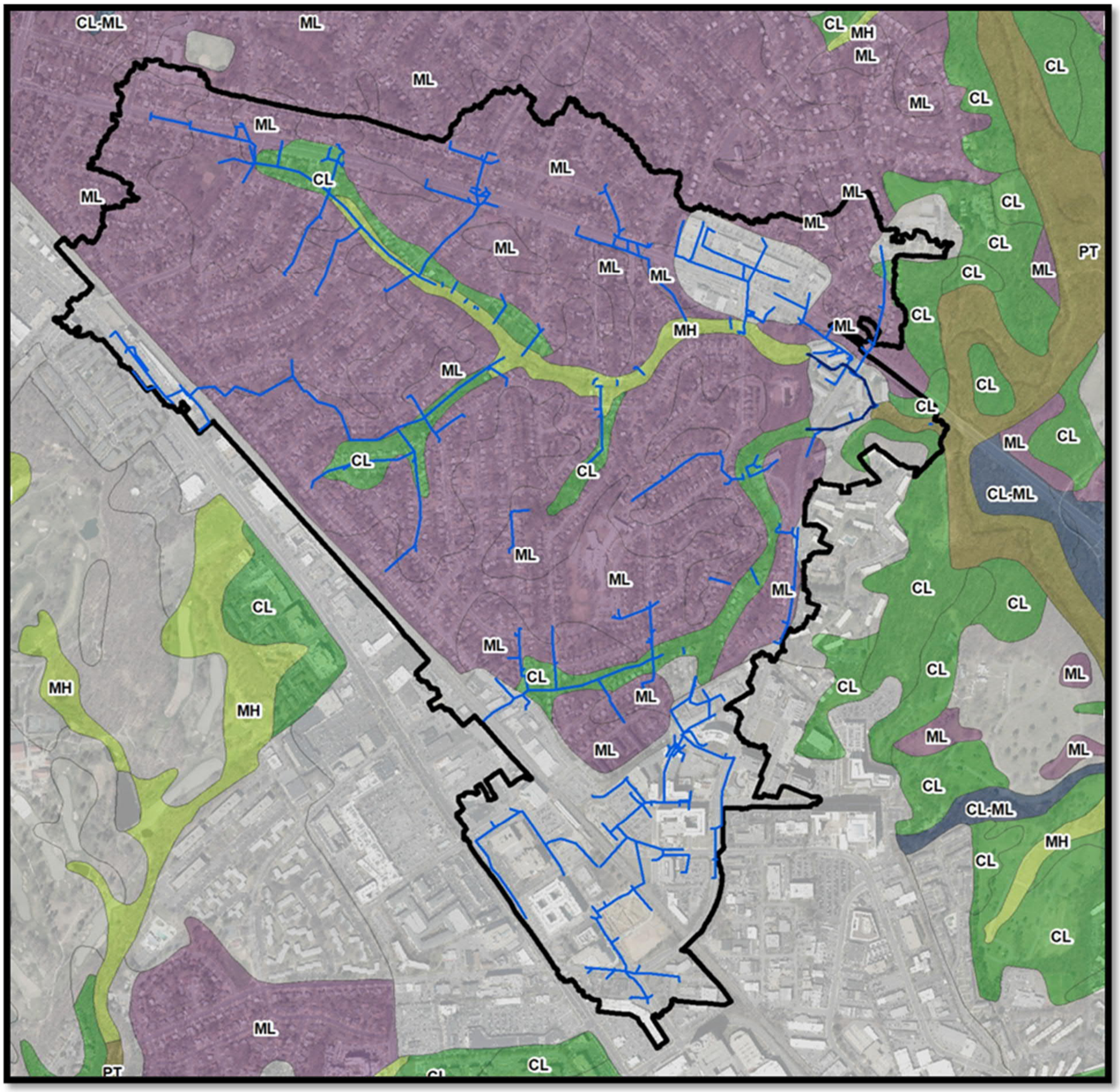


Figure 122 - USCS classification of soils in the drainage area

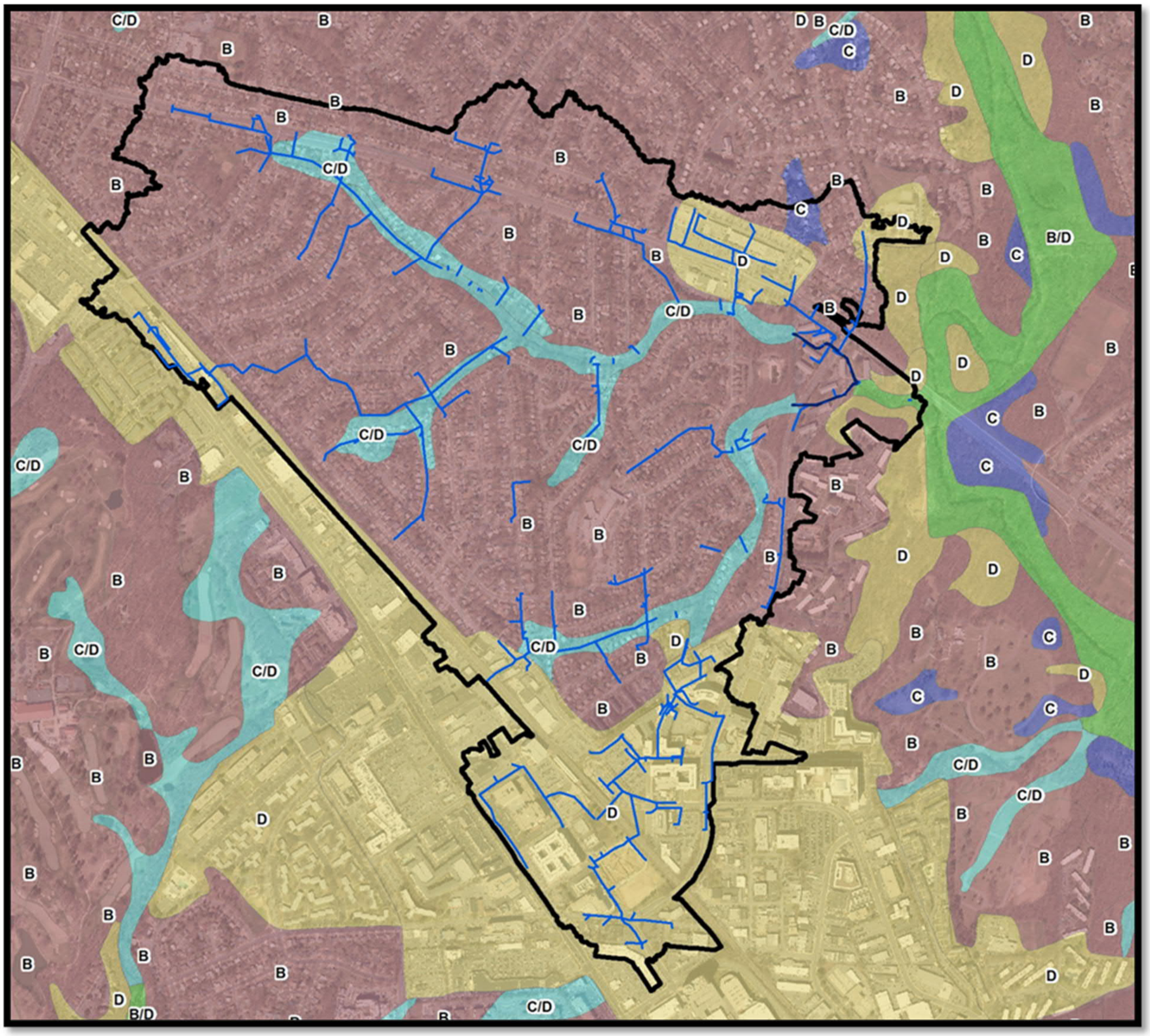


Figure 123 - NRCS Hydrologic Soil Groups

A full report of the area’s soil characteristics can be referenced in Appendix B.

6.2.2.4 Landcover

Land Cover or Land Use datasets are a traditional method of describing the relative roughness and imperviousness of terrain, aside from soil infiltration capacity. These divide the terrain into typological classifications such as Grass, Forest, Roads, and Crop Land. These classifications have been widely used amongst engineers and researchers, thus there are long-established typical roughness values and measures of imperviousness for them.

For this H&H modeling, a highly detailed Land Cover dataset (shown in Figure 124) was generated through compiled GIS layers sourced from Montgomery County Planning’s GIS database and supplemental GIS layers generated by the engineer. This land cover dataset classified the drainage area into the following 11 classes:

- Bare Soil;
- Building;
- Creek (further sub-divided);
- Forest;
- Grass/Shrub;
- Other Paved;
- Railroad;
- Road;
- Tree Canopy;
- Water;
- Wetland

Figure 124 - Landcover dataset developed for H&H Model

Roughness values (Manning's *n*) and runoff Curve Numbers (*C*) were assigned to each land cover designation. Further information on this process can be found in Section 6.2.3 Basis of Hydrology and Hydraulics.

6.2.2.5 Boundary Conditions

The Rock-Crest and Twinbrook drainage basins ultimately discharge to Rock Creek near the Veirs Mill Rd bridge. No other tributary drainage (natural or urban) exists upstream of these basins. Therefore, the approximate location of the Rock Creek floodplain serves as the sole boundary condition of the H&H models.

While no stream gauge data was available in suitable proximity to the discharge point, there were first-hand accounts by engineers of field conditions during the days following Hurricane Ida event. High water marks were visible in the immediate area downstream of the culverts and near the bridge.

Using GIS tools and the LiDAR DEM, an estimated stream stage of 254.0 feet NAVD88 approximately agreed with the observed highwater markings caused by the Ida storm event. A stage of 246.5 feet NAVD88 was likewise found to be a close approximation of normal dry weather conditions.

For determining the 100-year and 500-year downstream boundary conditions, the extent of FEMA National Flood Hazard Layer was reviewed. This layer displays the effective Rock Creek 100-year and 500-year regulatory floodplains previously mapped by FEMA. A stream stage of 256.5 feet was a good match to the extents of the 100-year floodplain, and 258.75 feet to the 500-year.

Considering the bounds of the dry-weather and 100-year stages, boundary stages for intermediate storms were interpolated by following the rate of change of NOAA Atlas 14 Precipitation Frequency Estimates for corresponding recurrence intervals. Table 6 displays all the boundary stages used for each categorical storm.

Table 6 - Rock Creek Boundary Stages (Feet, NAVD 88)

Recurrence Interval	NOAA PFE (24-hour)*	Boundary Stage*
1-year	2.61	246.50
2-year	3.15	***247.44***
5-year	4.05	***249.00***
10-year	4.84	***250.38***
11-year	***4.92***	***250.52***
12-year	***5.00***	***250.66***
13-year	***5.08***	***250.80***
14-year	***5.17***	***250.94***
15-year	***5.25***	***251.09***
25-year	6.06	***252.50***
Hurricane Ida	N/A	254.00
50-year	7.14	***254.38***
100-year	8.36	256.50
500-year	11.9	258.75

**Values in blue italics were interpolated.*

6.2.3 Basis of Hydrology and Hydraulics

The procedures applied in post-flood hydrologic analysis are usually chosen according to jurisdictional and site-specific requirements. The following subsection detail the methodology used for this H&H study, as appropriate for Montgomery County, MD.

6.2.3.1 Basin Delineation

The delineation of drainage basins for the 1D model was primarily completed via a supervised Arc Hydro (GIS) algorithm[21], with hands-on adjustments and additions as necessary to accurately capture the urban drainage patterns.

Drainage basins (subcatchments) were generated at the inlet level. For modeling purposes, very small subcatchments were combined into larger adjacent subcatchments. In total the 1D model consists of 424 subcatchments ranging from ~3,000 square feet to ~20 acres, and with time of concentrations ranging from 6 minutes to 41 minutes. Altogether the catchments reflect surface drainage patterns for on-grade and sag inlets, some rooftop drainage (in very urban areas), ponds/BMPs, and direct drainage to streams.

For the 2D model, previously delineated 1D subcatchments were removed in areas covered by the 2D mesh. Drainage patterns are directly captured by the overland flows computed on the mesh. 1D subcatchments outside of the 2D mesh zone remained, with their associated discharges coupled to the mesh at outfall locations.

[21] https://www.esri.com/en-us/industries/water-resources/arc-hydro

6.2.3.2 Roughness Assignments

In the selection of overland Manning's n roughness assignments for each landcover, a comprehensive review of engineering literature was conducted in order to tailor the values to be representative of the Rockville area. Literature included reports and studies published by the NRCS, the Hydraulic Engineering Center (HEC), the Federal Highway Administration (FHWA), and Montgomery County. The chosen roughness coefficients were ultimately adaptations of values originally put forth by Ven Te Chow (1959), Engman (1986), and McCuen (1998). Table 7 shows the finalized values used in model.

Table 7 - Manning's *n* by Land Cover

Landcover	Manning's *n*	Source	Notes
Bare Soil	0.01	Multiple	Typical
Building	0.011	Montgomery County[22]	Used equivalent of "Smooth Asphalt"
Forest	0.4	Montgomery County	Used "Light Underbrush"
Grass/Shrub	0.15	Montgomery County	Used "Short Grass"
Other Paved	0.012	Montgomery County	Used "Smooth Concrete"
Railroad	0.03	Multiple	Used high estimate of typical range
Road	0.015	Multiple	Used high estimate of typical range
Tree Canopy	0.32	Montgomery County	Used average of "Dense Grass" and "Light Underbrush"
Water	0.04	Ven Te Chow	Used "Open Water"
Wetland	0.12	Ven Te Chow	Used "Woody Wetland"
Creek	0.042	USGS/FHWA[23]	Used "Major Streams - Regular Section" median value
Creek Bank Boulder	0.055	USGS/FHWA	Used median value of range
Creek Bank Concrete	0.012	USGS/FHWA	Used low end of range
Creek Bank Grass/Shrub	0.12	Ven Te Chow	Used "Woody Wetland"
Creek Bank Riprap	0.035	Multiple	Typical
Creek Bank Stone	0.015	N/A	Used same as concrete
Creek Bed Boulder	0.055	USGS/FHWA	Used median value of range
Creek Bed Cobble	0.04	USGS/FHWA	Used median value of range
Creek Bed Concrete	0.015	USGS/FHWA	Used median value of range
Creek Bed Mixed Cobble/Boulder	0.0475	N/A	Used average of Cobble and Boulder
Creek Bed Sand	0.026	USGS/FHWA	Used low end of range for "Coarse sand"

[22] Montgomery County, MD, *Drainage Design Criteria*, *10th Ed.*, 2014
[23] U.S. Geological Survey (USGS), in cooperation with the Federal Highway Administration (FHWA), *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, 1989

For the routing within the storm drain conduits, typical Manning's n roughness coefficients were used for each conduit material, shown in Table 8 below.

Through field investigations a visible layer of rough, though minor, sediment was observed through the span of the culverts. As such, and unless otherwise indicated by laser scans of the culverts, a Manning's roughness factor of 0.017 was used for the bottom slab throughout the culverts.

Furthermore, the two 36x60-inch arched outfalls at Halpine Road were observed to have the bottom half of their cross-sections lined with concrete, and the top half lined with corrugated metal. This concrete was likely placed as a repair for past deterioration in the culvert invert. Since the wetted perimeter during full flow covers both materials, an averaged Manning's n of 0.018 was applied to both outfalls.

Table 8 - Storm Drain Roughness by Material

Material	Manning's *n*
Brick	0.015
CIP	0.012
CMP	0.024
HDPE	0.011
PE	0.01
PVC	0.01
RCP	0.012

6.2.3.3 Hydraulic Losses

To accurately capture head losses throughout the 1D model elements, hydraulic loss coefficients were applied for the following:

- Conduit flow across structural junctions such as inlets or manholes, variable by degree of bend[24];
- Bends in open channels and conduits (mid-span)[25];
- Expansions/Contractions of open channels[26] and conduits[27];
- Entrance/Exit losses at culverts[28];
- Potential hydraulic jumps (eddy losses)[27]; and
- Weir discharge[24].

[24] Federal Highway Administration (FWHA), *HEC-22 Urban Drainage Design Manual, 3rd Ed.*, 2009
[25] Travis Malone, A. David Parr, Ph.D., *Bend Losses in Rectangular Culverts*, 2008. University of Kansas
[26] US Army Corps of Engineers (USACE), *HEC-RAS Hydraulic Reference Manual*, 2016
[27] E. John Finnemore, Joseph B. Franzini, *Fluid Mechanics with Engineering Applications, 10th Ed.*, 2011
[28] Federal Highway Administration (FWHA), *Hydraulic Design of Highway Culverts, 3rd Ed.*, 2012

6.2.3.4 Volume Methods

As discussed in the Hydrologic Study section, actual precipitation amounts of the Hurricane Ida storm event were recorded through multiple mediums. Among them, the rain gauge station at the Twinbrook Community Recreational Center proved to have best captured the event. The gauge was in very close proximity to the Rock Creek Woods Apartments, operated without error, and recorded precipitation in 5-minute increments.

For the synthetic categorical storms (e.g., the 10-year, 25-year, etc.), cumulative rainfall values were sourced from NOAA Atlas 14 Point Precipitation Frequency Estimates (NOAA-14). These estimates are based on frequency analysis of partial duration series and are widely used amongst water engineers and scientists. NOAA-14 provides a table of banded precipitation estimates (upper and lower bounds of the 90% confidence interval) for different combinations of storm recurrence intervals and durations: from the 1-year 5-minute storm up to the 1000-year 60-day storm. The mid values of the bands were used, as shown in Figure 125.

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)[1]										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**0.347** (0.314-0.382)	**0.415** (0.376-0.457)	**0.493** (0.447-0.544)	**0.552** (0.498-0.608)	**0.625** (0.560-0.689)	**0.680** (0.606-0.750)	**0.734** (0.651-0.812)	**0.786** (0.692-0.873)	**0.854** (0.744-0.952)	**0.906** (0.784-1.01)
10-min	**0.554** (0.502-0.610)	**0.663** (0.601-0.730)	**0.790** (0.715-0.872)	**0.882** (0.797-0.972)	**0.996** (0.893-1.10)	**1.08** (0.965-1.20)	**1.17** (1.03-1.29)	**1.25** (1.10-1.38)	**1.35** (1.18-1.51)	**1.43** (1.23-1.60)
15-min	**0.692** (0.627-0.763)	**0.834** (0.755-0.918)	**1.00** (0.905-1.10)	**1.12** (1.01-1.23)	**1.26** (1.13-1.39)	**1.37** (1.22-1.51)	**1.48** (1.31-1.63)	**1.57** (1.39-1.75)	**1.70** (1.48-1.90)	**1.79** (1.55-2.01)
30-min	**0.949** (0.860-1.05)	**1.15** (1.04-1.27)	**1.42** (1.29-1.57)	**1.62** (1.46-1.78)	**1.87** (1.68-2.06)	**2.07** (1.84-2.28)	**2.26** (2.00-2.50)	**2.45** (2.16-2.72)	**2.71** (2.36-3.02)	**2.90** (2.51-3.25)
60-min	**1.18** (1.07-1.30)	**1.45** (1.31-1.59)	**1.82** (1.65-2.01)	**2.11** (1.90-2.32)	**2.49** (2.23-2.75)	**2.80** (2.49-3.09)	**3.11** (2.76-3.44)	**3.43** (3.02-3.81)	**3.88** (3.38-4.33)	**4.24** (3.66-4.75)
2-hr	**1.40** (1.27-1.54)	**1.71** (1.55-1.88)	**2.16** (1.96-2.38)	**2.52** (2.27-2.77)	**3.02** (2.72-3.32)	**3.44** (3.07-3.79)	**3.88** (3.43-4.28)	**4.34** (3.81-4.80)	**5.00** (4.34-5.56)	**5.54** (4.76-6.20)
3-hr	**1.50** (1.37-1.67)	**1.83** (1.66-2.03)	**2.32** (2.10-2.57)	**2.71** (2.44-2.99)	**3.28** (2.93-3.61)	**3.74** (3.33-4.13)	**4.24** (3.73-4.68)	**4.76** (4.16-5.28)	**5.53** (4.76-6.16)	**6.17** (5.24-6.91)
6-hr	**1.86** (1.69-2.07)	**2.25** (2.05-2.50)	**2.85** (2.58-3.15)	**3.34** (3.00-3.69)	**4.06** (3.62-4.49)	**4.68** (4.14-5.18)	**5.35** (4.69-5.93)	**6.08** (5.28-6.77)	**7.17** (6.12-8.03)	**8.09** (6.80-9.11)
12-hr	**2.27** (2.04-2.55)	**2.74** (2.46-3.08)	**3.48** (3.12-3.90)	**4.11** (3.66-4.61)	**5.08** (4.48-5.68)	**5.93** (5.18-6.64)	**6.87** (5.93-7.72)	**7.93** (6.75-8.94)	**9.56** (7.96-10.8)	**11.0** (8.98-12.5)
24-hr	**2.61** (2.38-2.89)	**3.15** (2.88-3.50)	**4.05** (3.70-4.49)	**4.84** (4.40-5.35)	**6.06** (5.46-6.65)	**7.14** (6.38-7.81)	**8.36** (7.39-9.12)	**9.75** (8.52-10.6)	**11.9** (10.2-12.9)	**13.8** (11.7-14.9)
2-day	**3.03** (2.76-3.35)	**3.66** (3.34-4.06)	**4.69** (4.28-5.19)	**5.58** (5.06-6.16)	**6.91** (6.23-7.60)	**8.07** (7.23-8.86)	**9.36** (8.32-10.3)	**10.8** (9.51-11.8)	**13.0** (11.3-14.2)	**14.9** (12.7-16.3)
3-day	**3.20** (2.92-3.54)	**3.87** (3.54-4.29)	**4.95** (4.51-5.47)	**5.88** (5.34-6.49)	**7.27** (6.57-8.01)	**8.49** (7.62-9.33)	**9.84** (8.75-10.8)	**11.4** (10.0-12.4)	**13.6** (11.8-14.9)	**15.6** (13.4-17.1)
4-day	**3.37** (3.09-3.73)	**4.07** (3.73-4.51)	**5.20** (4.76-5.76)	**6.18** (5.63-6.82)	**7.64** (6.91-8.41)	**8.91** (8.00-9.80)	**10.3** (9.19-11.3)	**11.9** (10.5-13.1)	**14.3** (12.4-15.7)	**16.3** (14.0-17.9)
7-day	**3.90** (3.59-4.29)	**4.70** (4.32-5.17)	**5.94** (5.46-6.52)	**7.00** (6.41-7.67)	**8.60** (7.82-9.39)	**9.97** (9.01-10.9)	**11.5** (10.3-12.5)	**13.2** (11.7-14.4)	**15.7** (13.7-17.1)	**17.8** (15.4-19.5)
10-day	**4.46** (4.10-4.89)	**5.36** (4.93-5.87)	**6.69** (6.15-7.32)	**7.80** (7.15-8.53)	**9.43** (8.59-10.3)	**10.8** (9.78-11.8)	**12.3** (11.0-13.4)	**13.9** (12.4-15.1)	**16.2** (14.3-17.7)	**18.1** (15.9-19.8)
20-day	**6.03** (5.60-6.53)	**7.18** (6.66-7.76)	**8.67** (8.04-9.37)	**9.89** (9.16-10.7)	**11.6** (10.7-12.5)	**12.9** (11.9-14.0)	**14.4** (13.2-15.5)	**15.8** (14.4-17.1)	**17.9** (16.2-19.3)	**19.5** (17.5-21.1)
30-day	**7.43** (6.93-7.97)	**8.79** (8.20-9.44)	**10.5** (9.75-11.2)	**11.8** (11.0-12.6)	**13.6** (12.7-14.6)	**15.1** (14.0-16.2)	**16.6** (15.3-17.8)	**18.2** (16.7-19.5)	**20.3** (18.5-21.8)	**21.9** (19.9-23.6)
45-day	**9.33** (8.76-9.95)	**11.0** (10.3-11.7)	**12.9** (12.1-13.7)	**14.3** (13.4-15.2)	**16.2** (15.1-17.2)	**17.6** (16.5-18.7)	**19.0** (17.7-20.2)	**20.4** (19.0-21.7)	**22.2** (20.5-23.7)	**23.5** (21.7-25.1)
60-day	**11.1** (10.5-11.8)	**13.1** (12.3-13.9)	**15.1** (14.2-16.0)	**16.6** (15.7-17.6)	**18.6** (17.5-19.7)	**20.1** (18.9-21.3)	**21.5** (20.1-22.8)	**22.9** (21.3-24.3)	**24.6** (22.9-26.1)	**25.8** (23.9-27.5)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Figure 125 - NOAA Atlas 14 Precipitation Estimates at the Rock Creek Woods Apartments

In modeling synthetic storms, precipitation-mass curves (unitless temporal distributions) are used to vary the rainfall over time to produce simulated precipitation (hyetographs). Two different 24-hour distributions were used in the modeling effort: 1) The SCS Type II rainfall distribution, and 2) The site-specific NOAA Atlas 14 distribution – which in this case references the regional distribution for the Ohio River Basin that covers Maryland. Figure 126 below shows a comparative plot of the two mass curves.

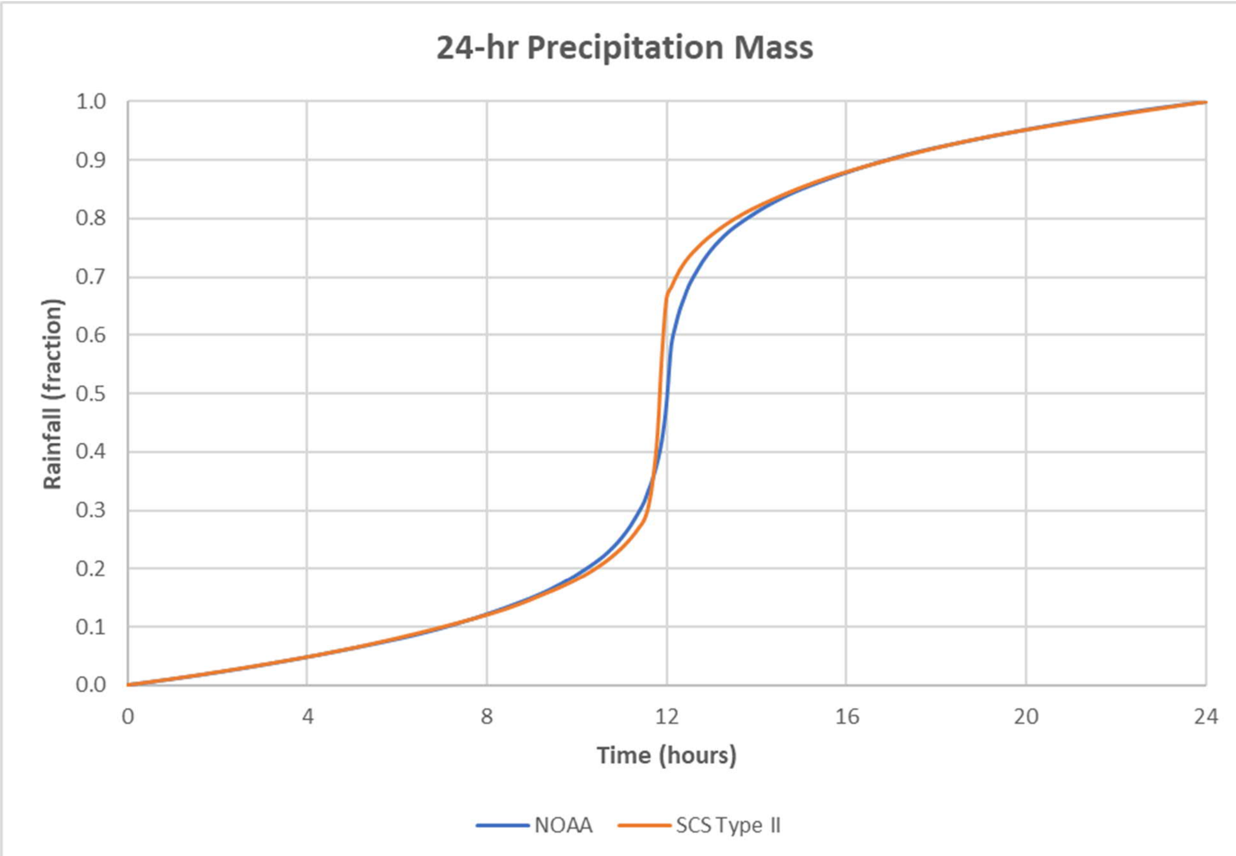


Figure 126 - 24hr Precipitation-Mass Curves

The SCS temporal distributions were developed in the early 1970s and were effectively used as the industry standard until *NOAA Atlas 14 Volume 2* was published. This led to the development of newer regional distributions (Merkel and others, 2006). Considering the prevalence of the SCS distributions in level of service analyses historically and still today, it was determined that modeling both the SCS and NOAA distributions was appropriate.

In comparing the two distributions, it's important to note that the NOAA precipitation-mass is more widely distributed over time, producing a lesser rainfall intensity than the SCS distribution. This is shown in Figure 127 below.[29]

[29] Both distributions peak at 12 hours. The slight discrepancy in the chart is due to significant figures and relatively small time increments.

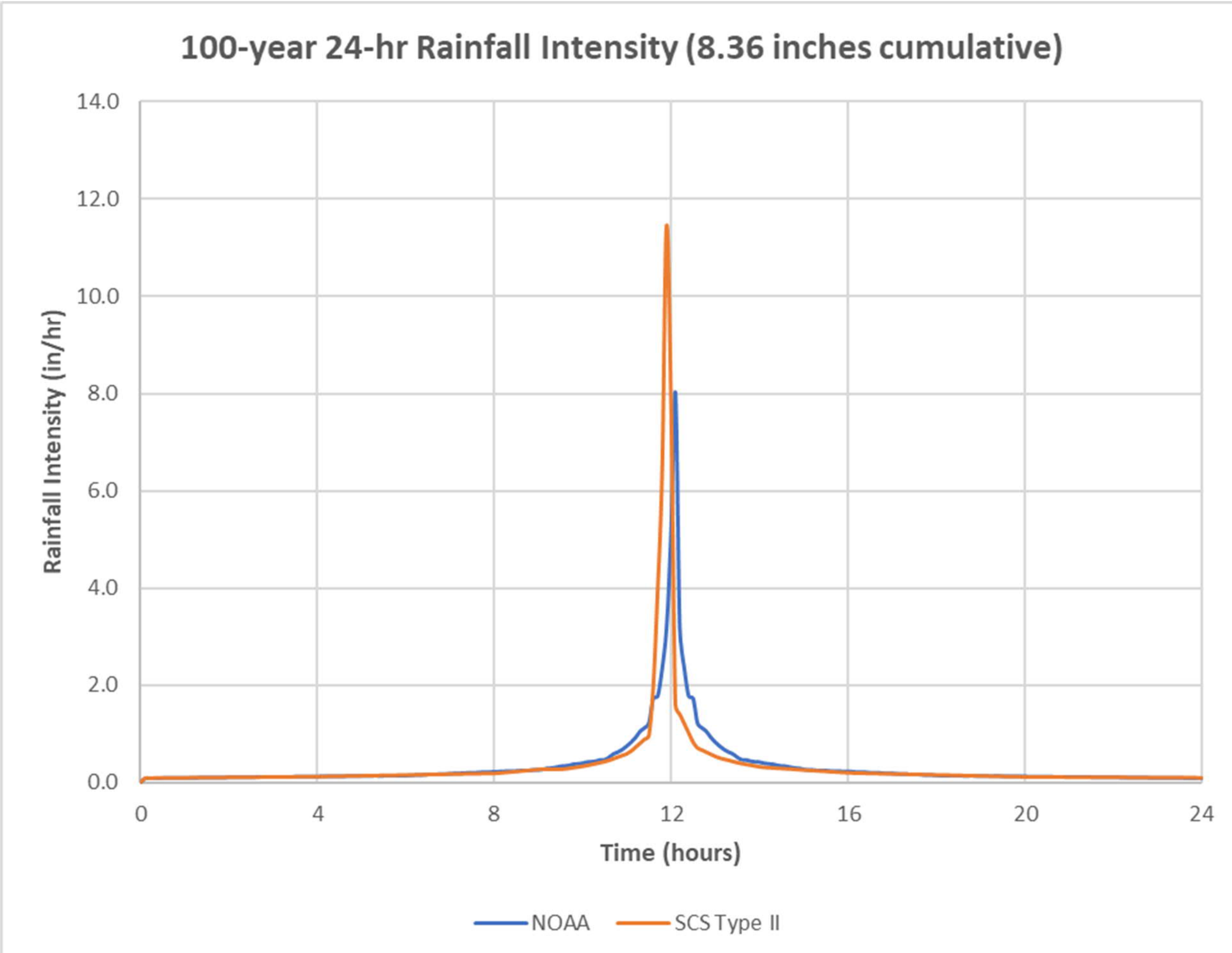


Figure 127 – 100yr-24hr Rainfall Intensity of Synthetic Distributions

To calculate rainfall excess (runoff) volumes, the methodology outlined in Technical Release 55 (TR-55) - *Urban Hydrology for Small Watersheds* (NRCS) was used for both the 1D and 1D/2D H&H models. This method utilizes runoff curve numbers (CNs) which are representative of the watershed's ability absorb rainfall. For more information on the development of curve numbers, reference chapters 4 through 10 of NEH-4 (SCS, 1985).

As noted previously, ICPR4 is unique in its application of the curve number method. Traditionally, a single composite curve number would be calculated for each drainage basin to be representative of all of the various land cover and soil types throughout each basin. ICPR, however, uses what is called a "distributed approach" to this traditional method. Instead, drainage basins are discretized into multiple sub-basins; one for each unique land cover & soil combination within a basin. Rainfall excess is calculated for each sub-basin individually and then added together before applying the time of concentration and unit hydrograph, altogether producing the runoff hydrograph for the lumped basin. In this way, the distributed approach produces more realistic levels of runoff for a given storm than the traditional lumped method would. The final composite curve number for each basin may still be inferred at the back-end of the hydrology calculations.

The following runoff curve numbers in Table 9 were used.

Table 9 - Runoff Curve Numbers

	Hydrologic Soil Group							
Cover Type	**A**	**B**	**C**	**D**	**A/D**	**B/D**	**C/D**	**Selection Basis**
Bare Soil	77	86	91	94	85.5	90	92.5	Fallow - Bare Soil
Building	98	98	98	98	98	98	98	N/A
Creek Bank Boulder	100	100	100	100	100	100	100	N/A
Creek Bank Concrete	100	100	100	100	100	100	100	N/A
Creek Bank Grass/Shrub	95	95	95	95	95	95	95	N/A
Creek Bank Riprap	100	100	100	100	100	100	100	N/A
Creek Bank Stone	100	100	100	100	100	100	100	N/A
Creek Bed Boulder	100	100	100	100	100	100	100	N/A
Creek Bed Cobble	100	100	100	100	100	100	100	N/A
Creek Bed Concrete	100	100	100	100	100	100	100	N/A
Creek Bed Mixed Cobble/Boulder	100	100	100	100	100	100	100	N/A
Creek Bed Sand	100	100	100	100	100	100	100	N/A
Forest	36	60	73	79	57.5	69.5	76	Woods - fair
Grass/Shrub*	49	69	79	84	66.5	76.5	81.5	Lawns, open space - Poor
Other Paved	98	98	98	98	98	98	98	Paved with curbs
Railroad	76	85	89	91	83.5	88	90	Gravel
Road	98	98	98	98	98	98	98	Streets and Roads
Tree Canopy*	57	73	82	86	71.5	79.5	84	Woods grass comb. - Poor
Water	100	100	100	100	100	100	100	N/A
Wetland	95	95	95	95	95	95	95	N/A

** Adjusted in calibration*

6.2.3.5 Routing Methods

The SCS Unit Hydrograph method was applied in the 1D model to temporally route the calculated runoff of each delineated subcatchment to a corresponding model node. The SCS method uses a Dimensionless Unit Hydrograph (DUH)[30] to produce a synthetic runoff hydrograph after applying the rainfall amount. The DUH, which is derived from the analysis of large number of watersheds varying in size and location, has the inherent assumption that 3/8 of the total runoff volume (37.5%) discharges before the peak discharge and the remaining 5/8 of the runoff volume (62.5%) discharges after the peak.

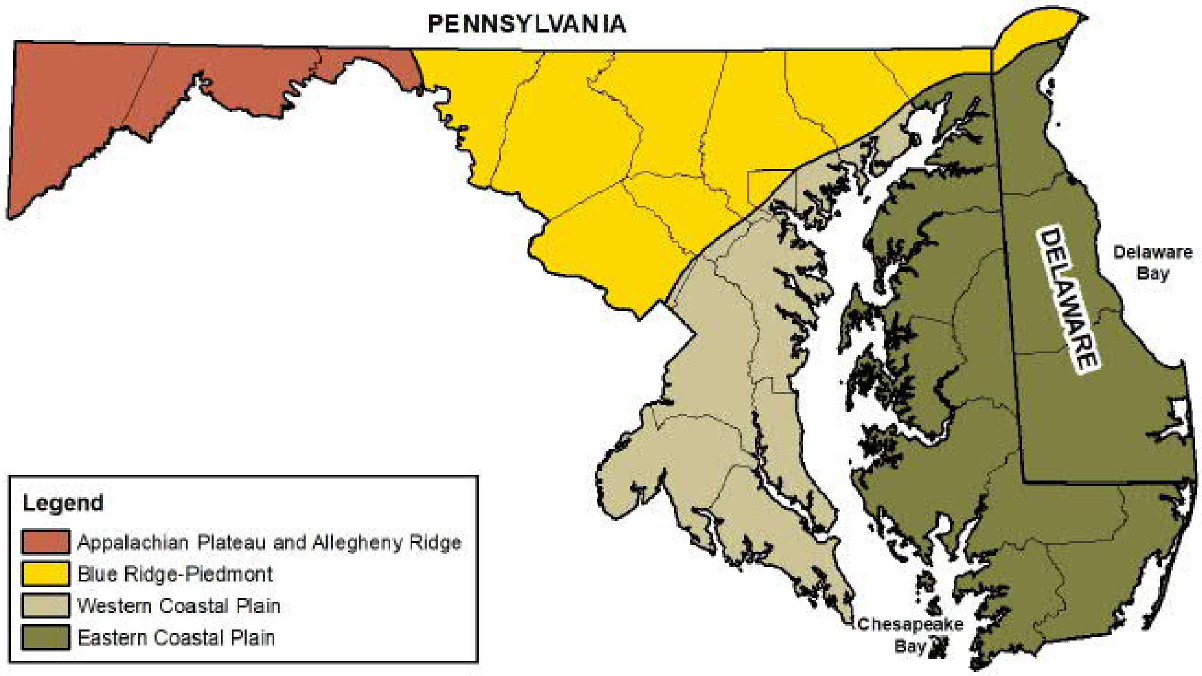
[30] Victor Mockus, Soil Conservation Service, *National Engineering Handbook, Section 4, Hydrology*, 1972

The runoff peak discharge produced from the DUH takes into account the basin drainage area, the generated runoff volume, the time to peak (calculated from the storm duration and basin time of concentration), and a peaking factor which may vary by geography.

In combination with the previously described overland roughness coefficients, the time of concentration for each delineated subcatchment was calculated using ArcGIS and the procedural methods outlined in TR-55. The minimum time of concentration permitted was 6 minutes.

As for the peaking factor, it should be noted that hydrodynamic models can be quite sensitive for this component of the SCS method. Thus, appropriate selection of a peaking factor is an important consideration for model development. In review of hydrologic literature, it was determined that a peaking factor of 484 was appropriate for the Rockville area[31], which lies in the Blue Ridge - Piedmont Plateau region of Maryland (see Figure 128).

[31] Maryland Hydrology Panel, *Application of Hydrologic Methods in Maryland, 5th Ed.*, 2020



REGION	PEAK FACTOR
Eastern Coastal Plain	284
Western Coastal Plain	284 or 484
Piedmont	484
Blue Ridge	484
Appalachian	484

Figure 128 - Unit Hydrograph Peaking Factors, from Maryland Hydrology Panel Report

6.2.4 Physical Model Construction

With the previously described inputs and methods in mind, an ICPR4 model is functionally a construction of connected links, nodes, and cross-sections.

The construction of the 1D model for this study consisted of the following.

- Link Types:
  - Channel – describes <u>the span between</u> trapezoidal, parabolic, or irregular sections.
  - Pipe – describes various closed-conduits including circular / rectangular / elliptical / arched sections.
  - Weir or Drop Structure – used for enabling weir flow calculations across physical obstructions or through orifices (vertical or horizontal).
  - Percolation – enables groundwater exfiltration as described by numerous geotechnical factors.

- Node Types:
  - Stage/Area – used for manholes, inlets, and junctions.
    - Stage-area tables were used at inlets to calculate above-ground storage during surcharge.
    - 1D subcatchment discharges were assigned to associated nodes.
  - Stage/Volume – used for representing BMPs.
    - Stage-volume tables were used to describe available storage of a particular BMP.
    - 1D subcatchment discharges were assigned to associated nodes.
  - Time/Stage – used for the boundary node (only one in this model)
    - A time-stage table was used to enforce the boundary stage of Rock Creek
- Cross Section Types:
  - Channel – used to describe the section of irregular-shaped channels.
    - Includes Manning's n roughness coefficients.
    - Uses an algorithm equivalent to that of HEC-RAS to calculate overbank flows.
  - Weir – used to describe irregular-shape obstructions such as roadways or berms.

Weir links can be used at inlet locations to accurately model inlet efficiency and inflows. However, this level of detail can only be achieved with survey-grade data. Therefore, inlets were modeled this way only within the Rock Creek Woods Apartments area that was surveyed by Mercado Consultants. Outside of this area, inlets were assumed to be unobstructed, though some head loss was assigned for flow through the openings/grates. Figure 129 shows the 1D model link-nodal representation of hydraulic assets at the Apartments.

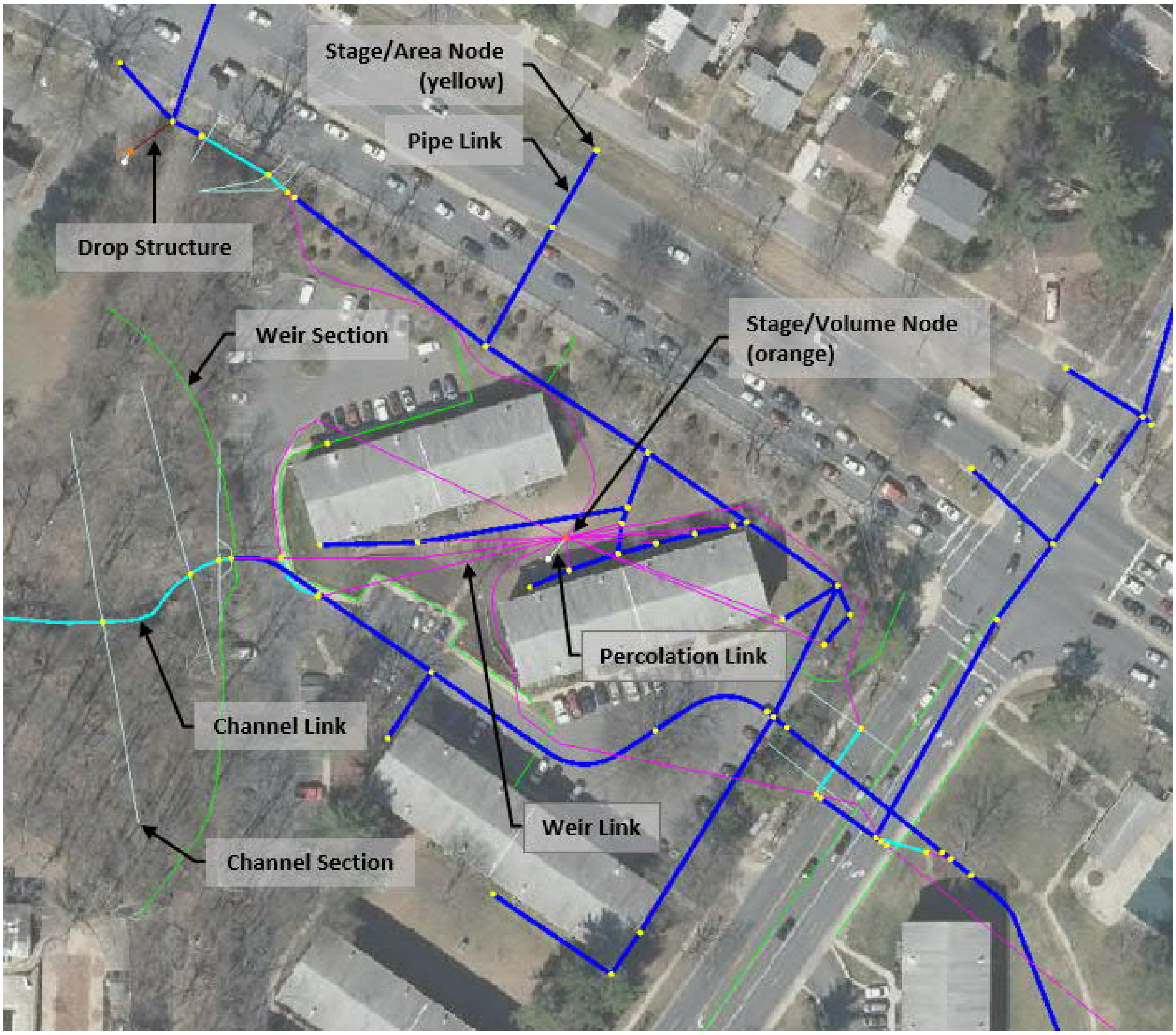


Figure 129 - 1D Model Construction Example

The 2D model retained much of the physical construction from the 1D model. However, the complex scheme of inlet weir links to a singular storage node, as shown in Figure 129, was not required for the 2D model. Instead, “interface nodes” were placed at inlets within the 2D overland flow region, and thus the inlets were coupled directly to the triangular mesh at geographically precise locations.

The overland flow regions of the 2D model use a similar scheme of links and nodes for the overland flow mesh. Figure 130 below displays the ICPR4 generated triangular mesh (used for routing) and underlaying honeycomb mesh (used for volume).

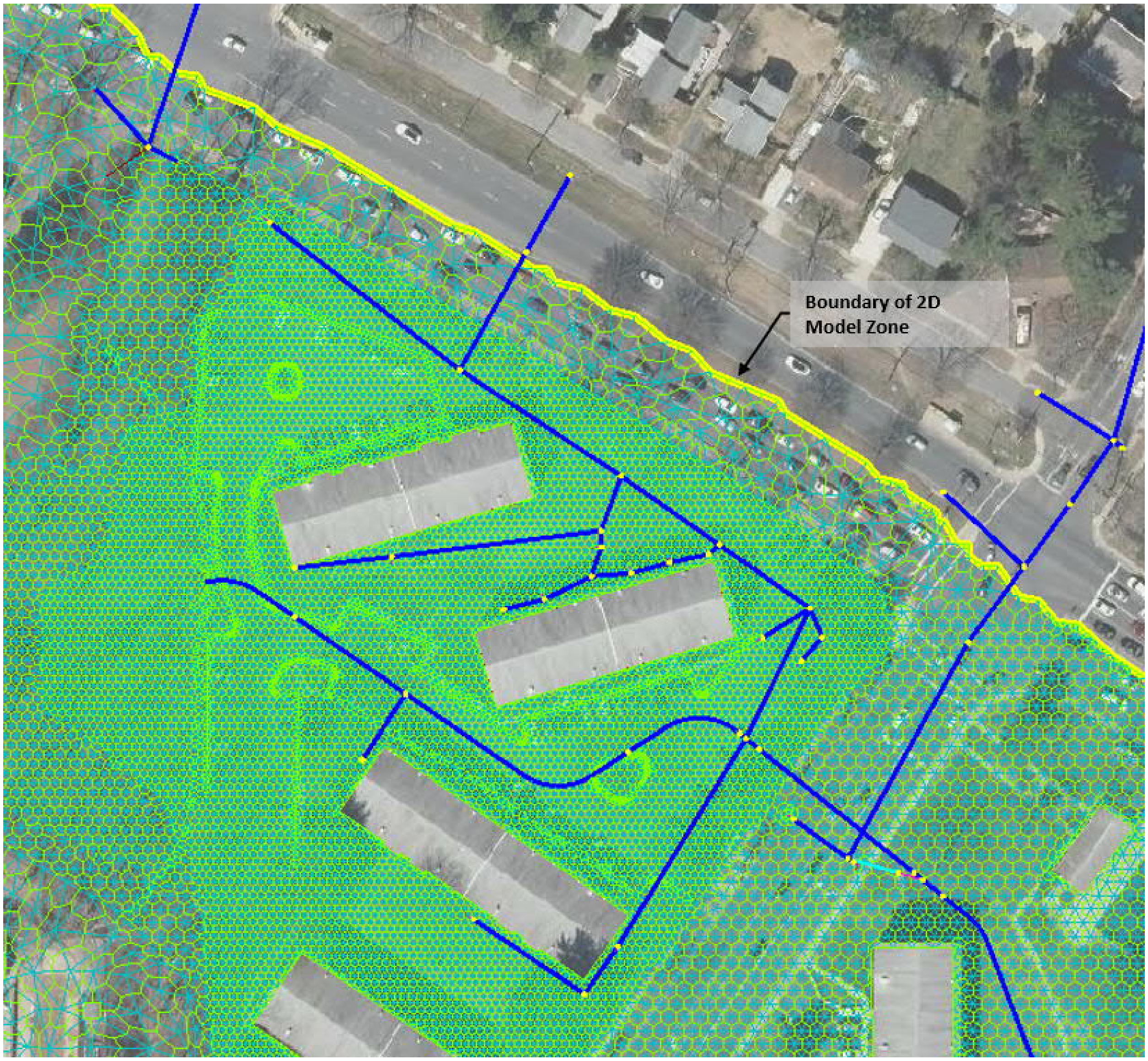


Figure 130 - 2D Model Triangular Mesh (light blue) & underlaying Honeycomb Mesh (green)

6.2.5 Model Calibration

In order to ensure the H&H models provide a reliable and realistic simulation of the Hurricane Ida storm event, and other potential storm events, several calibration targets were derived from observed field conditions of the Rock Creek Woods Apartments, and immediate areas.

- Highwater elevation markings on Apartment Buildings 7 and 8 (measured);
- Highwater elevation markings of disturbed vegetation along the channel downstream of the box culverts (approximated through documentation);
- Video documented overtopping of the Tributary 2 culvert's upstream headwall and berm, subsequently flooding Buildings 7 and 8;
- Photographic evidence of overtopping at the Atlantic Avenue box culvert (Tributary 2);
- Witness account of overtopping at the Twinbrook Community Recreational Center's access road culvert (Tributary 1, between Atlantic Avenue and Vandegrift Avenue);
- Witness account of overtopping at the two-barrel culvert just upstream of Rock Creek (furthest downstream culvert, adjacent to Veirs Mill Road); and
- Reported calls for help from residents at approximately 3:50 am.

The primary target(s) for calibration were the highwater elevation markings measured at Buildings 7 and 8, where severe residential inundation occurred. Table 10 and Table 11 below show the exact measurements (NAVD88 datum) taken at each inundated building.

Table 10 - Building 7 Highwater Elevations

Building 7 (East)	**Measure 1**	**Measure 2**	**Measure 3**	**Average**
High Water Line	286.19	286.09	286.12	286.13
Patio Elevation	278.34	278.17	278.26	
Water Depth (ft)	7.85	7.92	7.86	7.88

Table 11 - Building 8 Highwater Elevations

Building 8 (West)	**Measure 1**	**Measure 2**	**Average**
High Water Line	286.18	286.17	286.18
Patio Elevation	281.28	281.03	
Water Depth (ft)	4.9	5.14	5.02

Some degree of inaccuracy in modeling is to be expected, thus a margin of ±4 inches from the high (286.19) and low (286.09) was used to establish a band of acceptable flood elevations produced by the models. This would account for any probable "splashing" effects by the turbulence of the flood waters during the event.

The two buildings are generally in the same location from a 1D model standpoint, thus an average of all highwater elevations, 286.15, was taken as the primary calibration target, with an acceptable minimum highwater of 285.76 and maximum of 286.52.

Ultimately, the H&H models required little calibration. The following model parameters were adjusted:

- The Manning's "n" roughness coefficients for stream cross-sections along each tributary, and the downstream portion leading to Rock Creek, were adjusted for deep flow (halved). Time of concentration calculations still used the shallow (normal) Manning's *n* roughness coefficients, though they had an acceptable minimum Tc of 6 minutes. Many of the 1D delineated subcatchments were small enough that the calculated Tc was much smaller than the minimum. As such, some artificial attenuation of runoff discharges was expected. Adjusting the stream sections' surface Manning's *n* coefficient for deep flow thus recovered some of that attenuation.[32]
- The "Tree Canopy" and "Grass/Shrub" land cover class Curve Numbers were adjusted for "poor" conditions (from "fair"), as categorized by TR-55. This was done per suggestions described in a 2020 study by the Maryland Hydrology Panel[31]. In their own calibration efforts, the Panel noted that when using runoff curve numbers based on a "good" hydrologic condition, small urban watersheds with predominantly Type A or B soils may generate peak discharges well below the range calculated by the Fixed Region Regression Equations. Thus, the Panel suggested to use RCN values for urban land that are derived using 'fair' or 'poor' hydrologic conditions rather than 'good' to raise the discharges within expected range.

In adjusting these few parameters, the models were quickly brought into a reasonable range of all calibration targets. With special regard to the primary calibration target, the 1D model produced a maximum flood stage (at the inundated area of the Rock Crest Woods Apartments) of 286.12. The 1D/2D model reached an average maximum stage of 286.15. Both models produced maximum flood stages within 1-inch of the measured average, well within the acceptable band of accuracy. Figure 131 below exhibits the recorded 1D model stage during the Hurricane Ida simulation, extracted directly from ICPR4. The simulation results shown starts at 12:00am on the morning of September 1, 2021. Further details of the modeling results, as they relate to the other calibration targets, are discussed in the following section of this report.

[32] It is a well-known phenomenon that flow velocities increase as the distance from the wetted perimeter increases. This is due to a decrease in friction as particles of water begin to move across themselves in deepening flow. Given the particularly short duration and high intensity of the Hurricane Ida event, significant runoff and deep flows were to be expected.

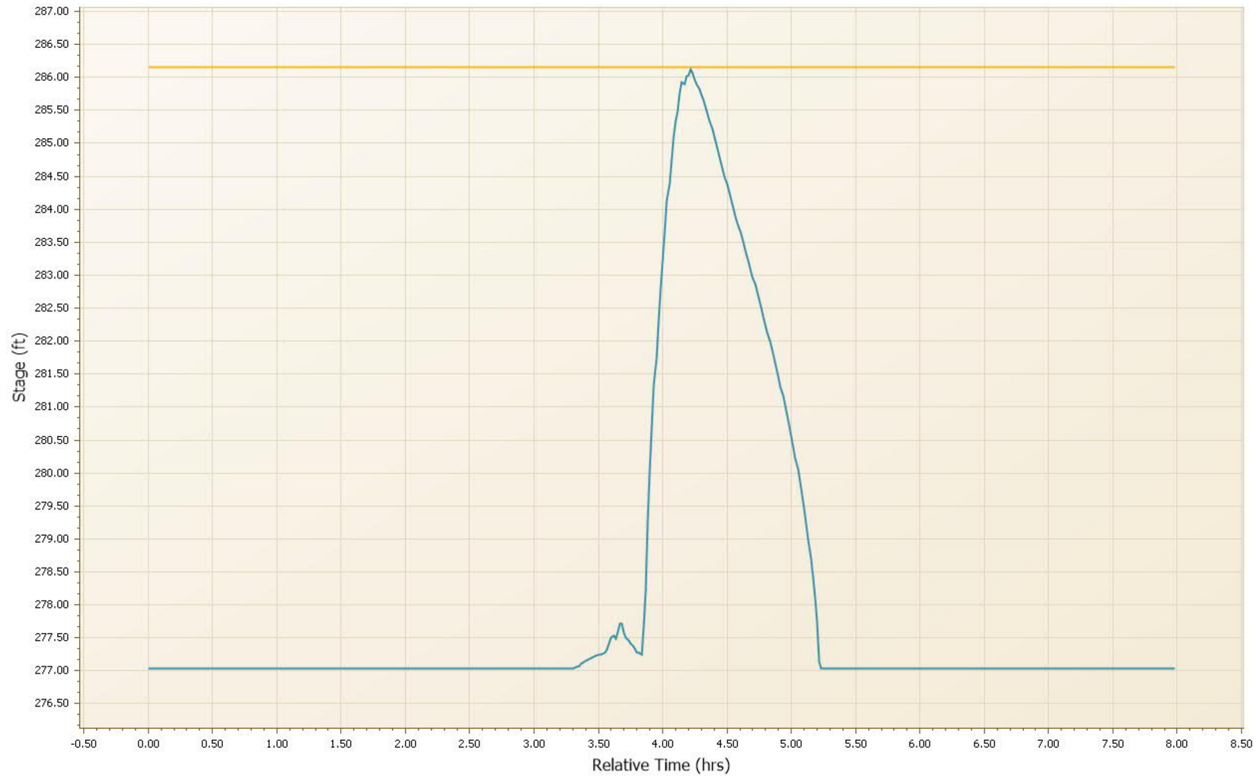


Figure 131 – Hurricane Ida 1D Flood Stage at Rock Creek Woods Apartments[33]

6.3 H&H Modeling Results & Analysis

6.3.1 Hurricane Ida Simulation

The ICPR4 model simulation of Hurricane Ida revealed complex hydrodynamics at play across the approximately 840-foot-long northern culvert of Tributary 2, and to a lesser extent through the southern culvert of Tributary 1. In summary, the culverts were subjected to multiple flow regimes and hydraulic jumps as the storm progressed. This in combination with sustained very high-intensity rainfall contributed to surcharge flooding at hydraulically connected inlet locations, and subsequent overtopping of the headwall and berm upstream of the northern culvert. In specific regard to this culvert, additional compounding factors are:

- Head losses due to:
  - An 8-foot long steep (18.9%) drop at the concrete apron leading to the culvert;
  - Significant flow area reduction at the upstream headwall;

[33] Photographic and video evidence show that high standing water was present in the residential area during the daytime following Hurricane Ida's passage. This suggests that at some point(s) during the event, debris blocked the relatively small ditch bottom inlets throughout the lawn adjacent to the apartments. The inlets serve 4- to 6-inch PVC pipes, which may have also clogged with debris and/or sediment. The model result above shows the tail of the stage curve receding within a matter of hours. This reflects a modeled condition of these inlets and pipes functioning as intended, without blockage, and some groundwater infiltration.

- o Friction of the wetted concrete surfaces;
  - o Five (5) Horizontal bends mid-span of the conduit, varying in degree of deflection (up to 72°);
  - o A 16-foot long transitional expansion (from 10-foot x 5-foot) to accommodate the original 14-foot x 7.5-foot section crossing Twinbrook Parkway;
    - ▪ Simultaneously, a steep (18.1%) flowline drop to match the existing invert;
  - o Establishment of a shallower conduit slope beginning at Twinbrook Parkway (from 3.7% avg. to 1.5% avg.);
  - o Downstream of Twinbrook Parkway, an <u>abrupt</u> height contraction back to 5 feet (from 7.5 feet);
    - ▪ Simultaneously, a 24-foot long transitional width contraction back to 10 feet (from 14 feet);
  - o At the terminal end of the culvert, an 8-foot long steep (15.2%) drop to the open channel;
  - o Significant flow area expansion at the outlet;
  - o Minor sedimentation along the entire flow line (no debris obstructions were observed, or expected); and
  - o The baffle system 20 feet downstream of the outlet.
    - ▪ There are three ~2-foot tall baffles (energy dissipators) just downstream of the culverts, intended to reduce flow velocities at the outlet. This expectedly creates higher upstream stages in compensation for the loss of energy. However, for this particular storm event the baffles do not hold enough influence on upstream stages to have prevented overtopping in their absence, as confirmed through modeling.
- Several <u>direct</u> storm drain discharges (break-in taps) into the box culvert sidewalls.
  - o Contributing drainage from Veirs Mill Road (18-inch diameter) and Twinbrook Parkway (bricked section, approx. 30-inch equivalent diameter);
- At the northwest corner (~8 feet outside of the property line), a 30-inch diameter outfall discharging (West to East) to a relatively small open channel.
  - o After overcoming a short length of adverse slope, flow will proceed eastward into the property towards a small vertically-faced inlet (~65 feet downstream of the outfall, with a trapezoidal opening of 1-foot height, 2-foot bottom width, 4-foot top width)

6.3.1.1 North Culvert (Tributary 2)

The following paragraphs describe the approximate timeline of events at the northern box culvert, as they occurred on September 1, 2021 (from ~2:00 am to ~4:30 am). This sequence of events was deduced from review of the H&H model results by hydraulic engineers. However, given the lack of measured flow data, these events should be taken as approximate only. The timestamps and/or stages quoted are derived from the 2D model simulation and are independent of any witness accounts or field-measured data previously noted in this report or elsewhere.

For visual reference, Figure 132 through Figure 138 exhibit the 2D modeled overland flow depth at critical time steps.

To begin with, the first hour of the storm event (2:00 – 3:00 am) consisted of light rainfall of less than 0.5 inches per hour in intensity. Two rectangular slots between the tiered baffles 20 feet downstream of the culvert outlet permitted passage of low initial runoff flows without congestion. These slots eventually became submerged at ~3:21 am, from which time sharp-crested weir flow took over the baffles. This transition from steady channel flow to weir flow occurred relatively quickly, over about a 10-minute period, as higher stream flows began to reach the culvert.

Concurrently, discharges from the 30-inch outfall at the northwest corner of the Rock Creek Woods Apartment property were flowing into the northwest (NW) lawn[34]. The discharge had started to bypass the nearest downstream inlet at ~3:18 am, and by ~3:25 the overland flow had reached the ditch bottom inlets adjacent to Building 7. Drainage capacity there was limited due to the relatively small 4- to 6-inch PVC pipes serving the inlets. As drainage capacity was exceeded, water began to quickly pool at this location.

Prior to submergence of the outlet, a subcritical[35] open channel flow regime persisted through the culvert's downstream portions, due to the shallower slope east of Twinbrook Parkway as well as that of the downstream channel. The reduction of flow velocity at the outlet baffles further contributed to this flow regime. On the other end, the steeper conduit slope west of Twinbrook Parkway permitted supercritical[36] flows through upstream portions of the culvert. While increasing flow rates filled the downstream capacity of the culvert a hydraulic jump continuously propagated upstream, as required for energy to be conserved through transitions from supercritical to subcritical open channel flow.

Moreover, the generally abrupt changes in conduit alignment, slope, and sectional geometry potentially contributed to a series of additional hydraulic jumps, transient in nature. In particular, the locations of noted steep drops would have been vulnerable to this effect. Additional head losses associated with swirling eddies likely resulted from the turbulence of any such hydraulic jumps and from the numerous changes in flow direction.

[34] "NW lawn" refers to the grassy area adjacent to the northern-most apartments west of Twinbrook Parkway, Buildings 7 and 8, where the primary inundation occurred.

[35] Subcritical flow is characterized as tranquil or upper-stage flow. It occurs when the actual water depth is greater than the critical depth (at which the specific energy is at minimum for a given section and flow rate).

[36] Supercritical flow is characterized as rapid or lower-stage flow. It occurs when the actual water depth is less than the critical depth (at which the specific energy is at minimum for a given section and flow rate).

At ~3:31 am the outlet of the culvert became fully submerged, and only 4 minutes later, ~3:35, the ditch bottom inlet residing in the northeast (NE) lawn[37] began to surcharge. This location is approximately 70 feet upstream of the outlet.

Before reaching peak discharge, Tributary 2 flows submerged the upstream opening of the culvert at ~3:38 am. Air pockets likely remained within the culvert upstream of any hydraulic jumps as they persisted, until the culvert became completely pressurized shortly thereafter. At ~3:41, rising pressure began to surcharge the curbside inlet near Building 8 (West). Within the next minute, this surcharge crested the adjacent sidewalk and flowed into the NW lawn, entering the ground floor of Building 8.

A sharp rise in stage at the culvert entrance was observed in the following minutes, and at ~3:44 am the upstream headwall and berm was overtopped. Tributary 2 flow immediately crossed the adjacent roadway and sidewalk, flowing onto the NW lawn. This point in time marks the confluence of flood waters from three distinct point-sources into this area, as well as the end of precipitation from the primary Hurricane Ida storm-cell. Rainfall intensities across the drainage area remained near zero until 2:00 pm in the afternoon following. However, overland flows lagged across each tributary and were still accumulating towards peak discharge.

Over the next 14 minutes, the NW lawn began to rapidly stage higher as no drainage was readily available. The previously mentioned lawn inlets and associated PVC pipes had already reached capacity, and the lowest overland pour point (at the northeast corner, near the intersection of Twinbrook Parkway & Viers Mill Road) was at an elevation of 286.15 [38], approximately 9 feet above the low point of the lawn (276.97). At ~3:58 am, the H&H model reported an average peak stage of 286.15 within the NW lawn, which agrees with the surveyed highwater marks in this area. This coincides with the peak discharges reported at the culvert entrance, of which the maximum was ~931 cubic feet per second. The peak discharge and peak stages can be viewed in Figure 141 and Figure 142, respectively.[39]

At ~4:00 am overland bypass of the inlet at the northwest corner had generally ceased. In total, the bypass lasted ~42 minutes. Lagging overland sheet flow along the north edge of the property continued toward the NW lawn for another 5 to 10 minutes.

At ~4:24 am overtopping of the upstream berm ceased; in total lasting ~40 minutes.

[37] “NE lawn” refers to the grassy area adjacent to the northern-most apartments east of Twinbrook Parkway
[38] Terrain elevations were extracted from the Mercado Consultants survey-generated TIN surface, in the NAVD88 datum.
[39] It should be noted that the modeled highwater is approximate. Overflow from the NW lawn eastward into Twinbrook Parkway cannot be precluded, nor confirmed, from these results alone. There is no documented evidence of flood waters crossing Twinbrook Parkway. However, the model suggests any such flood water would have been minimal and shallow in depth, and likely fully captured by the curb inlets at the sag of the roadway.

Finally, at ~4:30 am surcharge from the curbside inlet near Building 8 retraced back below the top of the adjacent sidewalk and was fully drained from the gutter line by ~4:34. In total, the surcharge lasted ~53 minutes.

This concludes the relevant modeled timeline of the Hurricane Ida simulation. From this point on, a standing pool of water remained in the NW lawn despite receding stream flows. This was evident by video and photographic documentation taken during the hours following the event. Contrarily, the H&H model indicated a drained condition about one hour after the peak stage was recorded. This result suggests that the lawn inlets and pipes likely became obstructed by debris and/or sedimentation. Some exfiltration through the soil would have provided some drainage, though this may have been limited by saturation.

6.3.1.2 South Culvert (Tributary 1)

The southern box culvert serving Tributary 1 was able to pass Hurricane Ida flows without any overtopping of its upstream headwall and berm. In fact, the 10-foot x 8.5-foot section crossing Twinbrook Parkway never reached full flow capacity, although the downstream 10-foot x 5-foot section did reach full flow capacity.

On average, the southern culvert's slope is steeper and more uniform than the northern culvert, which affords it a higher flow capacity. Furthermore, there are only three bends in this culvert (two of which have relatively small deflection angles), and only two steep drops that bound the larger section across Twinbrook Parkway. This is more efficient compared to the five bends and three steep drops of the northern culvert. Most importantly, the drainage area of Tributary 1 is 40% smaller than that of Tributary 2 (252 acres versus 427 acres, respectively).

Altogether, flows through the southern culvert were generally lower and conveyed more efficiently than in the northern culvert. However, the downstream condition is still shared with the northern culvert. Because the steep conduit slope produces supercritical flows, a propagating hydraulic jump forms in this culvert as well. In general, this culvert is subject to head losses similar in manner to those of the northern culvert, though to a lesser extent. Some surcharge was indeed produced at the inlet located within the overlaying lawn east of Twinbrook Parkway, and south of the outlet.

6.3.1.3 2D Simulation Exhibits (Rock Creek Wood Apartments)

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Figure 132 - 2D Simulation Depth Map, 3:18 am.
Stormwater has overwhelmed the open channel at the Northwest Corner of the Rock Creek Woods Apartments Property and is flowing towards the low point between Buildings 7 and 8.

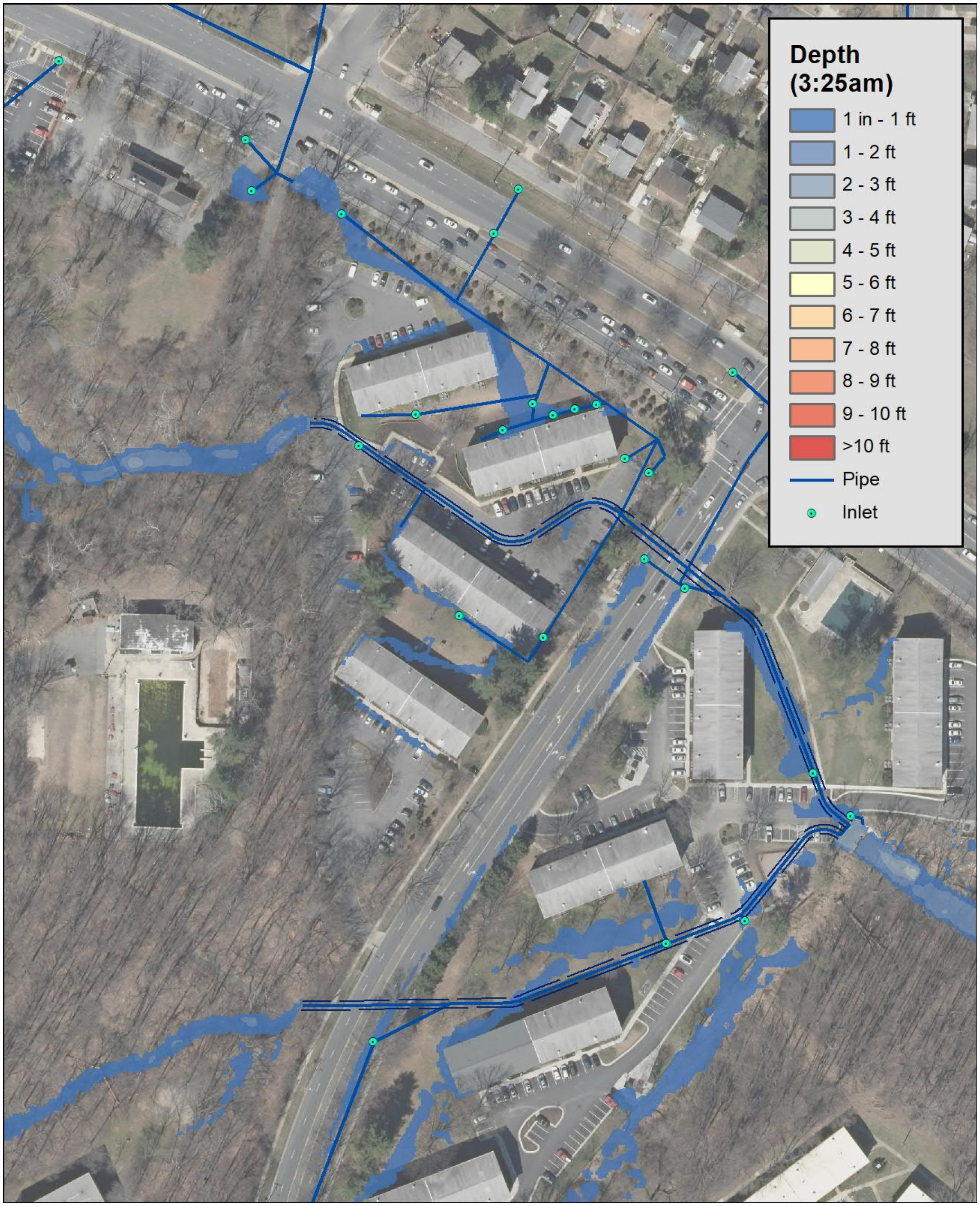

Figure 133 - 2D Simulation Depth Map, 3:25 am
Stormwater that has overwhelmed the open channel at the Northwest Corner of the Rock Creek Woods Apartments Property has reached the low point between Buildings 7 and 8.

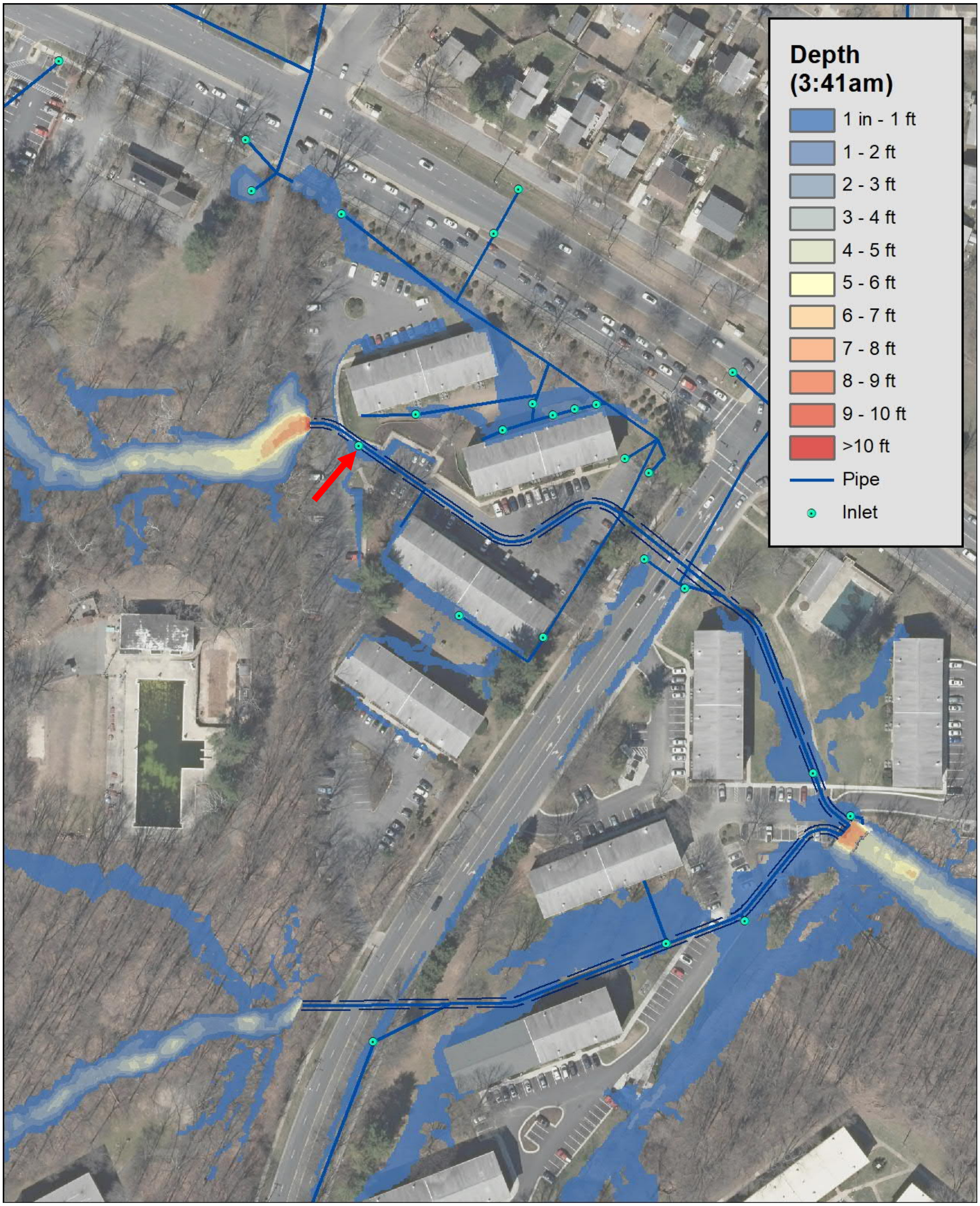

Figure 134 - 2D Simulation Depth Map, 3:41 am
Stormwater has started to surcharge out of the first inlet from the upstream end that is in the top slab of the culvert.

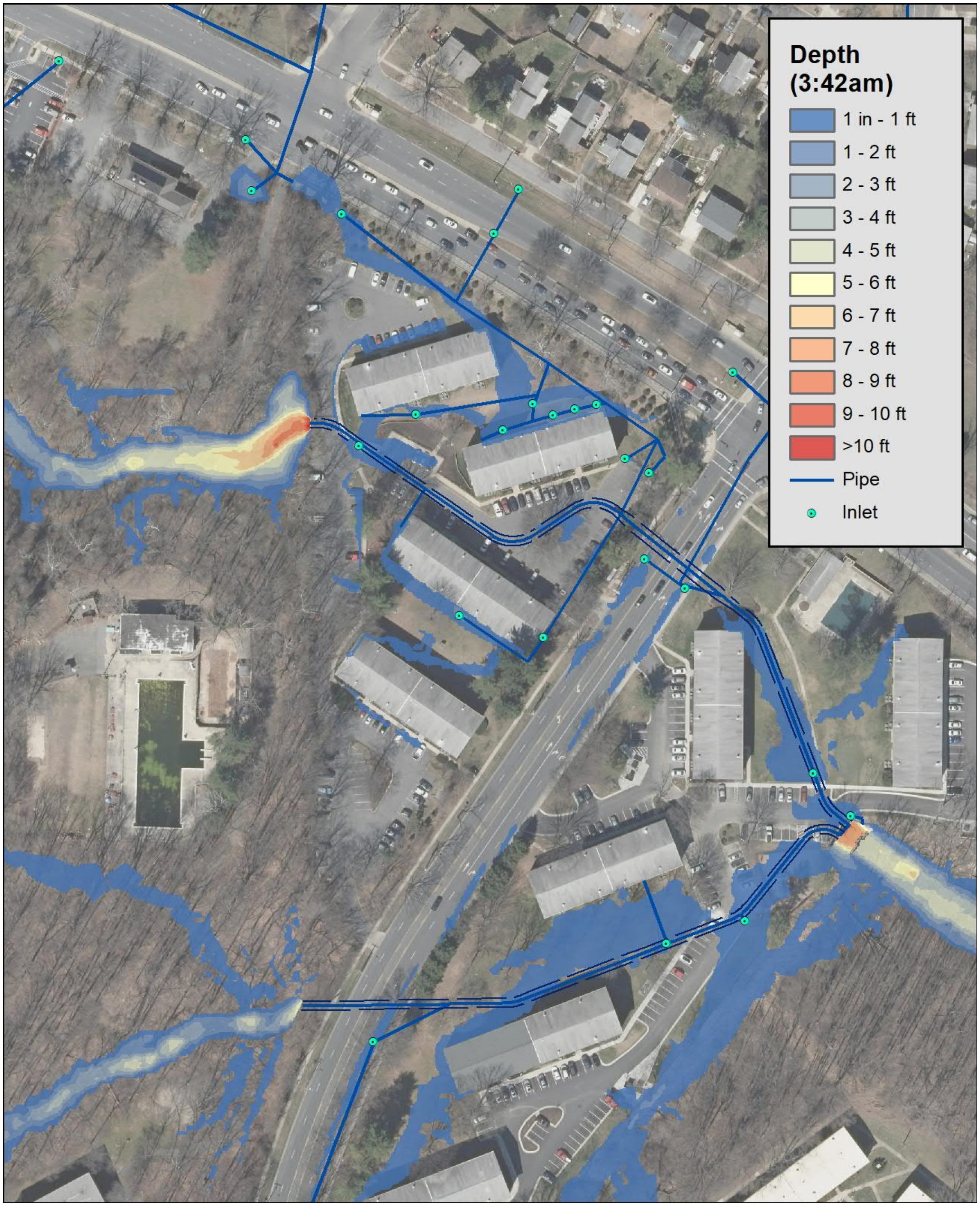

Figure 135 - 2D Simulation Depth Map, 3:42 am
Stormwater from the surcharged inlet has overtopped the adjacent curb in the parking lot and flowed to the low point between Buildings 7 and 8.

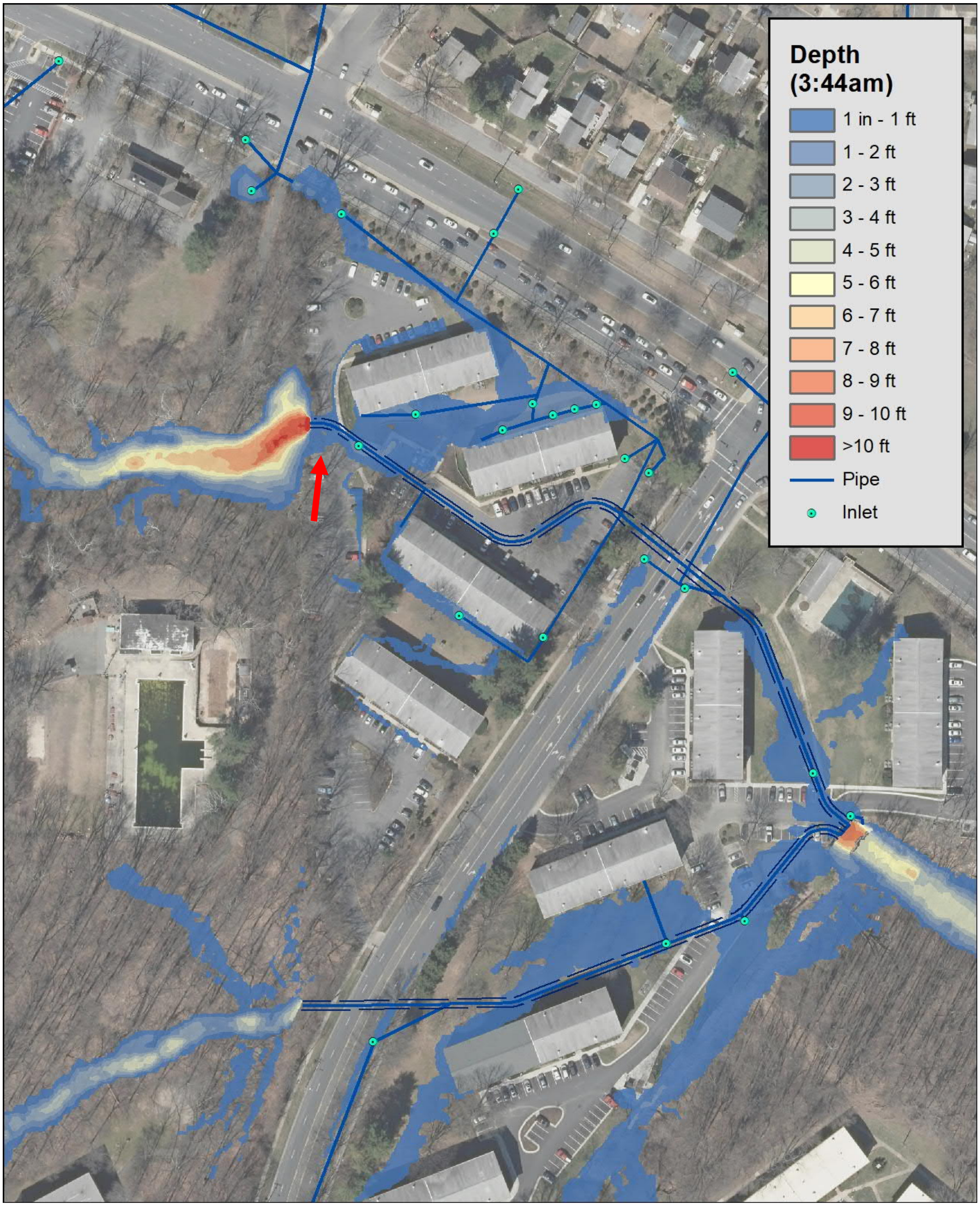

Figure 136 - 2D Simulation Depth Map, 3:44 am
Stormwater overtops the berm at the upstream end of the culvert and flows to the low point between Buildings 7 and 8.

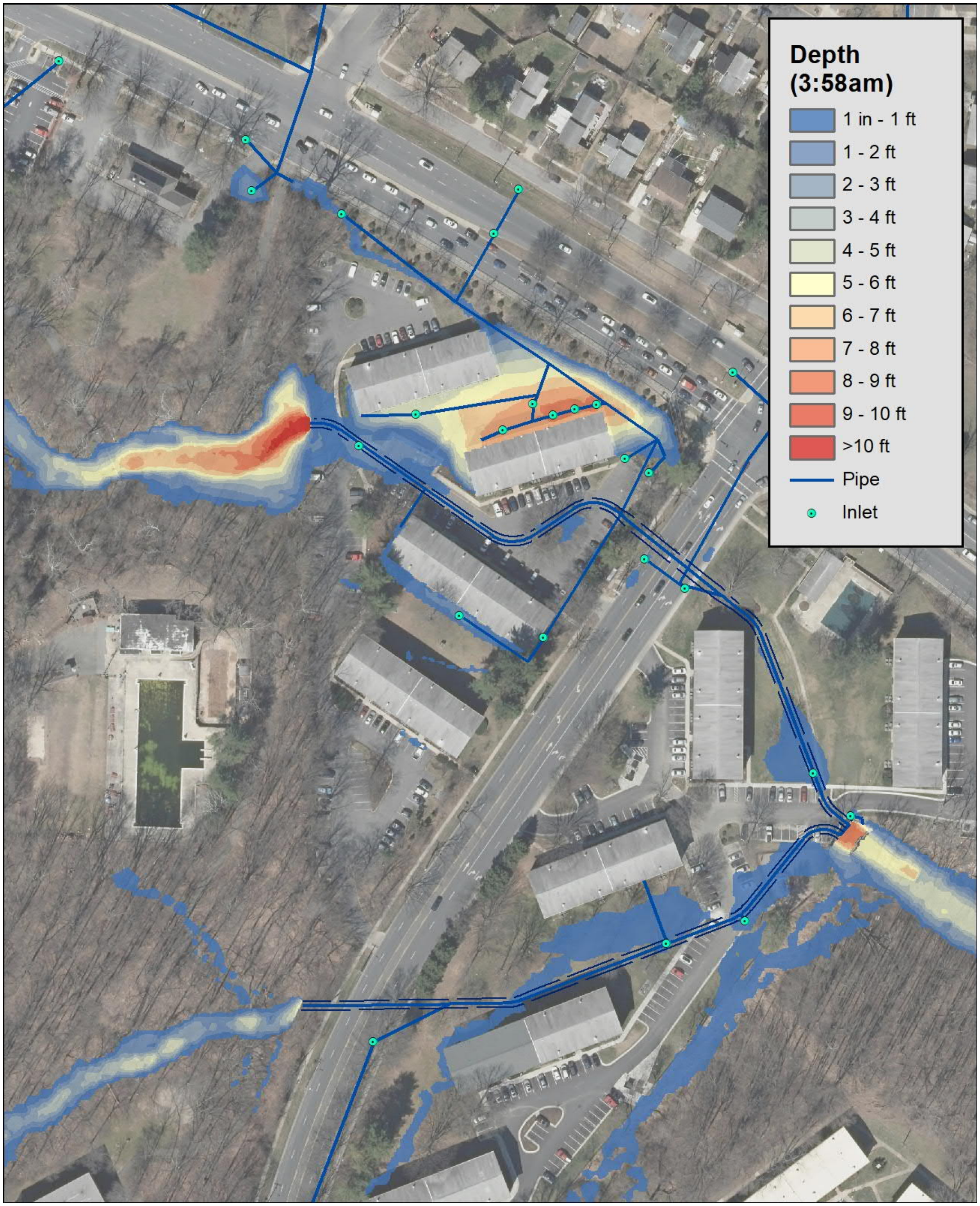

Figure 137 - 2D Simulation Depth Map, 3:58 am
Stormwater fills the depression between buildings 7 and 8.

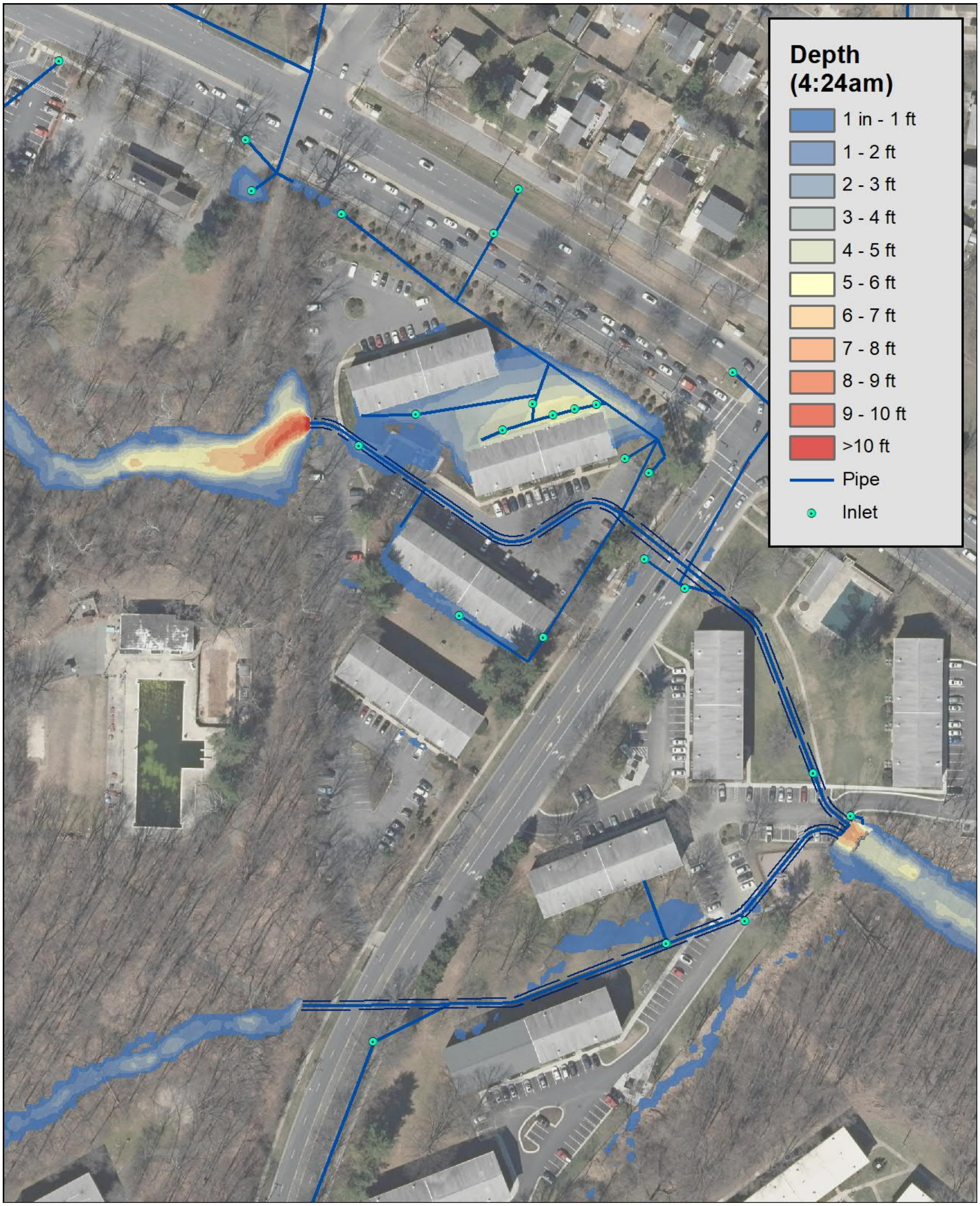

Figure 138 - 2D Simulation Depth Map, 4:24 am
Overtopping at the upstream berm has stopped and water begins to slowly drain out of the depression between Buildings 7 and 8.

A summary showing the depth of water at the low point of the depression between Buildings 7 and 8 is shown in Figure 139.

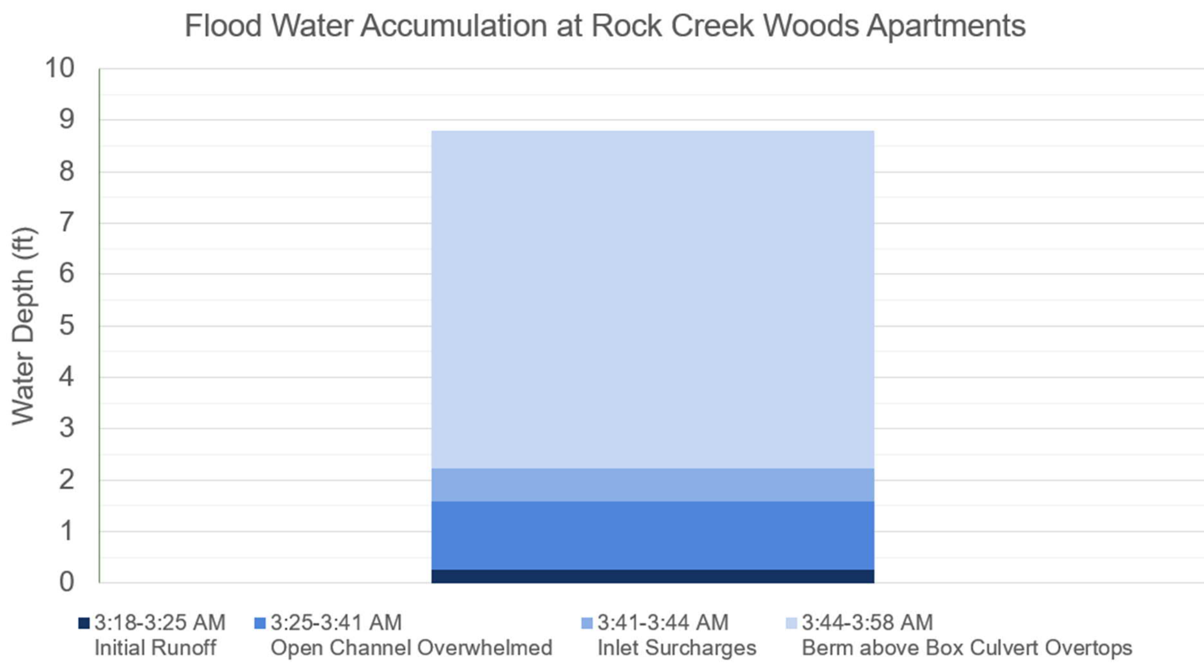

Figure 139 - Depth of Water between Buildings 7 and 8 during Hurricane Ida on 9/1/2021.

Figure 140 and Figure 141 display the total inflow rate to each culvert. Note that discharges vary by sectional area, thus quotes taken only slightly upstream or downstream may differ if sectional geometry changes. Previous drainage studies by FEMA and Montgomery County quote the peak discharge of Tributary 2 at a distance slightly upstream of the culvert headwall, on the stream itself. To be approximately consistent, the model flow rates displayed in the charts below are taken from nearest upstream node to each culvert entrance. For the northern culvert, the node was located 8 feet upstream, at the top of the concrete apron. For the southern culvert, the node was located 29 feet upstream.

The peak stages reported by the 2D model can also be viewed in Figure 142. For visual reference, Figure 143 shows the locations of each corresponding node in the chart.

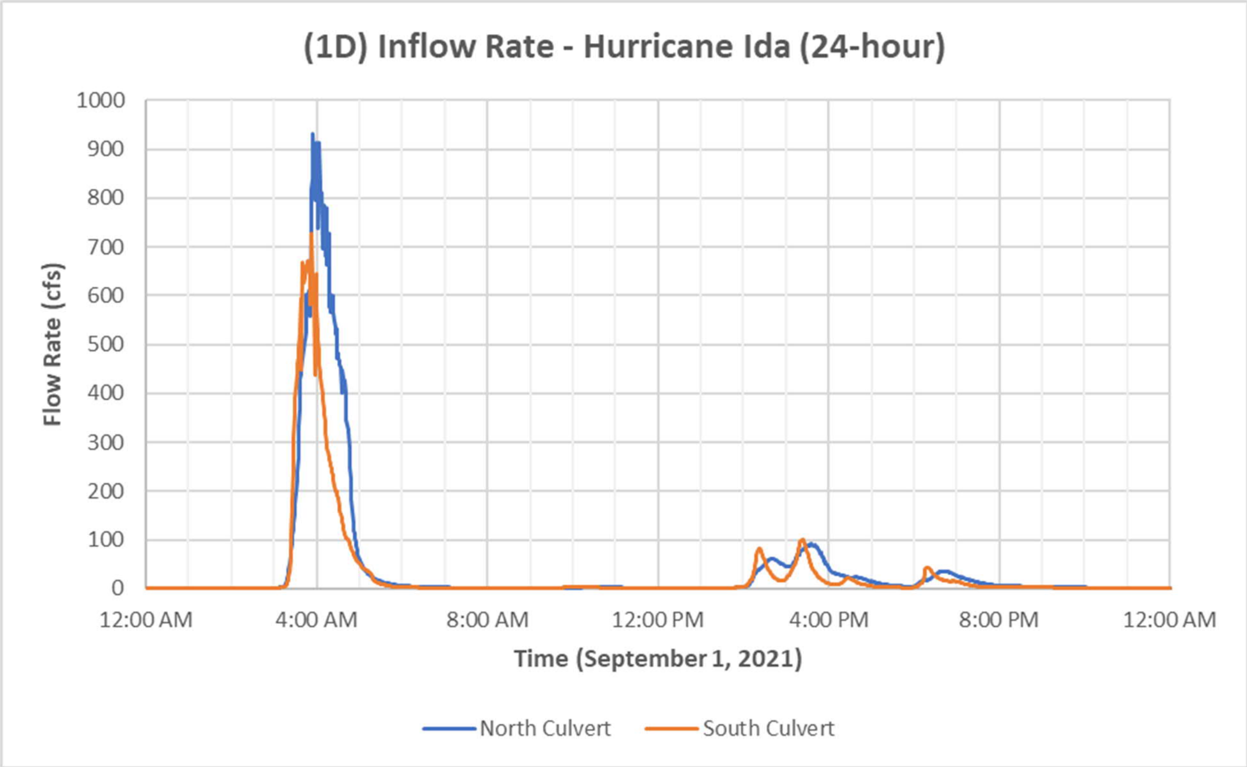

Figure 140 - 1D Modeled Inflow Rate Chart - Hurricane Ida (24-hour)

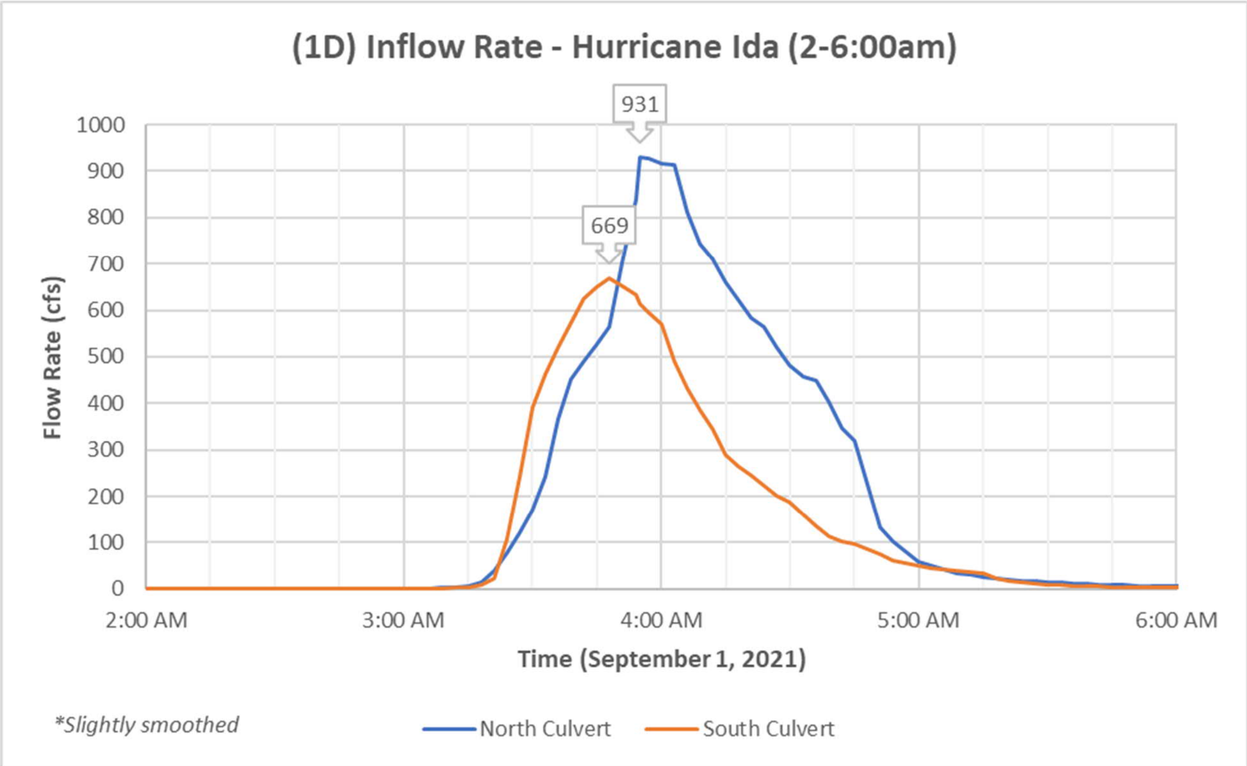

Figure 141 - 1D Modeled Inflow Rate Chart - Hurricane Ida (2-6:00am)

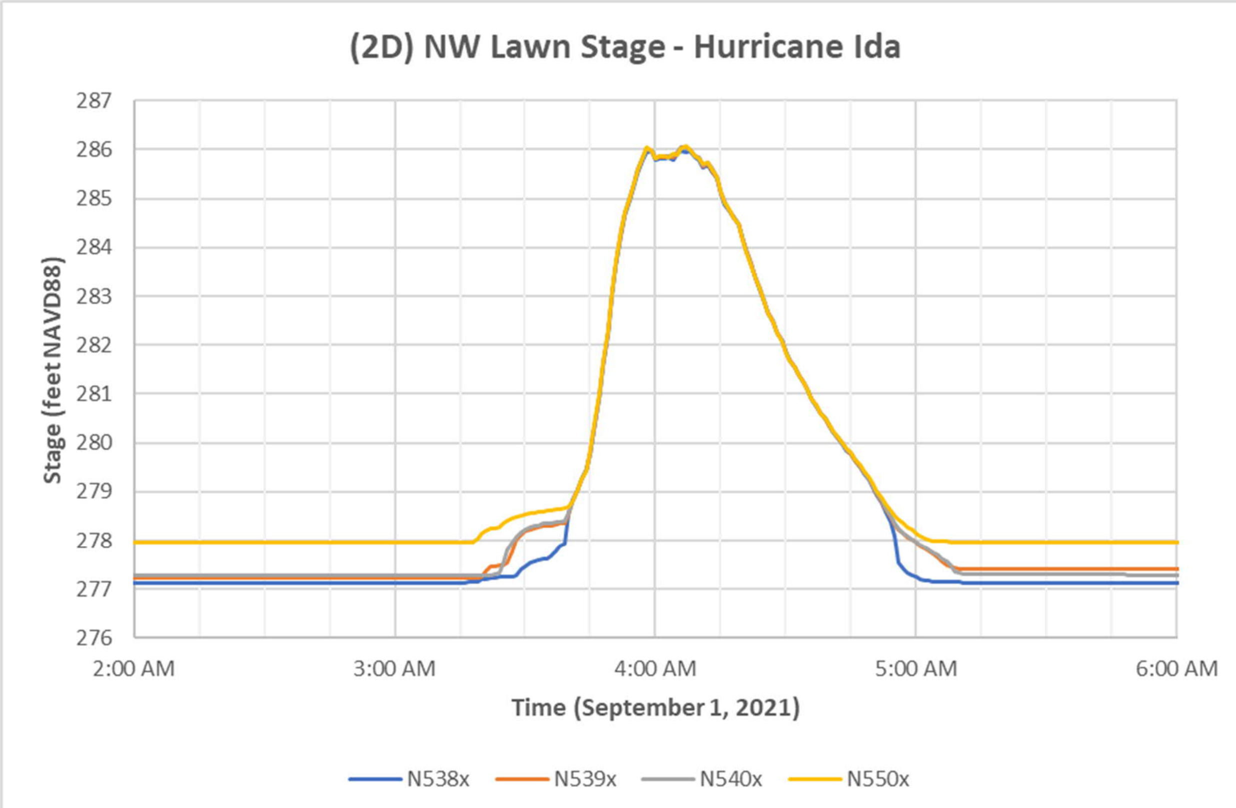

Figure 142 – 2D Model - Hurricane Ida flood stages at Rock Crest Woods Apartments by node number

Figure 143 - Map of ICPR 2D model showing inlet locations with labeled node numbers

6.3.1.4 Other Flooding Locations

In regard to flooding upstream and downstream of the Rock Creek Woods Apartments, Figure 144 through Figure 147 show the 2D modeled maximum overland flow depths, simulated for the Hurricane Ida event, at other notable locations. While the existence of flooding at these locations was known, there is no documented evidence of this flooding outside of witness accounts. As such, the flood depths and extents apparent in these figures should be taken as approximate only.

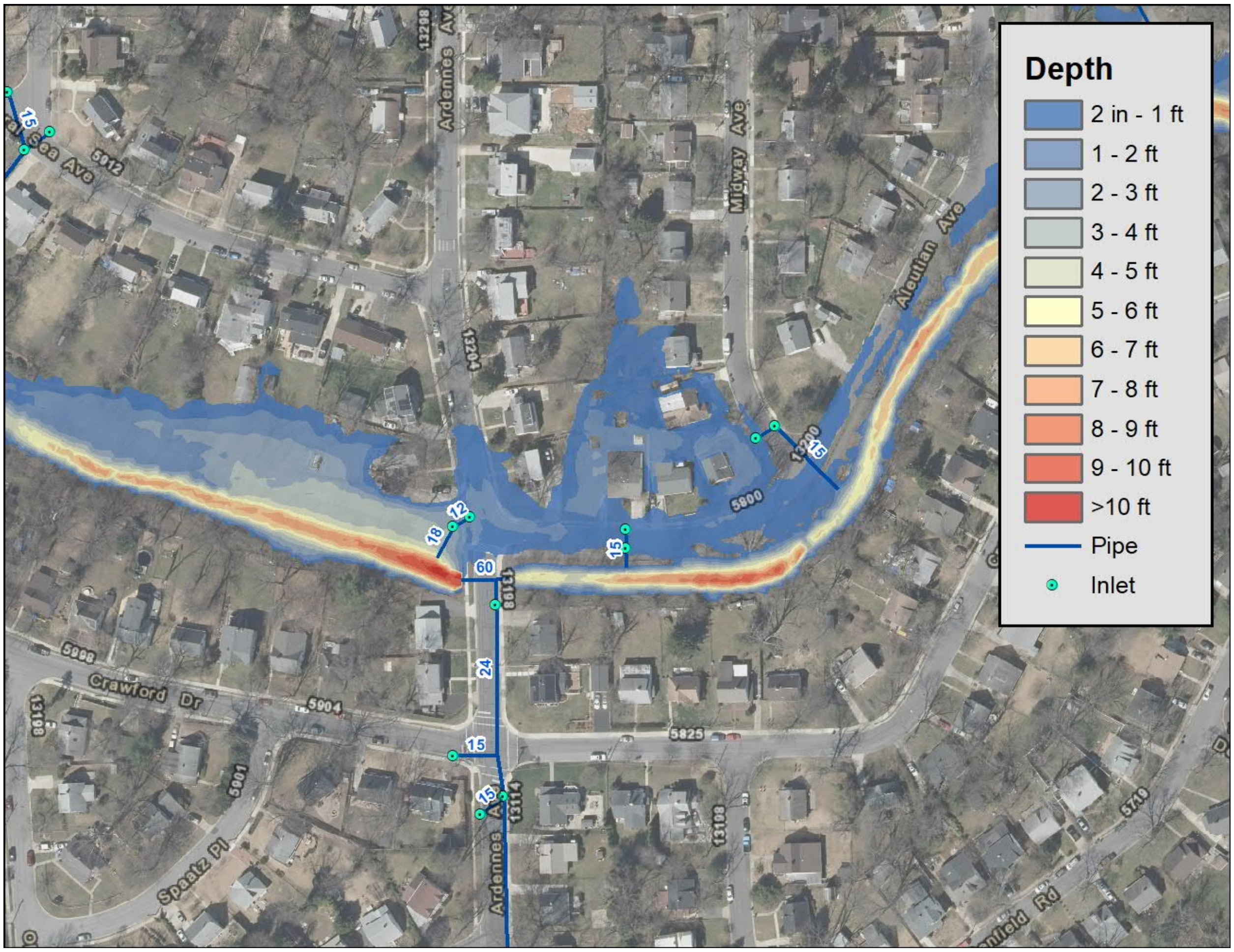

Figure 144 - 2D Modeled Max Depths of Hurricane Ida at Ardennes Ave

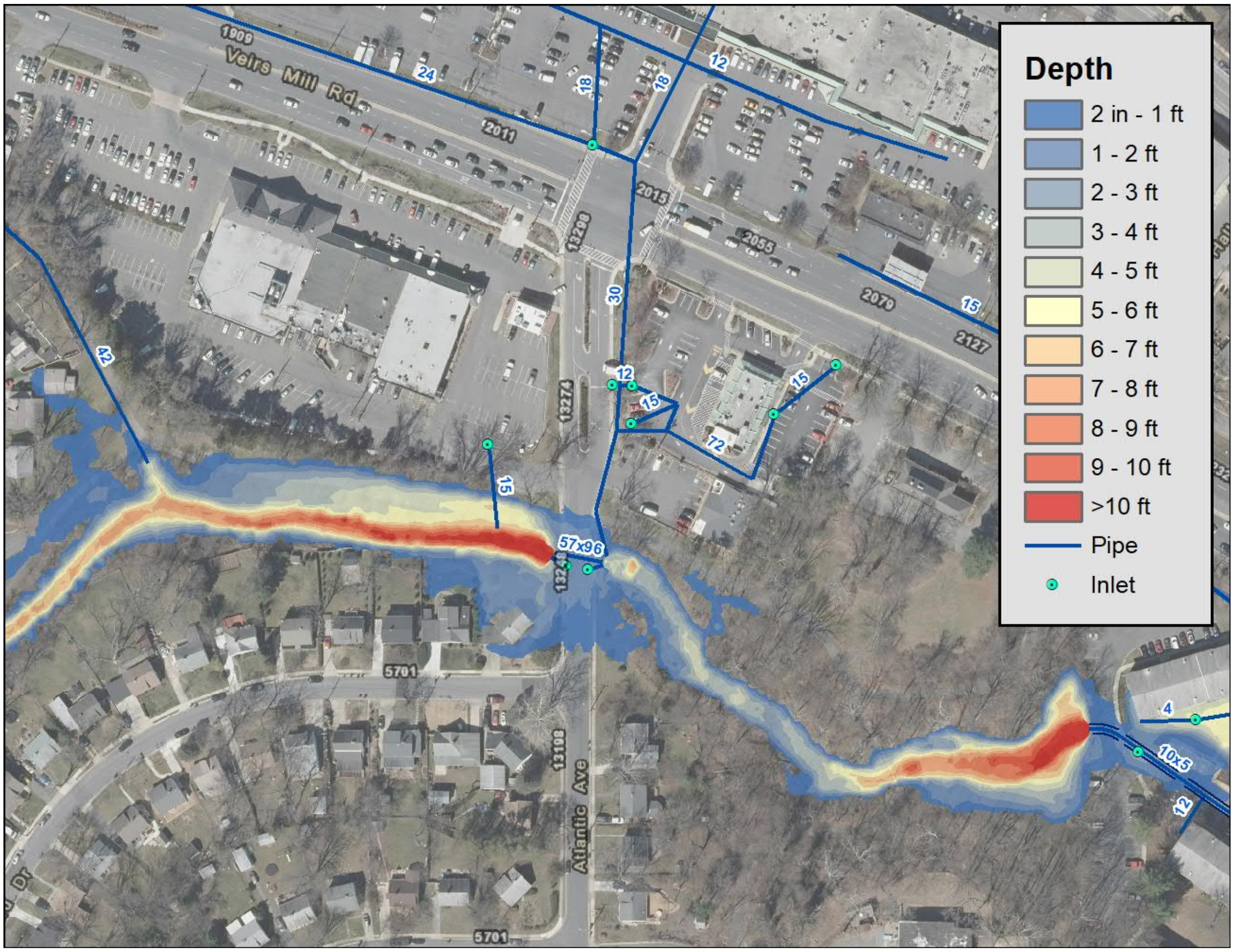

Figure 145 - 2D Modeled Max Depths of Hurricane Ida at Atlantic Ave

The results show that the Tributary 2 dual 60-inch culvert crossing Ardennes Ave, and the 8-ft x 4.75-ft box culvert crossing Atlantic Ave, both overtopped. Ardennes was likely the first to overtop at ~3:38 am, with Atlantic following only minutes later at ~3:42 am.

Some residential structures are shown to be inundated at each location. The first occupied floor elevations of these structures are unknown, as well as the presence of any basement floors. Therefore, it cannot be conclusively determined if these structures were in fact damaged by inundation.

Figure 146 - 2D Modeled Max Depths of Hurricane Ida at the Twinbrook Community Rec. Center

Flooding at the Twinbrook Community Recreational Center was known to have extended far into the lawn area adjacent to the primary structure. This was approximately confirmed by the model. Notably, this is also the location of the rain gauge from which record data that was used for this model.

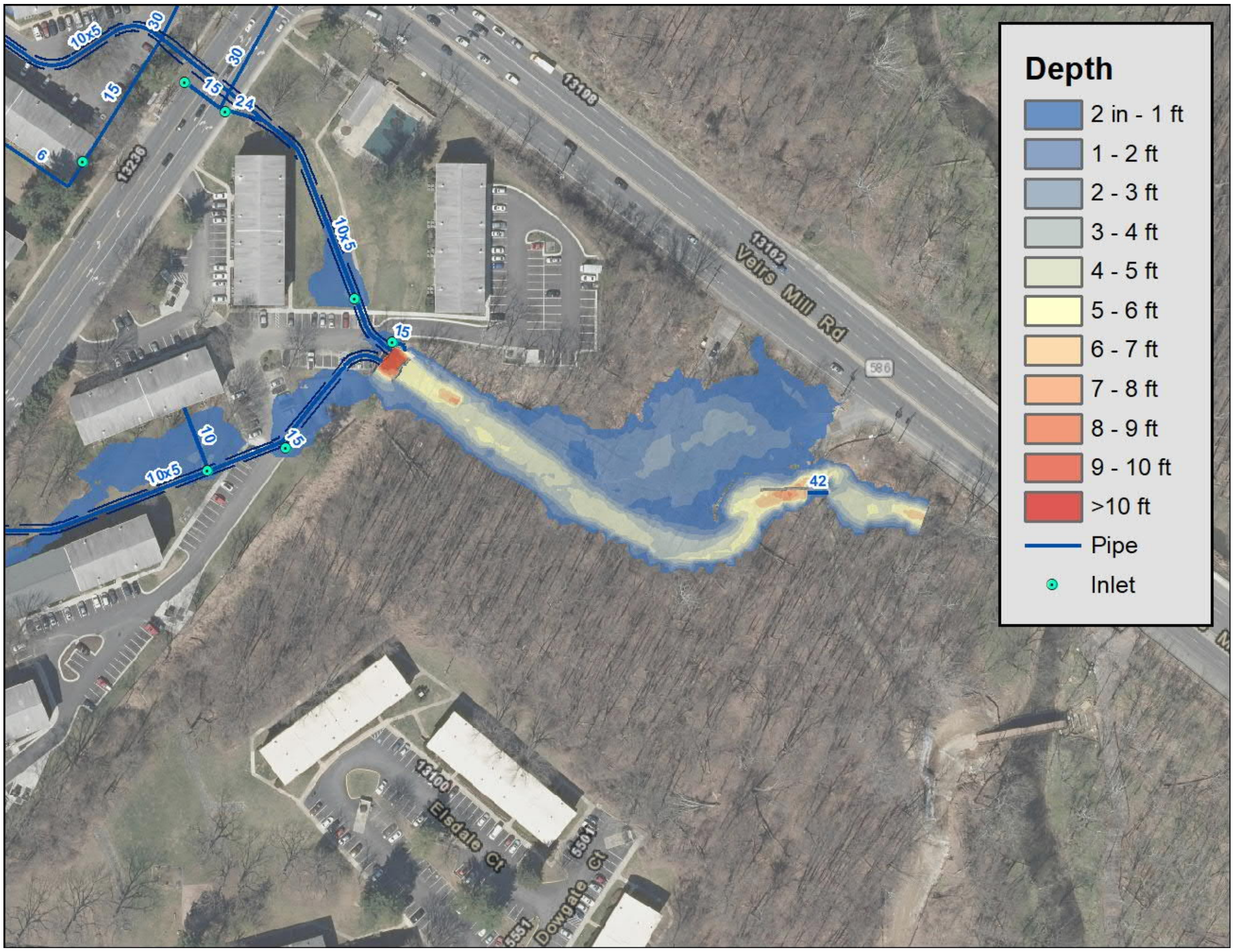

Figure 147 - 2D Modeled Max Depths of Hurricane Ida approaching Rock Creek

At the downstream end approaching Rock Creek, channel flows overbanked far into the northern floodplain. Waters did not breach into Veirs Mill Road, largely in part to the functional storage provided by the floodplain and associated vegetation.

The dual 42-inch RCP culvert at the abandoned roadway, and the adjacent bypass wall on the upstream end of the culvert, were also overtopped. The bypass wall generally has a top elevation of 255.0 NAVD88. This is 1 foot above the 254.0 modeled boundary condition at Rock Creek. The 2D simulation showed this wall was crested at ~3:26 am, and that the culvert was fully overtopped about 10 minutes later at ~3:36.

Figure 148 shows that the culvert was overtopped by approximately 3 feet during peak discharge (~4:00 am), with the depth tapering to the downstream end. This overtopping was sustained for ~70 minutes and terminated at ~4:45 am.

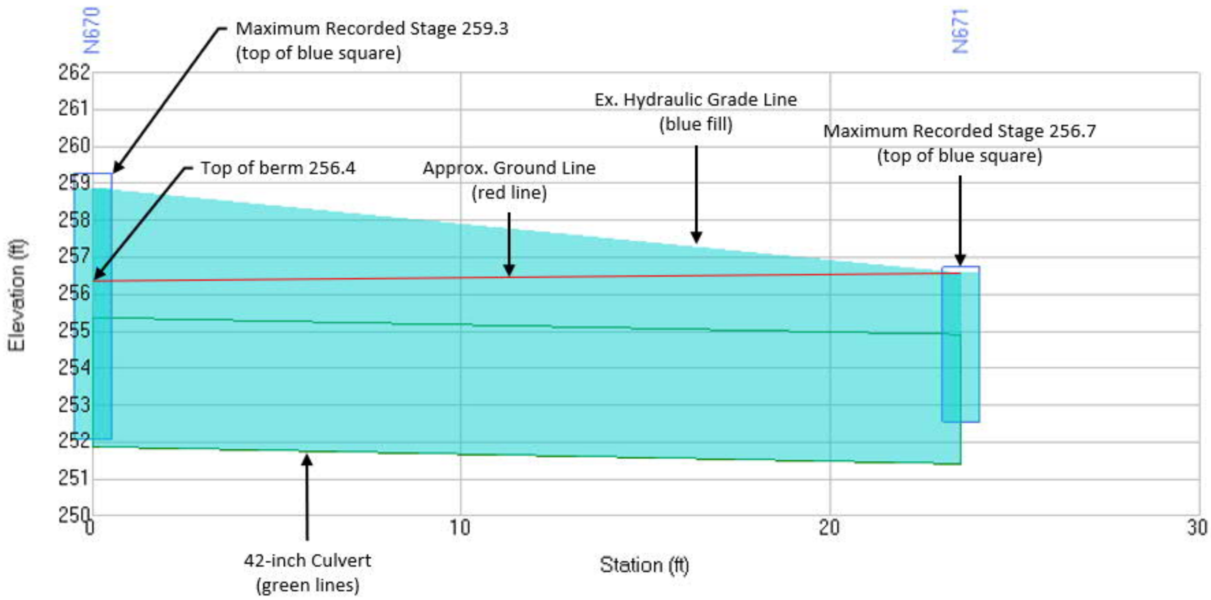

Figure 148 - 2D Model Maximum Stage Profile (Hurricane Ida) at Dual 42-inch Culvert

6.3.2 Level of Service Analysis

To determine the drainage level of service (LOS) provided by each culvert, the 1D model was used to iteratively simulate categorical synthetic storms. For each storm event, three different temporal rainfall distributions were tested: the SCS Type II distribution (24 Hour duration storm), the NOAA Atlas 14 site-specific distribution 24 Hour duration storm, and the NOAA Atlas 14 site-specific distribution 6 Hour duration storm.

Overtopping of the upstream headwall and berm was used as the determinate pass/fail criteria for each culvert. Note, a *design level of service* would normally factor a certain amount of freeboard. In this analysis freeboard was not factored as 1) design standards may vary, and 2) the “design” LOS may not reflect the as-built LOS. The H&H model was developed from as-built conditions, and thereby the performance of the culverts in the model reflect an as-built LOS.

Simulation results are shown for the northern culvert in Table 12 on the following page. The southern culvert is not included as the LOS analysis revealed that it is able to pass up to a 500-year event.

Table 12 – Northern Culvert As-Built Level of Service Analysis Results

24 Hour Storm Duration:

Storm Event	Rainfall (inches)	Overtopping Stage	NOAA Atlas 14		SCS Type II	
			Max Stage	Service	Max Stage	Service
01-year 24-hour	2.61	291.07	284.63	PASS	-	-
02-year 24-hour	3.15	291.07	285.03	PASS	-	-
05-year 24-hour	4.05	291.07	288.62	PASS	-	-
10-year 24-hour	4.84	291.07	290.67	PASS	290.90	PASS
11-year 24-hour	4.92	291.07	290.63	PASS	290.97	PASS
12-year 24-hour	5.00	291.07	290.77	PASS	291.25	FAIL
13-year 24-hour	5.08	291.07	290.87	PASS	291.29	FAIL
14-year 24-hour	5.17	291.07	291.11	FAIL	291.30	FAIL
25-year 24-hour	6.06	291.07	291.37	FAIL	291.79	FAIL
50-year 24-hour	7.14	291.07	291.86	FAIL	292.07	FAIL
100-year 24-hour	8.36	291.07	292.00	FAIL	292.31	FAIL

6 Hour Storm Duration:

Storm Event	Rainfall (inches)	Overtopping Stage	NOAA Atlas 14	
			Max Stage	Service
01-year 06-hour	1.86	291.07	284.33	PASS
02-year 06-hour	2.25	291.07	284.73	PASS
05-year 06-hour	2.85	291.07	285.11	PASS
10-year 06-hour	3.34	291.07	288.23	PASS
25-year 06-hour	4.06	291.07	290.49	PASS
38-year 06-hour	4.38	291.07	290.96	PASS
39-year 06-hour	4.41	291.07	291.09	FAIL
50-year 06-hour	4.68	291.07	291.43	FAIL
100-year 06-hour	5.35	291.07	291.76	FAIL

The LOS analysis results indicate that the northern culvert passes up to a 13-year 24-hour storm event when modeled with the NOAA Atlas-14 distribution and up to an 11-year 24-hour event when modeled with the SCS Type II distribution. As previously noted, the NOAA curve is slightly more distributed over time, and is thereby less intense than SCS curve. The NOAA 24-Hour storm duration curve represents the current the standard of measure in Montgomery County, therefore **the northern culvert was determined to have a current 13-year as-built drainage level of service**. When considering a shorter rainfall duration of higher intensities, the culvert can pass up to a 38-year 6-hour storm event when modeled with the NOAA Atlas-14 6-hour rainfall distribution. Regarding the determined hydraulic LOS, Table 13 indicates that the total volume over the 4-hour period between midnight and 4 am, bounding the primary Hurricane Ida rainfall storm event of 9/1/2021, correlated to the peak flow produced by a 46-year, 24-hour storm event. Modeled Tributary 2 peak flow(s) of the 50-year 24-hour synthetic storm event lend credit to that calculation. The Hurricane Ida simulation produced a peak flow of 931 cfs, whereas the 50-year (NOAA -14) simulation produced a peak flow of 932 cfs; remarkably close values despite significantly different durations (4-hr vs 24-hr). For reference, Table 13 below also displays the modeled peak flow rates of the 5- to 500-year categorical storm events (24-hour duration), near the entrance of the northern Tributary 2 culvert. Corresponding flow rates were extracted from previous stormwater and flood insurance studies (FIS) for comparison.

Table 13 – Flow rates of 1D Model, City of Rockville Stormwater Study (1974), and current effective FEMA FIS (2006)

Storm Event	Hurricane Ida Flow Rate (cfs)*	NOAA -14 Flow Rate (cfs)***	SCS Type II Flow Rate (cfs)***	1974 Rockville Study Flow Rate (cfs)	2006 FEMA FIS Flow Rate (cfs)
5-year	--	616	663	893	--
10-year	--	804	817	1018	1041
LOS**	--	841	832	--	--
25-year	--	860	915	1199	--
Ida (46 year)	931**	--	--	--	--
50-year	--	932	1013	--	1447
100-year	--	978	1129	--	1714
500-year	--	1122	1391	--	2540

** 1D modeled flow rate*
*** Per gaged rainfall data for 9/1/2020 Storm Event (*Twinbrook Community Recreation Center Rainfall Gage)
**** Using NOAA & SCS 24-hour storm duration distribution*

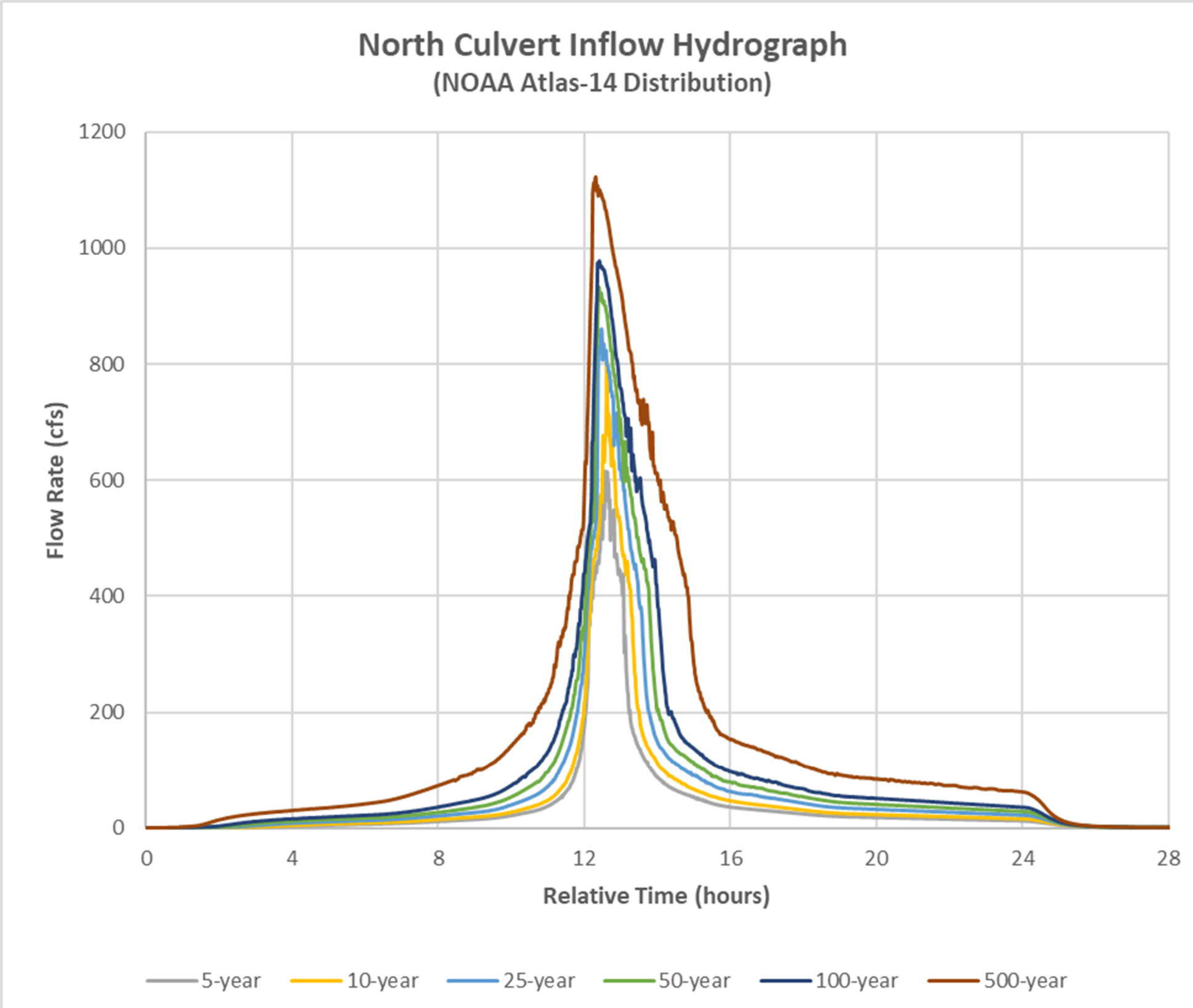

Figure 149 – North Culvert Inflow Hydrographs for Synthetic Storms

Hydrologically, Table 12 indicates that a rainfall event of approximately 4.41 inches in less than 6 hours or 5.17 inches in 24 hours will result in a culvert overtopping condition at the Twinbrook Tributary 2 culvert location. Total rainfall volume depends on rainfall duration and intensity which relates to storm event return frequency. Table 13 indicates that the Hurricane Ida peak flow rate of 931 cfs falls between the 25-year, 24-hour (860 cfs) and 50-year, 24-hour (932 cfs) storm event peak discharges which is approximately a 24-hour storm event peak discharge with a 46-year return frequency. However, this does not mean that the Tributary 2 culvert has a 46 year LOS as per the modeling rating analysis. The culvert flow rate with no overtopping was calculated at 841 cfs which is equivalent to a 13-year, 24-hour hydraulic LOS. Practically, the current Twinbrook Tributary 2 culvert conduit hydraulic condition was only able to assimilate 841 cfs of the Hurricane Ida storm event peak flow prior to overtopping and conveying the remaining 90 cfs via weir (over the top) flow.

While the Tributary 2 culvert was theoretically originally designed in the 1960's to discharge 1030 cfs, in reality the actual LOS is ultimately reduced due to the dynamic flood stage-relationship between the downstream open channel and Rock Creek, further influenced by the outlet baffle system, and additional head losses (besides friction) along the span of the culvert. Considering these hydraulic and hydrodynamic adverse effects to conduit flow, the current serviceable flow capacity of the Tributary 2 culvert is closer to a 10-year LOS or approximately 804 cfs.

6.4 Reproduction of HEC-2 Backwater Model

In historical review of the development methodology for the current 100-year regulatory floodplain of the Rockville area, it was revealed that the floodplain was originally mapped for the 1978 City of Rockville Flood Insurance Study.

Two years prior, a HEC-2 hydrologic & hydraulic model was developed to simulate backwater stages from Rock Creek, which thereby established the 1978 Rockville floodway/floodplain. This study was able to obtain the original HEC-2 model source code, which is presented in Appendix E.

With the source code in-hand, this study verified that the original HEC-2 model was later incorporated into subsequent FIS updates for Montgomery County, which included the Rockville area from 1991 and forward. A model conversion tool developed by FEMA was applied to convert the HEC-2 source code into a format compatible with HEC-RAS Version 6.1, the current software version at this time. As the conversion was not exactly 1:1, some necessary minor adjustments were made, such as those for the Special Bridge Routine. However, the overall model schematic and associated input data was essentially the same. Thus, the Rockville floodplain now depicted in current flood insurance rate maps (FEMA, effective 2006) remains unchanged from 1978.

To appraise the 1978 FIS modeling methodology, this study reproduced the HEC-RAS model from the original HEC-2 source code. Figure 150 shows the HECRAS 1D-geometry scheme for the reproduction.

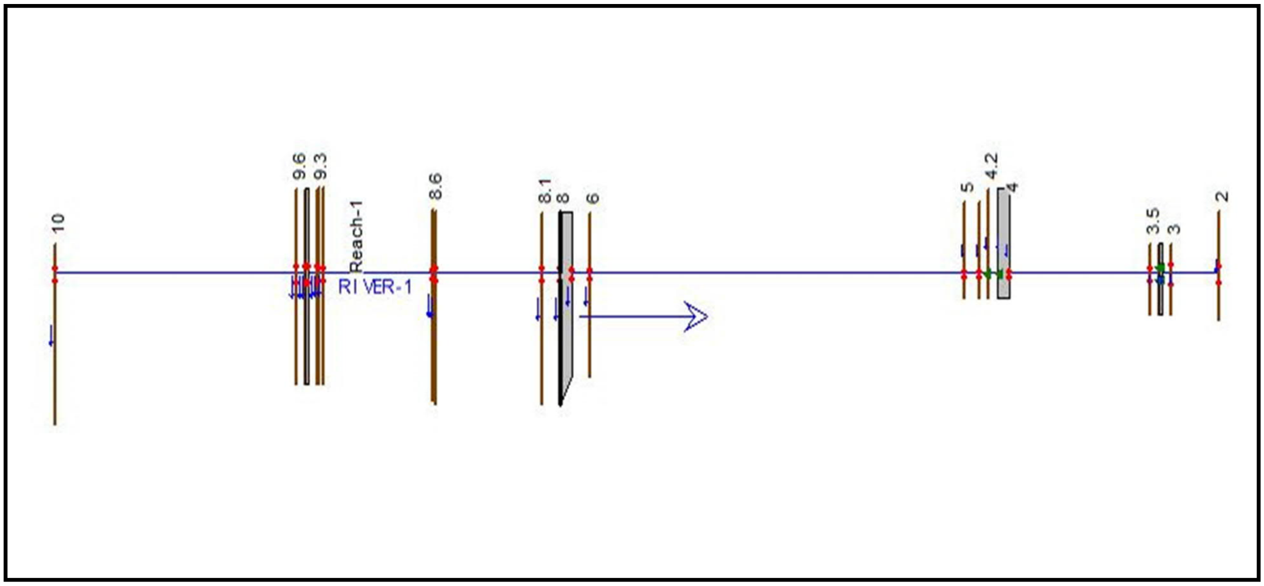

Figure 150 - HECRAS Schematic of 1978 HEC-2 Model Cross Section Coding

Cross-section “A” of the City of Rockville FIS profiles (Cross-section “2” of the HEC-2 model) was verified to be located at the Tributary 2 culvert entrance.

In establishing the starting 100-year water surface elevation at this section, the 843-ft culvert enclosure was not modeled. Instead, the Flood Insurance Administration consultant applied an iterative procedure which, by the calculated slope of the energy line, converged to what was estimated as the 100-year stage at the downstream floodplain.

Table 14 compares the original HEC-2 100-year stage results with the reproduced HEC-RAS run, as well as the 100-year stage calculated by the ICPR4 model. Table 14 indicates that the HEC-RAS reproduction is an accurate representation of the original HEC-2 code, and that the HEC-2 code was in-fact used, unchanged, to map the 100-year regulatory floodways presented in the current effective 2006 Montgomery County FIS.

Table 14 - Tributary 2 Model Stage Comparison for 100-year Event (Feet NGVD)

Cross-Section ID	**1978 HEC-2 Model**	**Reproduced HEC-RAS Model**	**ICPR4 Model**
XS-A *	293.3	293.3	291.61
** Approximate entrance to Tributary 2 enclosure*			

The lower 100-year stage calculated using the ICPR4 1-D hydrodynamic model can be explained using a lower 100-year 24-hour peak discharge of 1,129 cfs (SCS distribution) instead of the 1,714 cfs applied in the original HEC-2 model. The latter discharge was calculated using a modified Snyder formulation.

7.0 Study Findings and Results

This study researched all available public domain sources to acquire technical reports and/or documentation pertaining to the original design of the Rock Creek Tributary 2 culvert which overtopped on September 1, 2021. Although some identified documents were unavailable, key technical studies and copies of development plans were secured that provided insight for the assessment of the original culvert design.

In conjunction with the detailed survey and hydrologic & hydraulic modeling of this study, conclusions were drawn regarding the cause of the culvert overtopping, as well as the as-built level of service provided by the culvert. The following sections summarize the historical data and decision-making pertinent to the Hurricane Ida event, and the overall assessment of the culvert design and performance.

7.1 Culvert Design

This study obtained copies of the 1962 plans for the relocation of Halpine Road (Twinbrook Parkway), which contained critical hydrologic & hydraulic data for the design of the Rock Creek Tributaries 1 and 2 crossing of the new roadway. The culvert design data allowed Mercado Consultants and WRMA to confirm that the culvert hydraulic openings for both Tributaries were designed using the Talbot methodology, the prevalent methodology in the 1960s. The peak discharge data obtained for use with the Talbot formulation indicated that a 10-year drainage level of service was selected to design the original culverts under Twinbrook Parkway.

This study was also able to secure copies of the 1966 Rock Creek Tributary 1 and 2 culvert enclosure plans that allowed for the development of the Bullis property into the current Rock Creek Woods Apartments complex. Despite the lack of any specific hydrologic data within the plans, it was noted that the plans' Engineer of Record (Greenhorne & O'Mara) was the same engineer that designed the Halpine Road Relocation culverts.

There were not many design tools available to engineers in the 1960s to rate long span culverts. Even by today's computer-based methodologies, it is a complex task that requires advanced hydrodynamic modeling techniques. It can be surmised (not concluded) that the designer selected the smaller 10-foot (W) x 5-foot (H) concrete box enclosure based on available topographic gradients. In theory, if the Bullis property had the same gradients as were used for the design of the 14-foot (W) x 7.5-foot (H) box section under Twinbrook Parkway, then the same size would have likely been used for the entirety of the culvert enclosure. Instead, the designer appears to have taken advantage of the average 3% gradient for the design. This steeper slope theoretically afforded the same flow rate with a smaller section, as can be calculated with the Manning's flow equation.

It can also be surmised that the 10-year peak flow for the original culvert enclosure was carried over to the design of the extensions, since the tributary drainage area was basically unchanged.

Regarding the culvert's construction, this study determined the as-built horizontal and vertical alignment accuracy of the culvert layout. This was done by applying state-of-the-art laser scanning to establish any deviations from original 1966 plans. Horizontal and vertical deviations were found, but are within an acceptable margin, indicating that the culvert was generally constructed according to the permitted design plans.

Through this assessment, this study was also able to determine that adjacent storm drains as built did not match the original design. These storm drains entered the side of the box culvert enclosure and played an important role in the culvert overtopping event.

7.2 Floodplain Mapping

Although the original Tributary 2 culvert enclosure was designed in the 1960's prior to the enactment of the national Flood Insurance Program in 1973, the current Montgomery Flood Insurance Study (FIS) does show a Special Flood Hazard Area west (City of Rockville) and east (Montgomery County) of the Rock Creek Woods Apartments. Within this hazard zone, FEMA identifies 100-year regulatory floodplain elevations for the placement of first floor elevations. For this reason, this study requested all available historic FEMA/FIS products for the preparation of regulated floodplains shown on the effective 2006 Montgomery County FIS.

This study did not receive all the requested historical data but was able to acquire the key 1978 City of Rockville and 1979 Montgomery County FIS reports and partial HEC-2 model documentation used to map the 100-year floodplain. The assessment of the data indicated that the original HEC-2 model data used for the City of Rockville in 1978 was incorporated without changes into the successive Montgomery County FIS updates in 1979, 1984, 1992 and 2006.

A thorough review and reproduction of the original HEC-2 Backwater model used in the 1978 City of Rockville FIS indicated that the Rock Creek Tributaries 1 and 2 culvert enclosures were not modeled. Instead, the Flood Insurance Administration consultant applied an iterative procedure which, by the calculated slope of the energy line, converged to what was estimated as the 100-year stage at the downstream floodplain.

It was noted that the same consultant (and/or its subconsultants) was also preparing the Montgomery County/Rock Creek H&H study at the time.

7.3 Hydrologic (Rainfall) Assessment

The hydrologic assessment revealed that the overtopping of the Rock Creek Tributary 2 Culvert below the Rock Creek Woods Apartments' Property was the unfortunate result of a rarely intense storm event resulting from the remnants of Hurricane Ida. Upon entering the US as a hurricane along the gulf coast, Ida began a period of rapid weakening. It dropped below hurricane strength early on August 30, 2021, before weakening to a depression later that day. As the system moved through the Northeastern United States on September 1–2, 2021, it combined with a frontal zone to unleash unprecedented rainfall across the Delaware/Maryland/Virginia region. The rainfall was particularly concentrated across the City of Rockville and Montgomery County.

The front was moving from SW to NE and between approximately 2:00 am to 4:00 am a frontal cell with intense rainfall precipitation was directly above the City of Rockville and the Twinbrook area in particular. The Twinbrook Community Recreation Center recorded 3.1 inches mostly between 3:00 and 3:45 am with a maximum precipitation rate of 6.36 inches at 3:40 am, coinciding with overtopping event. The accumulation of 2.56 inches of rainfall in a 30-minute period, during the most intense rainfall precipitation with an average of 5.12 inches per hour corresponds to a return frequency of occurrence of approximately 327 years.

7.4 H&H Model Results Discussion

Hydraulically speaking, the development of this ICPR4 model marks the first time the Rock Creek Tributaries 1 and 2 culvert enclosures have been analyzed in this capacity.

The hydrodynamic simulation revealed that the Tributary 1 culvert did not overtop during the Hurricane Ida event. Similarly, the LOS analysis indicates this culvert could pass much larger peak discharges, up to and potentially beyond the 100-year event. The primary reason for this is the substantially smaller drainage area of Tributary 1 (0.4 square miles) versus the Tributary 2 drainage area (0.7 square miles). Moreover, the southern culvert flow capacity is generally higher than that of the northern culvert, due to its relatively more efficient design.

On the other hand, the model showed that the northern culvert was not sufficient to pass the Hurricane Ida discharges through Tributary 2. The overtopping of the upstream berm was indeed largely due to the intensity of the Hurricane Ida rainfall, but was also partially the result of additional compounding factors which adversely influenced the flow capacity of the culvert.

These factors include substantial vertical & horizontal alignment head losses and associated hydraulic jumps, energy loss at the downstream tiered baffle system, and additional loading by direct discharges of overlaying site runoff through smaller, storm drain break-in taps at several locations along the culvert.

Aside from the substantial head losses inherent to the design of <u>both</u> culverts, the prolonged high-intensity precipitation (4 to 6 inches per hour for nearly 30-minutes) of Hurricane Ida was undoubtedly the largest contributing factor to the overtopping at the upstream berm of the northern culvert. It is not uncommon for rainfall events to reach these high intensities, but it is rare that such intensities are sustained for so long. As noted in Section 6.1 Rainfall Data Analysis, the rainfall accumulated during the <u>30-minute peak</u>

of Hurricane Ida approximately corresponded to an equivalent 327-year storm event (0.3% annual chance).

In typical engineering practice, culverts are not designed the pass a 300-year event of any duration. Most agencies have standardized the design of culverts to pass a 10- to 25-year, 24-hour event. This is partially due to the fact that drainage systems across the United States were- and are currently designed only to manage the most frequently occurring storm events. These range in the 1- to 10-year recurrence intervals (10 to 33% annual chance). To design drainage systems for rarer events is cost-prohibitive and sometimes financially infeasible, especially in urbanized and largely impervious locations. In cases where infrastructure is critical to public health and safety, the culvert passage of a 50- to 100-year event may be necessarily mandated if not a bridge section. For instance, in Montgomery County a culvert crossing a major highway evacuation route would be designed to at least pass a 50-year event, whereas a culvert on crossing an arterial or county road would be designed for a 25-year event[40].

With that in mind, through this flood study it was determined that the culverts serving the Twinbrook & Rock-Crest drainage areas were designed to provide a 10-year level of service; and were designed appropriately to the engineering standards and available methodology of their day.

[40] Montgomery County DOT, *Drainage Design Criteria Manual, 10th Ed*, 2014.

7.5 Final Remarks

The box culvert carrying Tributary 2 below the Rock Creek Woods Apartments has functioned without overtopping for more than 55 years, even during Hurricane Agnes in 1972 which documented hourly rainfall intensities of over 200-years frequency in the Rock Creek watershed. This fact is a testament to the historical rarity of the September 1, 2021 rainfall event; however, the occurrence of the frontal zone and the resulting extreme high rainfall intensities should not be seen as an occasional event in the future.

Climate change and warming is rapidly changing rainfall intensity patterns throughout the United States and indeed the planet. Montgomery County should consider enacting updated technical specifications for culvert design with regard to modern climate patterns. One recommendation is the adoption of a shorter (6- or 12-hour) rainfall duration for analysis, as traditional 24-hour rainfall distributions are based on rainfall monitoring data prior to the broad academic & political acknowledgment of climate changes. Another recommendation is the requirement that a future climate change event condition be modeled to ascertain the adequacy of the design. This, for instance, could be accomplished through increasing the design rainfall intensity by a given percentage using the NOAA 14 Atlas data as reference.

8.0 Appendices

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