City Of Covington

Drainage Study

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Quality information

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Executive Summary

Areas within the City of Covington, Virginia (City) have historically experienced floods caused by urban and fluvial flooding which inundates over two dozen roadways in the City. Flooding in the City primarily endangers the health and safety of residents on the western side of the CSX railroad, which bisects the City. The main traffic emergency escape routes are underpasses at North Monroe Avenue (Monroe) and East Chestnut Street (Chestnut), which frequently flood, obstructing safe vehicle passage. A picture captured of Chestnut underpass flooding is provided in **Figure 1.**

In addition, riverine flooding of the Jackson River has occurred at all times of the year, and during all major floods high velocity flood flows and hazardous conditions exist in the mainstream channel and in some parts of the floodplain.

Flooding in urban areas, such as the City, poses several significant risks, including health and safety hazards from contaminated floodwaters, which can lead to waterborne diseases and injuries. Infrastructure damage is another major concern, as flooding can severely impact roads, bridges, buildings, and utilities, resulting in costly repairs and service disruptions. Economically, businesses may face property damage, inventory loss, and operational interruptions, leading to financial setbacks. Environmental issues such as soil erosion, water pollution, and habitat destruction also arise from flooding, affecting local ecosystems and wildlife. Additionally, flooding can cause displacement and social disruption, forcing residents to evacuate and endure significant stress, particularly among the more vulnerable residents. Transportation disruptions frequently occur due to flooded roads and underpasses, which obstruct traffic flow and could potentially hinder emergency response efforts. These disruptions not only cause delays and inconvenience for daily commuters but also pose serious challenges for emergency services trying to reach affected areas promptly. Addressing these risks requires comprehensive planning, which includes the development and implementation of a stormwater model to design resilient infrastructure and effective emergency response strategies to protect communities and minimize the impact of flooding events.

The City received two Community Flood Preparedness Fund (CFPF) grants from the Virginia Department of Conservation and Recreation (DCR) to develop a Resilience Plan and Drainage Study. The City-wide Drainage Study is required to be completed prior to the completion of the Resilience Plan, as drainage improvement recommendations that are developed as part of the Drainage Study will need to be included in the Resilience Plan. Phase 2 of the Drainage Study conducted a condition assessment of the storm sewer system and developed an hydraulic and hydrologic (H&H) model to align with the CFPF Study priorities, as listed in the Grant Manual.

The City does not have their stormwater network system mapped in Geospatial Information System (GIS), and the condition of this network is unknown in most areas. City personnel routinely provide reactive maintenance of the stormwater system when complaints are filed due to flooding. Damaged and deteriorated piping is commonplace throughout the City and is likely contributing to recurrent pluvial flooding. In addition, the impact of climate change on rainfall intensity must be factored into H&H modeling efforts to determine if capacity increases are needed.

In collaboration with GAS, a separate contractor who conducted its own site survey from May 10, 2022, to April 11, 2023, AECOM also gathered site information and performed its own field investigation of the stormwater network. This investigation identified instances of conduit blockages, inaccessible structures (often due to cover lids being paved over), and other maintenance issues throughout the City. AECOM collected this information and documented the findings of maintenance items along the CSX (Chessie System and Seaboard Coastline Industries), railroad which is provided in **Appendix A** of this stormwater drainage study (study).

AECOM had previously drafted a Regional Rainfall Study which provides an analysis of the effects of climate change to the regional rainfall data and is attached to this study as **Appendix B**. The collected data was then used to develop a Personal Computer Storm Water Management Model (PCSWMM) 1-Dimensional (1-D) model simulation of the existing and proposed alternatives. The modeled results of the existing conditions (base) are provided as maps in **Appendix C**, and the proposed alternatives are provided in **Appendix D**. The base and proposed models were evaluated based on the total number of flooded junctions and the reduction in the number of flooded junctions, respectively. This dual assessment approach provides a comprehensive understanding of each modeled alternative's performance in managing and mitigating flood occurrences. This visual representation helps to clearly identify areas of improvement and highlight the differences between the two models.

The main objective of the proposed alternatives is aimed at identifying individual improvements to the stormwater system within the study area's two main underpasses, namely the Chestnut and Monroe underpasses, to allow for a safer vehicular passage. The proposed improvements adhere to the Virginia Department of Transportation (VDOT) drainage manual guidelines for underpasses, ensuring that a 100-year storm event does not reach the rim elevation of any stormwater structure within the underpass. The recommended improvements include several key actions to improve the stormwater network system's conveyance and capacity. The first recommendation is to clear out all conduits of debris and sediment fill to ensure unobstructed water flow. Secondly, the diameter of the Chestnut trunkline will be increased to 72 inches, and its negative slopes will be eliminated to improve its performance. Similarly, the Monroe trunkline will be upgraded by increasing its diameter to 60 inches and removing its adverse slopes. These comprehensive measures are designed to significantly improve the overall functionality and reliability of the drainage infrastructure. While these improvements won't eliminate flooding at all junctions within the study area, they will enable the implementation of safer and more reliable passages for traffic to cross via the Chestnut underpass or the Monroe underpass. This will improve the overall traffic flow and safety during flood events.

Additionally, modelling efforts were performed to identify a minimum diameter of conduit to be applied to the study area's stormwater network systems. This objective focused on identifying simpler recommendations for future improvements to the City's stormwater network systems, specifically those involving storm sewer replacement. By determining a recommended minimum conduit diameter, the goal is to provide the City with a simpler guideline to address flooding. This guideline was tested for a range of diameters ranging from 15-inches to 24-inches. The results show that applying a minimum diameter of 24-inches across the entire network reduces the largest number of total flooded junctions within the study area.

Flooding Incident at East Chestnut Street Underpass Documented in August 2024

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List of Accronyms

ac – Acre
CCTV – Closed-Circuit Television
CFPF – Community Flood Preparedness Fund
CMP – Corrugated Metal Pipe
CN – Curve Number
CPP – Corrugated Plastic Pipe
CSX – Chessie System with Seaboard Coast Line Industries
DEM – Digital Elevation Model
el – Elevation
IDF – Intensity-duration-frequency
in – Inch
FEMA – Federal Emergency Management Agency
FIRM – Flood Insurance Rate Map
ft – Foot
GIS – Geographic Information System
GPS – Global Positioning System

HEC– Hydraulic Engineering Circular
HEC-RAS – Hydrologic Engineering Center – River Analysis System
HGL – Hydraulic Grade Line
hr – Hour
HRM – Hydraulic Reference Manual
HUC – Hydrologic Unit Code
H&H – Hydraulic and Hydrologic
HSG – Hydrologic Soil Group
NFHL – National Flood Hazard Layer
NOAA – North Oceanic and Atmospheric Administration
NRCS – Natural Resources Conservation Service
NWI – National Wetland Inventory
LiDAR – Light Detection and Ranging
RCP – Reinforced Concrete Pipe
PCSWMM – Personal Computer Storm Water Management Model
sf – Square Feet
SOW – Scope of Work
USDA – United States Department of Agriculture
USGS – United States Geological Survey
WSE – Water Surface Elevation
VA – Virginia
VDOT – Virginia Department of Transportation
VGIN – Virginia Geospatial Information Network
yr – Year
1-D – 1-Dimensional
° – Degree

1. Introduction

1.1 Study Background and Location

The City of Covington located in the Alleghany Highlands region of Alleghany County, Virginia, (as shown in **Figure 1-1**) is situated along the Jackson River within a mixed urban, industrial, and forested areas. A key feature of the City is the CSX-operated railroad tracks, which divide the study area into East and West sections. This study primarily addresses urban and fluvial flooding issues, with a significant focus on urban flooding at the railroad underpasses on East Chestnut Street and South Monroe Avenue. These underpasses are particularly prone to flooding, causing disruptions and hazards for both vehicular and pedestrian traffic.

Additionally, the study examines fluvial flooding that impacts the remainder of the City's infrastructure, assessing how natural watercourses and heavy rainfall events contribute to broader flooding challenges such as the Jackson River flooding. By addressing both urban and fluvial flooding, the study aims to develop comprehensive solutions to improve the City's resilience and infrastructure.

Figure 1-1: Study Area Map

1.2 Study Objectives

As part of fulfilling the CFPF grant objectives, the City has conducted a study to evaluate the performance of the existing stormwater infrastructure, investigate which structures experience flooding within the project site, and assess potential solutions to improve the existing stormwater infrastructure's performance.

The PCSWMM model types have been analyzed to assess existing conditions (base model) and evaluate proposed alternatives (proposed model). The modeling setup included the analysis of five distinct storm events, each increasing in intensity. The analysis utilized a 1-D PCSWMM model to evaluate the flooding impacts on the City's stormwater network junctions. The study was conducted evaluating the 2-, 10-, 25-, 50-, and 100-year storm event return periods using regional rainfall data, 10% increased regional rainfall data, and action level flood conditions of Jackson River. The proposed models feature various combinations of alternative improvements, each modeled separately to demonstrate the effectiveness of the proposed mitigation measures. These potential improvements were evaluated against the base model.

To meet these objectives, AECOM has performed the following:

- **Data Collection and Watershed Development**
    a. Provided a comprehensive map of the surveyed utilities by AECOM and GAS within the study area.
    b. Provided a summary of recommended maintenance items within the existing stormwater networks.
- **Existing Conditions Flood Assessment**
    a. Modeled the available stormwater infrastructure to the practicable extents and identified the total amount of structures in the stormwater network that experience flooding during the 2-year, 10-year, 25-year, 50-year, and 100-year storm event model simulations
        i. Provided analysis of the Chestnut underpass flooding.
        ii. Provided analysis of the Monroe underpass flooding.
    b. Provided an exhibit of flooded infrastructure due to Jackson River's floodplain.
- **Proposed Alternatives**
    a. Identified seven potential improvements to the base model's stormwater infrastructure.
- **Alternatives Evaluation**
    a. Modeled the impacts of the seven identified proposed improvements.
    b. Provided a comprehensive recommendation of stormwater network infrastructure improvements.
- **Climate Change Impact on Stormwater Networks Map Exhibits**
    a. Provided the forecasted increases in rainfall depth for both the existing and proposed scenarios following the 10-year 24-hour return period models.

2. Data Collection and Watershed Development

2.1 Watershed Description

The City is located in the Upper James Hydrologic Unit Code (HUC) 8 watershed, one of the only watersheds in Virginia that Federal Emergency Management Agency (FEMA) has not funded to study yet. There are no new FEMA floodplain maps in the works right now, but AECOM suspects that FEMA may fund this watershed in the next couple of years.

The City is topographically divided by a CSX railway into two distinct areas: the east section and the west section. The west section is bounded by the Jackson River on the north, west, and south sides, making it more directly influenced by the river's hydrological dynamics. This proximity facilitates natural stormwater runoff into the river but also increases susceptibility to riverine flooding. In contrast, the east section extends into the mountainous region of the George Washington National Forest to the east and is bordered by urban development to the north and south. This area does not have a direct overland stormwater runoff path to the Jackson River. Instead, stormwater is conveyed through stormwater networks to the Jackson River, which can result in potential water accumulation and localized flooding within urban areas. Consequently, the east section requires engineered stormwater management solutions, such as an improved drainage system, to effectively manage runoff and mitigate flood risks. Understanding these distinct topographical and hydrological characteristics is crucial for effective urban planning and flood risk management in the City.

The City currently operates an existing stormwater network system that is unmapped, and there is no publicly available stormwater GIS database within the City's records. This lack of detailed mapping and GIS data presents significant challenges for effective stormwater management and urban planning. Without a comprehensive understanding of the network's layout, it is difficult to identify and address potential issues such as blockages, inefficiencies, or areas prone to flooding.

A large part of the study area falls within the 100-year floodplain of the Jackson River (as seen in **Figure 2-1**), which exacerbates the flooding challenges in this region. FEMA's flood zone AE encroaches upon 78 stormwater junctions while FEMA's flood zone X encroaches upon 44 stormwater junctions; flood zone AE signifies a 1% chance of flood hazard and the flood zone X represents 0.2% chance of flood hazard. The floodplain impact to the City's land area is provided in **Figure 2-1,** and an in-depth discussion on the impact of the FEMA-designated floodplain on the City's property is provided in **Section 4.2.3 Jackson River Flooding Impact** .

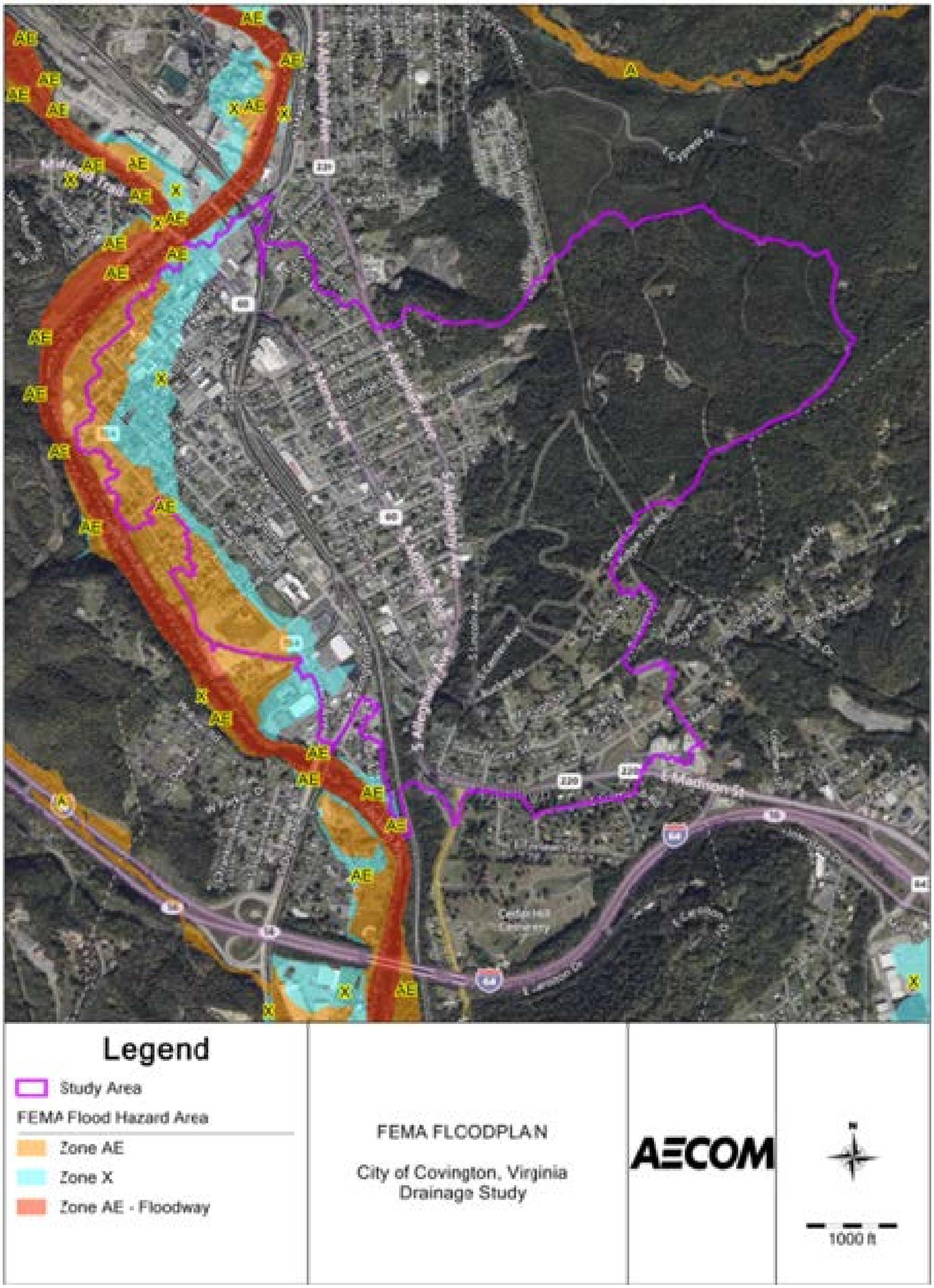

Figure 2-1: FEMA 100-Year Flood Hazard Map For Jackson River

2.2 Field Data Collection

A portion of this study involved an AECOM field investigation to collect stormwater related data and assess the current condition of the stormwater structures. The total surveyed area for this study amounted to 274 acres, with AECOM surveying 133 acres and GAS surveying 141 acres. The aerial representation of the survey areas and conduits is shown in **Figure 2-2**. The study area's collected inventory contains 110 manholes, 215 inlets, 16 outfalls, and 42,250 feet of conduits. No other hydraulic PCSWMM components were included in the model development.

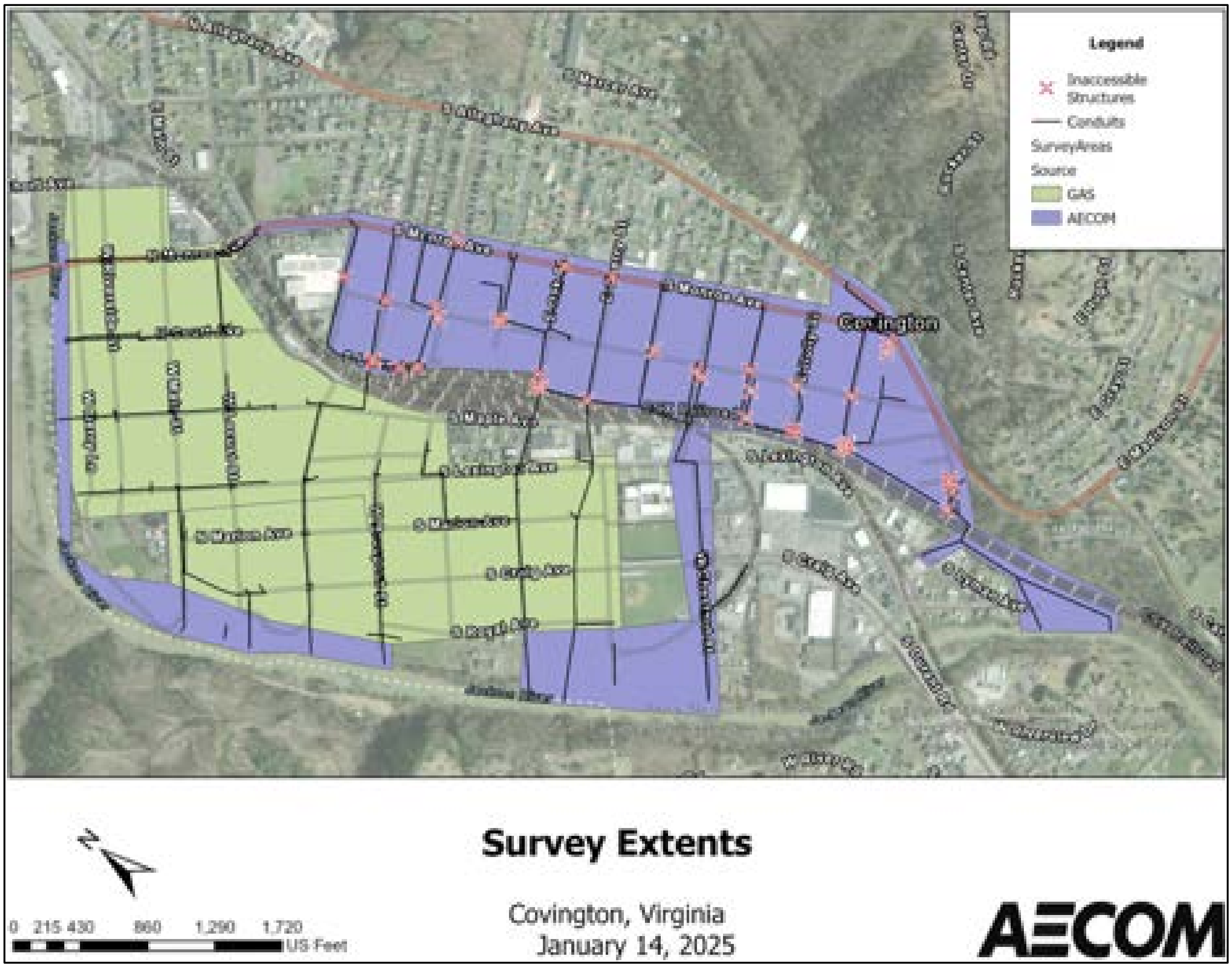

Figure 2-2: Survey Extents Map

The survey discovered that many of the stormwater structures' top covers have been paved over, preventing access to these structures (i.e. manhole covers, storm inlet covers, etc.). Due to this issue, the surveyors were unable to collect junction invert elevation, conduit invert elevation, conduit configuration, and conduit size information for the inaccessible structures. Multiple outfall invert elevations could not be collected due to overgrown vegetation and high-water elevation. Refer to **Section 3.2.9 Missing Data, Connectivity Issues, and Other Data Deficiencies** for all assumptions regarding the missing data. The field conditions and observations are subsequently incorporated into the base models, as shown in **Appendix A**.

Additionally, the stormwater infrastructure east of Monroe was not surveyed, and no data was collected for this area. As a result, the absence of surveyed inlets and network information limits the accuracy of the model, particularly where the larger subcatchments east of Monroe converge at the initial junctions. Collecting the missing data elements will improve the model's accuracy and enable a more reliable solution to address flooding throughout the City.

Analyzing the collected data, it became evident that the study area consists of missing invert elevations, missing conduit inverts, missing conduit configurations, missing conduit size, missing conduit connectivity, negative conduit slopes, conduit offsets, debris and sediment accumulation in conduits, and numerous isolated small stormwater networks where conduits and junctions don't appear to connect to any nearby network. Consequently, the small, standalone networks were deemed non-functional and excluded from the study to prevent any potential inaccuracies during the modeling process.

2.3 Data Characterization

Data Characterization involved gathering, compiling and reviewing geospatial data, aerial imagery, elevation data, Jackson River stream elevation data, land cover data, soil data, and rainfall data. No GIS data from the City was available for use in this study, which limited the scope of the analysis and potentially affected the comprehensiveness of the findings. The data type and sources used for model and map exhibit development are listed below:

- Imagery – Microsoft Bing Maps and Google Maps StreetView.
- Topographic Elevation – Light Detection and Ranging (LiDAR) tile data obtained from Virginia Geographic Information Network (VGIN).
- Stormwater Infrastructure – Field Data Collection by AECOM and GAS.
- Jackson River Stream Gauge – United States Geological Survey (USGS) Current Water Data for the Nation.
- Jackson River Floodplain – FEMA's National Flood Hazard Layer (NFHL) Flood Insurance Rate Map (FIRM)
- Soils Data – Hydrologic soils obtained from United States Department of Agriculture (USDA) web soil survey.
- Land Cover Data – Land cover LiDAR tile obtained from VGIN.
- Rainfall Data – Intensity-Duration-Frequency (IDF) obtained from National Oceanic and Atmospheric Administration (NOAA) Atlas-14.

2.4 Basin and Subbasin Delineation

The resulting drainage area to the study area is 803.96 acres, consisting of 224 subcatchments. The subcatchment parameter inputs are summarized in **Section 3.2.5 Subcatchments**. The watershed delineation was established using a watershed delineation tool in PCSWMM version 7.6.3655. The elevation data was obtained from VGIN as LiDAR datasets to determine the entire City's watershed extents. The LiDAR data was converted to a Digital Elevation Model (DEM) dataset which represents the surface topography of the study area. The resulting comprehensive map exhibit displays the basin and subbasin delineations superimposed on the DEM and is provided in **Figure 2-3.**

The watershed delineation tool used a discretization value of 5-ac to identify the smaller subbasins (subcatchments) within the DEM area to establish drainage patterns within the study area. The tool initially delineated 5,705 subbasins, which were subsequently manually adjusted based on contour elevations and aerial imagery to reflect the study area's junction locations and routing. In general, the subcatchments were routed to follow gutter flow and roadway crowns where applicable. This routing was further refined through desktop analysis using StreetView data, elevation data, and aerial imagery. The watershed extents of the study area were analyzed based on the overland flow paths determined by PCSWMM for each subcatchment.

Incorporating manual delineation to the subbasin delineation involved desktop analysis of StreetView perspective which provided visual confirmation of gutter flow and roadway crowns, while elevation data helped identify natural flow paths and potential barriers. Aerial imagery offered a broader perspective of the landscape and buildings allowing for the determination of surface water flow.

The resulting subbasins vary in size, with the eastern areas generally being larger than the western subbasins. This size difference is accounted due to the lack of infrastructure data east of South Monroe Avenue. To remedy the lack of infrastructure data, the eastern subcatchments' boundaries were joined together and routed to the nearest downstream inlet junction following the overland flow paths.

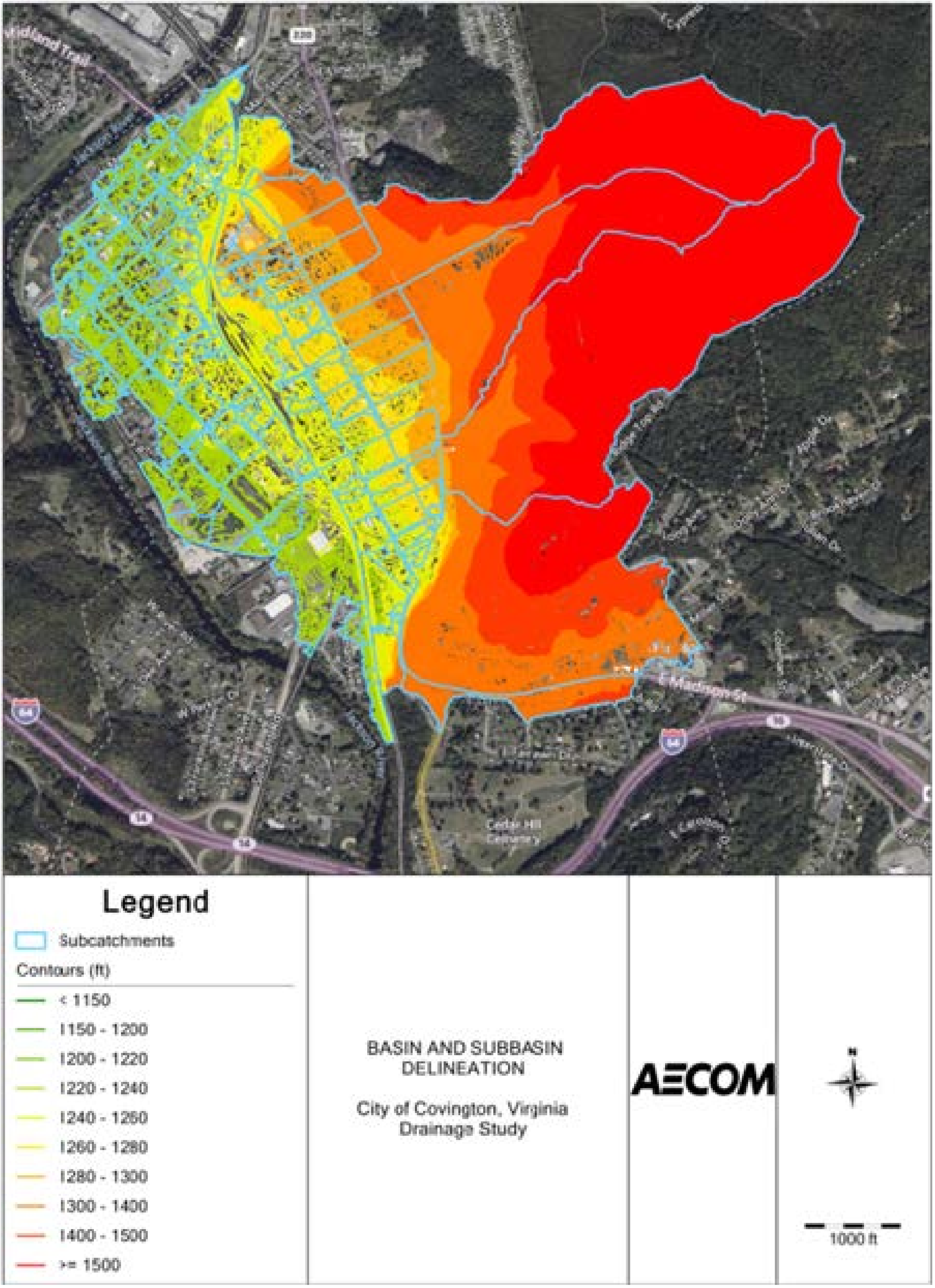

Figure 2-3: Basin And Subbasin Delineation Map

3. Hydrologic and Hydraulic Model

3.1 Stormwater Modeling Approach

The stormwater flood model for this study was created using PCSWMM version 7.7.3895 software. PCSWMM is a comprehensive software designed for stormwater and watershed modeling. The 1-D approach provides insights into the behavior of stormwater systems, allowing for precise identification of problem areas.

The 1-D PCSWMM model approach focuses on simulating the flow of water through a network of channels, pipes, and other conveyance structures in a single dimension, which is typically along the length of these structures. The model routing utilizes the dynamic wave method to accommodate complex routing conditions, including negative slopes and surcharged networks. Typically, the flow direction is governed by the elevation difference between upstream and downstream nodes in a standard non-pressurized gravity flow condition. When the hydraulic grade line (HGL) reaches the crown elevation of a conduit, the pressure computation engine reroutes the flow based on 5-second time-step calculations. Additionally, the model allows ponding over junctions, giving the computation engine a storage capacity at each network node. This represents the HGL above each structure, rather than overland flooding, thereby minimizing the system's quantity losses from the network.

In this study, the Natural Resources Conservation Service (NRCS) Curve Number (CN) methodology was selected to define the hydrologic subcatchment parameters. The NRCS method is a widely used approach in hydrology for modeling rainwater infiltration processes, developed by the United States Department of Agriculture (USDA). This methodology provides an understanding of how water moves into and through the soil, which is crucial for stormwater model predictions of surface runoff flow and losses via a simulation of rainfall-runoff processes. It includes the calculation of surface runoff, infiltration, and other hydrologic processes that affect the quantity and timing of water entering the stormwater system. Although the VDOT drainage manual recommends using the Rational Method and VDOT-adjusted precipitation tables, this study was developed using the NOAA-Atlas 14 Type C rainfall values. Therefore, the NRCS method was deemed appropriate and valid for the purposes of this study. This approach ensures that the study's objectives are met while adhering to relevant guidelines and methodologies.

Using the data collected under Phase 1 of the project and Task 1 and available LiDAR for the City, AECOM prepared the base and proposed PCSWMM models to evaluate the hydraulic performance of the stormwater network system for the 2-year, 10-year, 25-year, 50-year, and 100-year storm events using regional rainfall data and Jackson River flood level information. These PCSWMM models were used to confirm the locations where recurrent flooding events have been observed and determine if capacity limitations contribute to road flooding in the study area.

3.2 1-D Model Parameters

3.2.1 Land Cover

The land cover data analysis results show that the western section of the City is largely covered by impervious surfaces, while the eastern section features a combination of impervious surfaces and forested areas. The study site's land cover is provided in **Figure 3-1.** This pattern underscores the contrast between the two sections and suggests a more severe runoff volume conditions within the western section of the City. Land cover data utilized in this report was sourced from the VGIN database. According to the VGIN Land Cover Use Case Scenarios documentation, the dataset includes detailed classifications of various land types such as open water, impervious extracted, impervious local datasets, barren, forest, tree, shrub/scrub, harvested/disturbed, turf grass, pasture, cropland, and national wetland inventory (NWI)/other. This comprehensive dataset provides a representation of Virginia's diverse landscapes, which is essential for effective planning, environmental management, policymaking, and stormwater surface-runoff modeling.

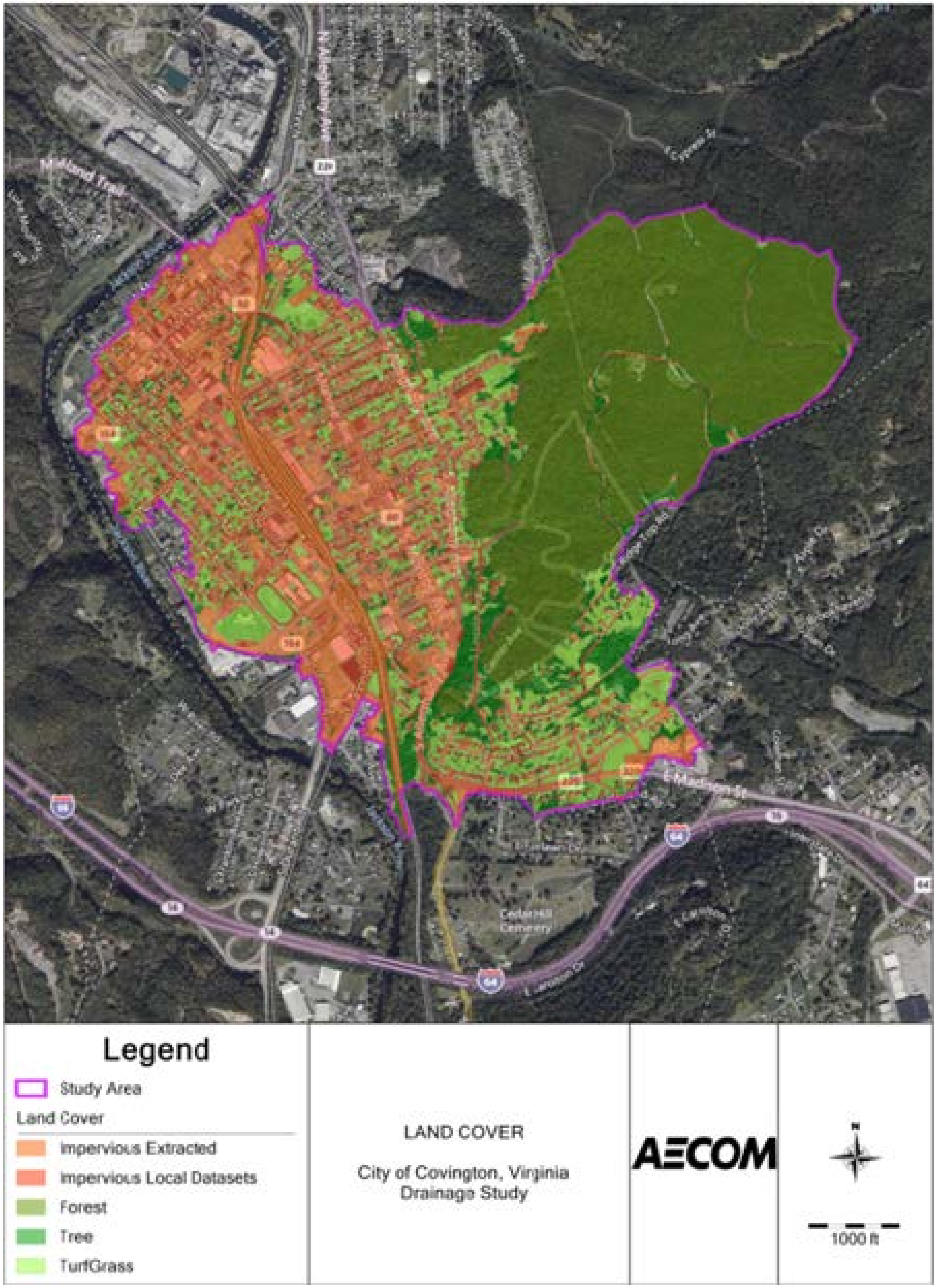

Figure 3-1: Land Cover Map

3.2.2 Hydrologic Soils

Soil data used in this study was sourced from USDA web soil survey. The web survey revealed that the site contains a wide variety of soil classifications. **Figure 3-2** presents the USDA's MUSYM-classified soil distribution map, illustrating the soil distribution throughout the study area. The map indicates the presence of 23 individual soil groups within the study area, classified using USDA's standard MUSYM identifiers. These values were converted to Hydrologic Soil Groups (HSG), as shown in the accompanying **Table 3-1**. The western section predominantly features HSG B and A classifications, while the eastern section comprises HSG D and C. These findings align with the understanding of soil distribution, where more developed land often contains outsourced fill used in urban development, whereas undeveloped areas tend to have better infiltration. This distinction is crucial for planning stormwater management and understanding the hydrological behavior of different sections of the site.

Table 3-1: HSG Classifications

MUSYM	HSG
50	D
11B	B
12B	B
12C	B
13A	C
17D	B
18E	A
22C	C
22D	C
24C	B
34C	B
39D	B
40B	D
40C	D
45D	A
47C	B
48C	B
52E	D
55C	C
55D	C
58B	D
59B	D
W	D

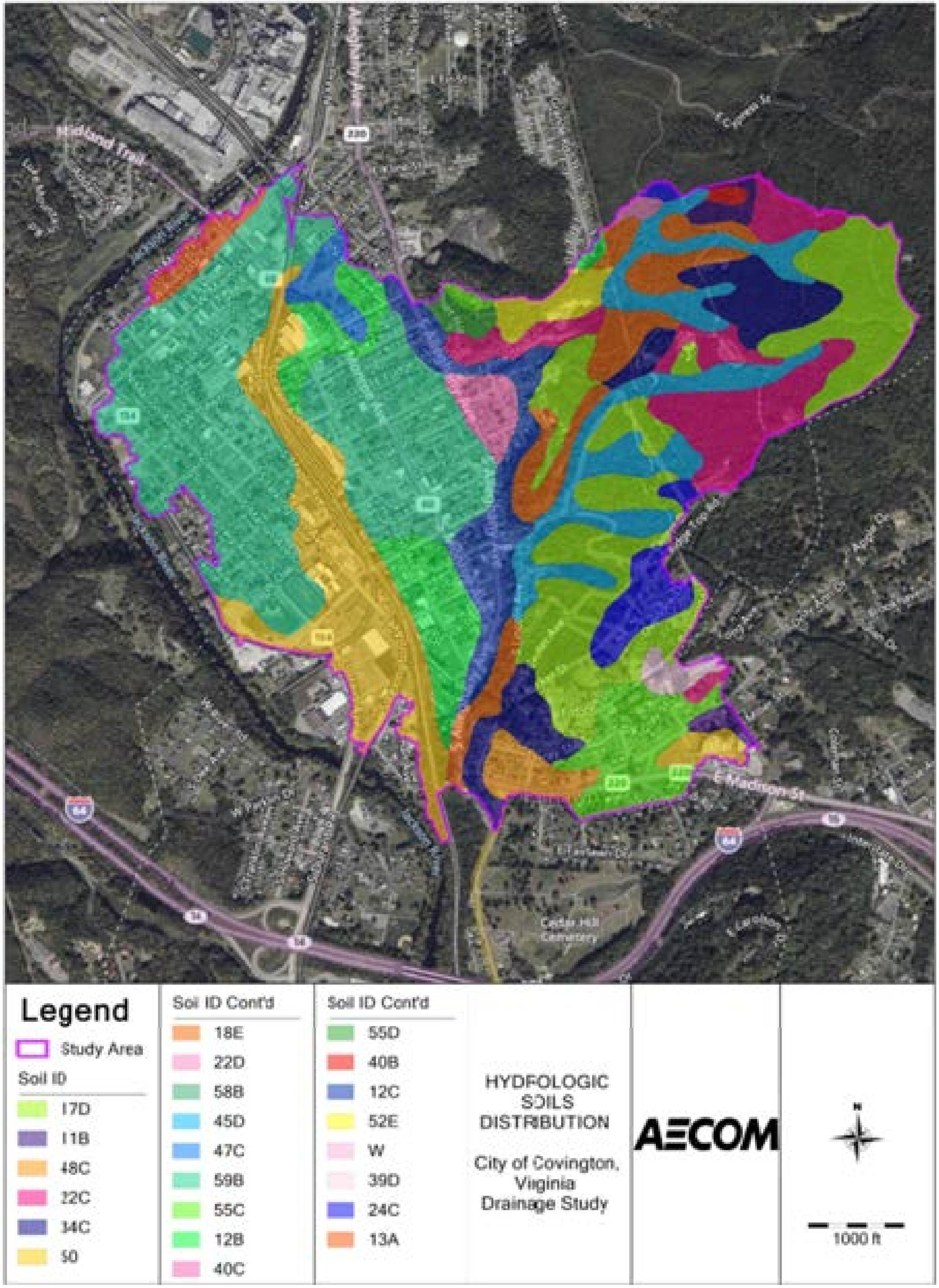

Figure 3-2: Soils Distribution Map

3.2.3 Precipitation

Rainfall data was sourced from NOAA Atlas-14 and is summarized in **Table 3-2**. The Covington, VA location contains 2 stations, Covington and Covington Filter Plant, respectively. The Covington Filter Plant station was chosen for this study given the study's extents are situated near this station. The station's ID number is recorded as 44-2044 and is located at 37.8106° latitude and -79.9883° longitude, with an altitude of 1,230-feet referenced to the horizontal datum North American Vertical Datum of 1988 (NAVD88). Data retrieved from NOAA included the precipitation frequency estimates with 90% confidence interval.

The NOAA Atlas-14 data was imported into PCSWMM's IDF curve analysis tool. The data type obtained consisted of precipitation depth in inches. The rain depth values were collected and fitted into a Type C rainfall distribution. The return periods modeled in this study were the 2-year, 10-year, 25-year, 50-year, and 100-year events, with corresponding depths of 2.74 inches, 4.04 inches, 4.88 inches, 5.58 inches, and 6.33 inches, respectively. All return periods were modeled and simulated using cumulative rainfall depth values for a 24-hour duration. For example, the graphical representation of a 100-year storm event's cumulative rainfall over the 24-hour period is displayed in **Figure 3-3**.

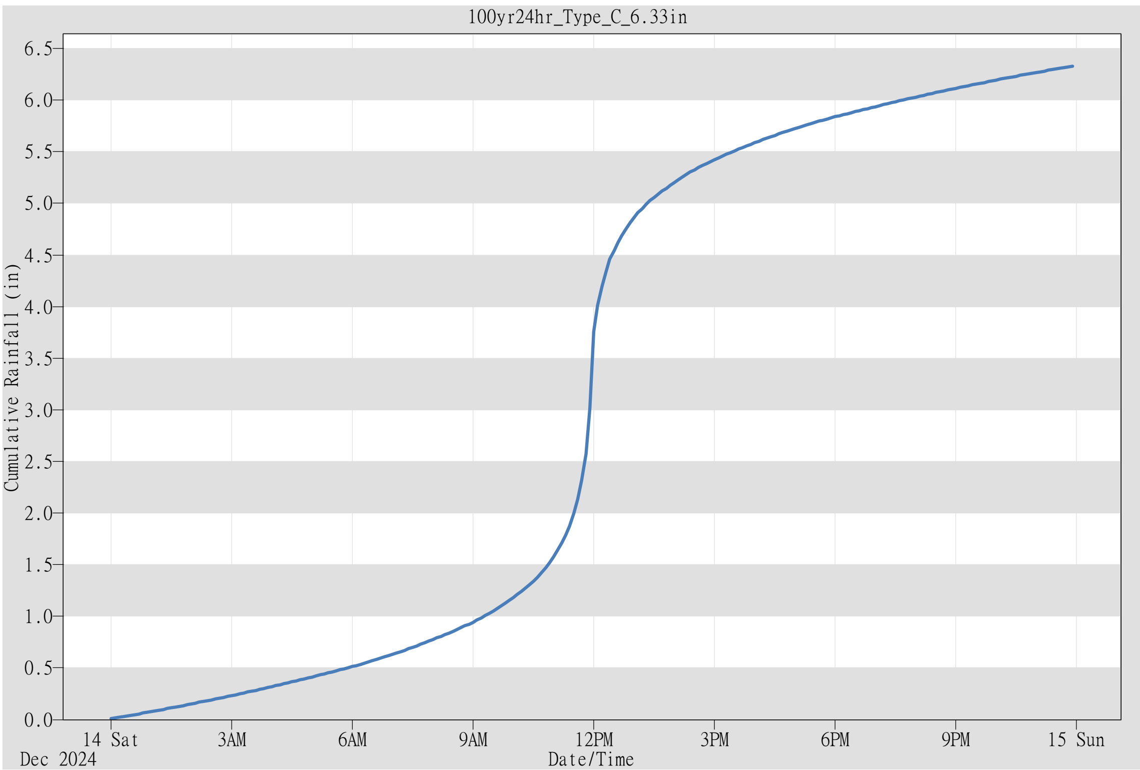

Figure 3-3: 100-Year 24-Hour Cumulative Precipitation

3.2.4 Climate Change Impact on Precipitation

An additional objective of this report is to provide flooding analysis based on future rainfall depth increases forecasting due to climate change. AECOM conducted a regional rainfall study to provide an approximate estimate of appropriate increases to rainfall data used for this study. The results of the regional rainfall study suggest that the rainfall depth will proceed to increase by approximately 10% of the current values. The complete report for the regional rainfall study is provided in Appendix B.

Modeling the estimated rainfall increase in PCSWMM, the NOAA Atlas-14 values were adjusted by approximately 10% for each of the 2-year, 10-year, 25-year, 50-year, and 100-year storm events. The results are provided in **Table 3-2**. For example, a 10% increase in the 100-year, 24-hour rainfall depth of

6.33 inches results in a total rainfall depth of 6.96 inches. For the purposes of this study, the 10-year 24-hour 10% adjusted depth storm event was modeled and provided in **Appendix E**.

Table 3-2: Forecasted Increase To Precipitation Depth

Rainfall Type	Return Period (24-hr duration)				
	2-yr	10-yr	25-yr	50-yr	100-yr
NOAA Atlas-14 Depth (in)	2.74	4.04	4.88	5.58	6.33
10% Adjusted Depth (in)	3.01	4.44	5.37	6.14	6.96

3.2.5 Subcatchments

The determination of subcatchment properties was conducted for overland sheet flow, width, average surface slope, impervious cover, Manning's n-values for both pervious and impervious surfaces, and detention storage for pervious and impervious areas. These properties were derived from various data components and sources discussed in **Section 2.3 Data Characterization** of the report. The subcatchment surface area-dependent parameters were derived by spatially weighting the individual components across the total area of each subcatchment. This approach yields a comprehensive value that accurately reflects the site conditions.

The overland sheet flow and average surface slope were computed using elevation data developed from LiDAR, utilizing the built-in tools of PCSWMM and ArcGIS Pro. The overland elevation ranged from 16.40 feet to 200.00 feet, with a mean of 183.30 feet. The study's maximum flow path length varied with distances exceeding 200 feet; however, in accordance with VDOT's drainage manual guidelines, flow paths were restricted to a maximum of 200 feet for overland sheet flow. The average surface slope percentage varied between 0.15% and 52.12%, with a mean value of 3.26%. The width property is a unique feature in PCSWMM that simplifies the computational process. It is calculated by dividing the subcatchment area by the flow path length. These subcatchment characteristics are summarized in **Table 3-3**. The subcatchments were routed to follow gutter flow and roadway crowns where applicable. This routing was developed and adjusted through desktop analysis of StreetView data, elevation data, and aerial imagery.

Table 3-3: Subcatchment Characteristics

Subbasin Property	Minimum Value	Average Value	Maximum Value
Area (ac)	0.0038	3.5891	158.3650
Flow Length (ft)	16.40	183.30	200.00
Width (ft)	10.09	785.48	34,491.90
Slope (%)	0.15	3.26	52.12

The subsequent analysis focused on determining the depth of detention storage for both pervious and impervious surfaces. The values assigned for these depths were sourced from PCSWMM's drainage manual and are detailed in **Table 3-4**. Impervious surfaces such as open water or impervious extracted were allocated a minimal depth due to their limited infiltration capacity. Conversely, pervious land cover types were assigned greater depths, reflecting their ability to facilitate water infiltration into the ground.

The n-values represent the surface roughness associated with the variable types of land cover within the study area. The roughness values were sourced from VDOT's drainage manual for shallow sheet flow. Manning's n-value summary is provided in **Table 3-5**.The impervious surface's recommended roughness value is 0.011, this value was additionally applied to barren land, harvested/disturbed, and NWI/other.

The analysis of surface cover parameters is another essential input for hydrological models like PCSWMM. It affects the simulation of various hydrological processes, including infiltration and surface runoff. The surface cover classifiers are listed in

Table 3-6. The effects of impervious and pervious surface cover are computed as a percentage of the total subcatchment area. A conservative value of 100 percent was applied to each surface type identified as having infiltration challenges.

Integrating data for HSG and land cover, the CN analysis is performed to determine the overall infiltration parameter in accordance with NRCS methodology. In PCSWMM, the CN value is utilized to calculate the cumulative infiltration parameter based on cumulative rainfall depth. To select the appropriate CN values, Win-TR 55 CN library was utilized as a reference for matching land cover types with HSGs. The selected CN values are detailed in **Table 3-7.** The resulting subcatchment maximum CN is 98, a minimum 58.57, and a mean of 93.47. These values reflect the combined effects of varying land cover and HSG.

Table 3-4: Subcatchment Detention Storage Values

Class Name	Storage (in)
Open Water	0.05
Impervious Extracted	0.05
Impervious Local Datasets	0.05
Barren	0.1
Forest	0.3
Tree	0.3
Shrub/Scrub	0.2
Harvested/Disturbed	0.1
Turf Grass	0.15
Pasture	0.2
Cropland	0.2
NWI/Other	0.1

Table 3-5: Subcatchment Manning's n-Value

Class Name	Manning’s n-Value
Open Water	0.011
Impervious Extracted	0.011
Impervious Local Datasets	0.011
Barren	0.011
Forest	0.8
Tree	0.8
Shrub/Scrub	0.4
Harvested/Disturbed	0.011
Turf Grass	0.2
Pasture	0.1
Cropland	0.1
NWI/Other	0.011

Table 3-6 Subcatchment Surface Cover Classification

Class Name	% Impervious
Open Water	100
Impervious Extracted	100
Impervious Local Datasets	100
Barren	100
Forest	0
Tree	0
Shrub/Scrub	0
Harvested/Disturbed	100
Turf Grass	0
Pasture	0
Cropland	0
NWI/Other	100

Table 3-7: Subcatchment CN Values

Class Name	Hydrologic Soil Group			
	A	B	C	D
Open Water	98	98	98	98
Impervious Extracted	98	98	98	98
Impervious Local Datasets	98	98	98	98
Barren	77	86	91	94
Forest	36	60	73	79
Tree	36	60	73	79
Shrub/Scrub	35	56	70	77
Harvested/Disturbed	72	78	85	89
Turf Grass	49	69	79	84
Pasture	49	69	79	84
Cropland	72	78	85	89
NWI/Other	32	54	68	75

3.2.6 Junctions

The junctions within the PSCWMM stormwater network represent inlets, manholes, and junction boxes. The base model includes 325 junctions, comprising of 215 inlets and 110 manholes. These junctions are essential for connecting different elements of the stormwater management system, such as underground or overland conduits. The key inputs for modeling these junctions are invert elevation, rim elevation, and ponded area. The junction 1-D model component of ponded area was modeled with a total of 10,000-square feet, while the surcharge depth and initial depth were set to 0-feet, respectively. The ponded area parameter provides a storage area to allow the model's computations to contain the overflow within the flooding junction to allow the excess to be reintroduced back into the junction as the downstream conduit HGL decreases over the duration of a storm event.

3.2.7 Conduits

Conduits in the PCSWMM model represent the various stormwater conveyance elements that direct the incoming runoff inflow to the outfall. The conduit properties consists of length, cross-section, slope, roughness, energy losses, and offsets which determine water flow. To supplement the study's conduit setup, a table detailing various maintenance items along the CSX railroad was summarized in **Table 3-8**. The major findings include debris and sediment accumulation within conduits reducing capacity and conveyance efficiency. The PCSWMM parameter modified for this maintenance item included applying the percentage of fill identified in photos, as well as increasing the roughness of conduit to a poor condition. The base model consists of a total of 328 conduits for a total of 42,250 feet. The modeled cross-sections include 298 circular conduits totaling 36,605 feet, 7 circular conduits with fill of approximately 942.2 feet, 5 rectangular closed conduits totaling 407.15 feet, 1 modified basket handle conduit of approximately 162.9 feet, and 17 triangular conduits totaling 4,132 feet.

Table 3-8: Maintenance Items Along CSX Railway

Location	Maintenance Item	Conduit Modification Description
Inlet/Ditch near S Lawn Ave and E Prospect St.	Inlet full of riprap	Junction and conduit removed from the model due to missing connecting links
Cherry St and CSX	Debris and sediment accumulation	Applied 97% fill and increased roughness of conduit C34.
Cherry St and CSX	Concrete culvert partially full of sediment	Applied 90% fill and increased roughness of conduit C94.
Cherry St and CSX	Debris and sediment accumulation	Applied fill and increased roughness to conduit C94. Even though the entire conduit is not at 90% fill height as shown in upstream picture, this is still modeled as 90% fill as the conduit would need to fill up and hold water until it reaches the invert elevation of the downstream ditch.
E Chestnut St and CSX	Debris and sediment accumulation	Applied 80% fill and increased roughness of conduit C132.
S Fitzgerald Ave and Spruce St	Debris and sediment accumulation, overgrown vegetation	Lowered the total depth and width by 50% and increased roughness of conduits: C63, C64, C65, C77, C79, C114, C115.
CSX and Alleghany. Near school bus mechanic.	Debris and sediment accumulation, overgrown vegetation	Lowered the total depth and width by 50% and increased roughness of conduits: C109, C110, C111
CSX and Alleghany	Debris and sediment accumulation	Applied 25% fill and increased roughness of conduit: C86
CSX and S Lyman Ave	36”x48” Brink Culvert is partially blocked	Lowered the total depth by 10% for pipe fill and increased roughness of conduit C136.
E Oak St	Debris and sediment accumulation	Applied 97% fill and increased roughness of conduit C34.
S Monroe Ave	Debris and sediment accumulation	Applied 40% fill and increased roughness of conduit C10.
Chestnut St	Debris and sediment accumulation	Applied 25% fill and increased roughness of conduit C128.
W Chestnut St	Debris and sediment accumulation	Applied 25% fill and increased roughness of conduit C112.

The cross-section data was primarily modeled based on the collected survey data. During the modeling process, instances of overland channels were identified and incorporated into the model. Since the geometry and elevation of these overland channels were not surveyed, the data was derived from the DEM and approximately estimated for each section of the observed channel.

The study area consists of various conduit materials listed in **Table 3-9** for a total of 15 types. The table outlines the abbreviations used in the tag parameter for each conduit and provides the roughness values for both good and poor conditions. A good condition indicates no debris or sediment accumulation, while a poor condition signifies the presence of fill or overgrown vegetation within the conduit. Additionally, the conduits on the site appear to have been installed at different times or repaired with different material, leading to inconsistencies between sections. For example, conduit 3070D04 receives flow from an upstream conduit made of Corrugated Polyethylene (CPP), transitions to a Polymer Coated Corrugated Steel Double Wall SP(SI) in conduit 3070D04, and then reverts back to CPP in conduit 3071E01. While material variability is not uncommon at older sites, the differing roughness values between these materials can lead to unnecessary disruptions in flow conveyance. The conduit roughness values were sourced from VDOT's drainage manual.

Table 3-9: Conduit Roughness

Conduit Type	Abbreviations	Roughness Good Condition	Roughness Poor Condition
Piped Conduits			
Reinforced Concrete Pipe	RCP	0.013	0.015
Reinforced Concrete Box	RCBC	0.013	0.015
Corrugated Steel Pipe Fully Concrete Lined	CP	0.013	0.015
Polymer Coated Corrugated Steel Double Wall (Smooth Interior)	SP(SI)	0.013	0.015
Corrugated Metal Pipe – Steel, Aluminum or Polyethylene	CMP/CPP	0.024	0.03
Clay / Terracotta	C/T	0.013	0.015
Polyvinyl Chloride	PVC	0.011	0.013
Channels Not Maintained, Weeds And Brush Uncut			
Concrete Lined	C-C	0.013	0.015
Dense weeds, high as flow depth	C-DW	0.08	0.12
Grass, some weeds	C-GW	0.03	0.033
Excavated or Dredged			
With short grass, few weeds	E-G	0.027	0.033
Brush			
Scattered brush, heavy weeds	B-H	0.05	0.07
Medium to dense brush, in summer	B-M	0.1	0.16
Light brush and trees, in summer	B-L	0.06	0.08

To further improve the base model, the total energy losses (H_t) were computed following VDOT Drainage Manual, Chapter 9: Storm Drains (Chapter 9). The study's total junction losses were only computed for the closed conduits, as the open channels were considered to not experience the same losses as junctions. The **Equation 3-1** summarizes the energy losses computation which is an additive representation of all the conduit energy losses modeled in this study.

$$H_t = H_i + H_\Delta + H_o$$

Equation 3-1: Total Junction Losses

- The entrance loss (H_i) was modeled with a coefficient of 0.35 and populated as entry loss coefficient, while initial inlet of a system was attributed with a coefficient of 0.3.
- The exit loss (H_o) was modeled with a coefficient of 0.25 at each upstream conduit approaching an outfall.
- The bend loss (H_Δ) and H_o were grouped together in the model's conduit parameter of exit loss coefficient by adding the PCSWMM calculated K Factor value and the exit loss coefficient for each individual conduit.

H_Δ was included in the model to include the energy losses associated with the angle of interaction between the junction to conduit configuration. Bend losses were applied to junctions in which the outgoing conduit is at an angle greater than 0 degrees to the incoming conduit. This method was applied to all conduits within the 1-D model via the built-in PCSWMM tool and populated in the exit loss coefficient attribute. The losses shown in **Table 3-10** were derived from the K factor. These bend loss values were extracted from the VDOT Chapter 9 graph entitled, *"Losses in Junction Due to Change in Direction of Flow Lateral"* indicating a maximum value of 0.70 and minimum value of 0.00 The losses were then populated within the entry and exit loss coefficient field in PCSWMM, where the H_Δ + H_o sum was inserted into the exit loss coefficient field.

Table 3-10: Energy Losses Due To Conduit Angle

Angle°	K Factor (unitless)
0	0.00
10	0.13
30	0.35
40	0.43
50	0.50
60	0.61
90	0.70

3.2.8 Outfalls

The end section that represents the discharge point from the stormwater network system consists of the outfall component. The base model is constructed with 16 outfalls along Jackson River's floodway, as shown in **Figure 4-7**. The City's outfalls discharge the various networks' runoff waters, directing them into Jackson River. The modified outfall parameters in this study include the invert elevation, rim elevation, and tail waiter conditions. The survey of the study area was unable to reach certain outfalls, which are documented in **Table 3-12**. The invert elevations were collected from the DEM for the instances where missing data was observed.

To determine the appropriate tailwater elevation for the base model analysis, data from the Jackson River gauge, specifically the USGS stream gauge named Jackson River BL Dunlap Creek at Covington, VA, was utilized. Additionally, NOAA's national water prediction service leverages the USGS stream gauge records to provide a comprehensive summary of Jackon River's various depth and elevation stages. These data, accessed on December 5th, 2024 is summarized in **Table 3-11**. The Action stage was designated as the tailwater condition because it signifies the onset of overland flooding, prompting residents to initiate their evacuation routes.

The key findings from the stream gauge data indicated six distinct types of Jackson River water surface elevation (WSE). The types of Major Flooding, Moderate Flooding, and Minor Flooding are considered as more severe flood conditions leading to the WSE breaching into the low-lying elevation areas near the river banks and even further inland. Action flood stage is the early sign of a flood and serves as a warning to the residents to take preparatory actions to mitigate potential damage and ensure safety. The Latest Value symbolizes a most recent WSE dated December 5th, 2024 and doesn't provide any additional information. Gauge Zero elevation represents the elevation of the gauge referenced to the NAVD88 datum. By monitoring and responding at the action stage, authorities and residents can implement measures such as reinforcing flood defenses, preparing evacuation plans, and securing property.

Table 3-11: Jackson River Flood Stages

Type	Elevation (ft)	Depth (ft)
Major Flooding	1228.94	23
Moderate Flooding	1225.94	20
Minor Flooding	1222.94	17
Action	1219.94	14
Latest Value (December 5, 2024)	1210.55	4.61
Gauge Zero	1205.94	0

3.2.9 Missing Data, Connectivity Issues, and Other Data Deficiencies

AECOM's field investigation, documented in **Section 2.2 Field Data Collection**, summarizes the issues identified during the analysis of the collected survey data. The findings revealed a variety of issues within the study area. Consequently, the modeling efforts required assumptions to be made on a case-by-case basis.

The study area consisted of many paved over structures that limited the access to collect adequate junction invert and rim elevation data, hence the approach to remedy this issue consisted of utilizing a PCSWMM built-in tool to gather the surface elevation for rims, channel invert elevations for open channel conveyances, and invert elevations for outfalls. The tool approximates the elevation by reading the DEM data and assigning the appropriate value. A typical output of the tool is displayed with three decimal places, as shown in **Table 3-12** for the estimated outfall invert elevations. It is important to note that while this method may yield less accurate elevation data compared to a field survey, it remains the most suitable approach for addressing the absence of missing elevation information for the various 1-D model parameters.

Table 3-12: Typical DEM Approximated Elevation Results

Node	Invert Elevation (ft)
DS_306F9	1211.099
DS_30707	1223.336
DS_3071906	1214.737
OF_518D8	1222.795

OF6	1202.508
OF7	1202.423
OF8	1207.085

Additional assumptions were made in the model to address missing conduits. This issue was critical because the upstream network converges at a junction without evidence of overland relief. For instance, at junction 1676C, incoming flows from three separate trunklines converge. Without a connecting downstream conduit between junctions 1676C and 7D87D, major flooding would occur. A StreetView review of the area, revealed no obvious pipe outlet. Therefore, the missing link was added, as illustrated in **Figure 3-4** and **Figure 3-5**. In instances where the missing link is added in to the model, the size and roughness of the conduit is assumed to copy the upstream conduit parameters.

Figure 3-4: Typical Missing Stormwater Network Link

Figure 3-5: Typical Added-In Stormwater Network Link

To address the issue of missing junction invert elevations, an interpolation approach was deemed most effective for determining the bottom elevation. This method involved using the upstream or downstream junction elevation, the conduit length, and the slope between these structures. In cases where a single trunkline contained multiple junctions with missing elevations, the slope of the first upstream conduit was used to compute the downstream invert elevation.

In cases where missing conduit size, or conduit material was missing, the largest upstream converging conduit was assumed to match the downstream conduit parameters.

The Chestnut underpass was among the missing data points. Due to the nature of LiDAR data collection, the DEM could not provide an approximate elevation of the underpass inlet's rim. However, StreetView observations revealed that this underpass has a clearance of 12-feet and 3-inches. For the purposes of this study, this depth is subtracted from the DEM's identified surface elevation and applied to these junctions. The typical depth of such junctions was assumed to be 5-feet.

The methods outlined in this section are designed to address obstacles related to model setup and should be applied with caution. It is highly recommended to validate the assumptions made, as they may not accurately represent the actual operation of the existing stormwater network. Junction invert elevations, conduit invert elevations, conduit configurations, conduit sizes, and outfall invert elevations have not been uniformly accounted for and require a more thorough verification. Additionally, the stormwater network east of Monroe has not been surveyed and needs to be identified in the field. Refer to **Figure 2-2** for the survey records conducted for this study.

4. Existing Conditions Flood Assessment

4.1 Existing Flood Assessment Narrative

The City currently lacks adequate preparedness to address flooding issues at the public road underpasses that intersect with the CSX railway. This deficiency poses a significant risk, as these underpasses are critical points in the transportation infrastructure. The City has two separate underpasses that intersect with the CSX railway tracks: Chestnut and Monroe.

These two underpasses are crucial for roadway traffic, serving as entry and exit points to access the east and west sections of the City. Consequently, flooding at these underpasses poses a significant risk to public health and safety, as it can disrupt traffic flow and potentially endanger lives during even the 2-year 24-hour storm event. It is essential for the City to implement effective flood management strategies to mitigate these risks and ensure the safety of its residents. The flooding of the Chestnut underpass is illustrated in Figure 2-4, which depicts a storm event that occurred around July and August. The most severe storm within this timeline occurred on August 9th, with a total cumulative rainfall of 3.15 inches, as recorded by the nearest rainfall gauge. This storm event demonstrates the impact to the Chestnut underpass and the scale of inundation that occurs along this road. The high water elevation obstructed access between the west and east section of the City, endangering the health and safety of residents and visitors. This event highlights the severe consequences of extreme weather on community infrastructure and the importance of effective flood management systems.

Figure 4-1: Flooding Of E Chestnut Street Underpass Dated August 2024

4.2 Base Model Flooding Results and Analysis

The base model results are provided in **Table 4-1** for the return period of the 2-year, 10-year, 25-year, 50-year, and 100-year 24-hour storm events. The results are presented in the context of the total number of flooded junctions compared to the total junctions' inventory collected for this study of the City's stormwater network. The base model map exhibits show the impacts of flooding on the stormwater network for each storm event which are provided in **Appendix C**. The model results indicate that even a minor 2-year 24-hour return period poses significant challenges for the study area, with 158 out of 325 junctions experiencing flooding, accounting for approximately 49% of the City's inventory. All subsequent storm events, including the 100-year storm event, contribute to an increase in the total number of flooded junctions at varying rates. During a more severe storm event of 100-year 24-hours, 221 out of 325 junctions flood, representing 68% of the total inventory which is only a 7% increase from the 10-year 24-hour storm event. These findings underscore the urgent need for substantial improvements to the stormwater networks.

Table 4-1 Base Model Flooded Junctions Summary

Scenario	Return Period				
	2-yr	10-yr	25-yr	50-yr	100-yr
Base Model	158	197	210	214	221
Percent of Total Inventory Flooded	49%	61%	65%	66%	68%

4.2.1 Chestnut Existing Stormwater Network Analysis and Results

To further discuss the objectives of this study, an analysis of the Chestnut trunkline reveals that this existing network experiences debris/fill in conduits, negatives slopes, choke points, and outfall location within the Action flood stage. These existing deficiencies are shown in **Figure 4-3**. The Chestnut trunkline is located along E Chestnut St and W Chestnut St, crossing under the CSX railroad. Given the current adverse slope of the trunkline just downstream of the underpass, it is evident that the incoming collected surface runoff will cause flooding during 2-year 24-hour storm event. It should be noted that conduit C72 geometry was assumed as a triangular section to reflect the undefined drainage path towards the underpass.

A Choke point (or bottleneck) in stormwater networks can substantially reduce water flow, resulting in increased flooding and poor conveyance capacity of conduits. Within the Chestnut stormwater network, these choke points were identified at three conduits (C112, C128, and C132). This condition was determined by identifying conduits where the trunkline diameter transitions from larger to smaller, creating bottlenecks that restrict conduit conveyance. Addressing these choke points generally leads to improvement in conveyance of the stormwater network.

The negative slopes of this network system are shown in **Figure 4-3**. The seven adverse conduits (C20, C21, C112, C128, C127, C131, and C132) disrupt the flow and significantly contribute to the system's overall inefficiency due to standing water. The underpass configuration is particularly problematic because the invert elevation at the underpass junction 2408_X_12 is set at 1,225.0 feet, while the second downstream junction 2408_X_15 has an invert elevation of 1,239.5 feet. Consequently, this network configuration forces the HGL to rise by 14.5 feet at the underpass junction before any discharge can occur. Additionally, it appears that the negative slopes resulted in conduit fill accumulation, as multiple conduits were observed to have both conditions, further reducing this network's conveyance capacity.

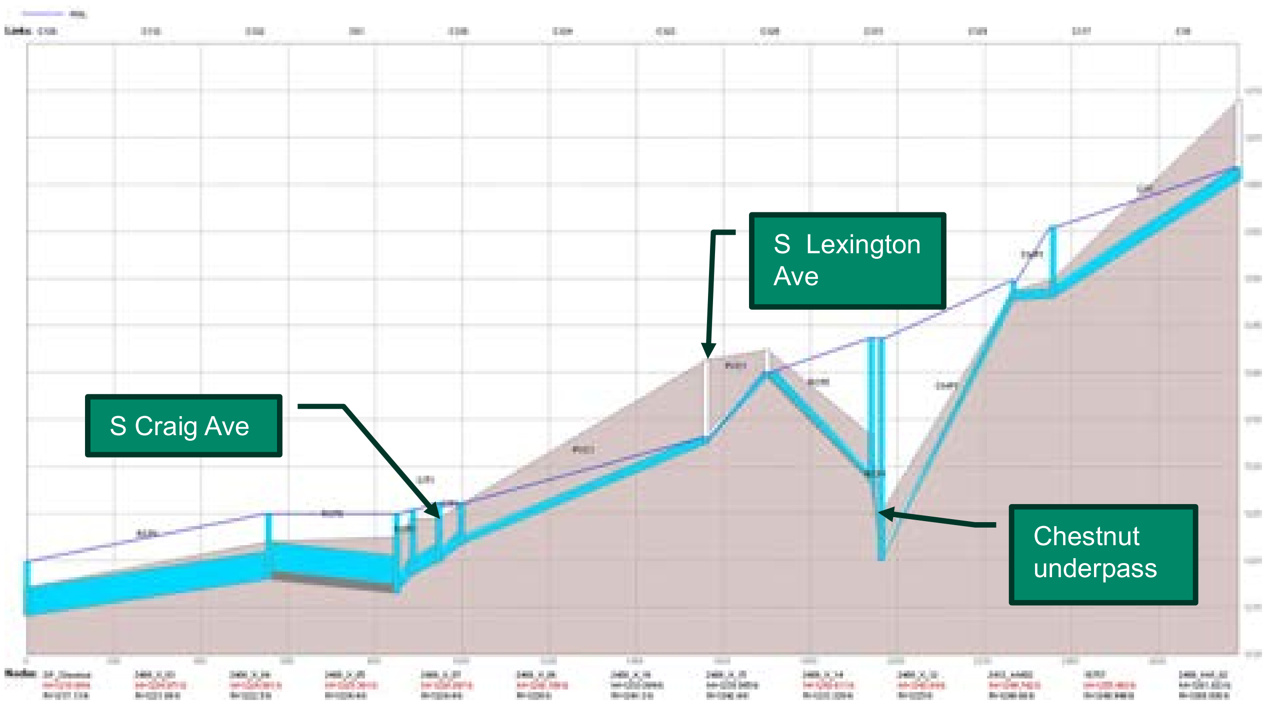

Figure 4-2: 100-yr 24-hr Existing Chestnut Network Profile

Fill in conduits can significantly reduce the conveyance capacity of a stormwater network, leading to increased flooding and decreased efficiency in water flow management. **Figure 4-3** shows the location of this occurrence within the Chestnut stormwater network, where three conduits (C112, C128, C132) have been identified. These conduits, compromised by sediment or debris accumulation, restrict the flow of stormwater, causing backups and potential overflow storm events.

The Chestnut network system's outfall invert elevation is located at 1,214.13 feet, which discharges into Jackson River below the Action flood stage elevation by 5.81 feet. This is problematic as the River will begin to trespass into the network and reduce the trunkline's storage capacity. Typically, this issue may be eliminated by either relocating the outfall further upland where the elevations are higher in grade, or by implementing tide gates at the pipe ends.

The 100-year, 24-hour storm event is illustrated in the **Figure 4-2** to demonstrate the impacts on the network under the VDOT guideline for underpasses. During this storm event the HGL at the underpass was computed to reach an elevation of 1,242.26 feet, while the top of the underpass junction is approximately 1,225.0 feet according to the DEM, suggesting an HGL overflow of roughly 17.26 feet. Other observations during the 100-year storm event include flooding of nearly all junctions within this network, while only 2 of 9 junctions downstream of the underpass don't flood. Reviewing the 100-year HGL in the, the effects of flooding appear to be reasonable; however, it should be noted that the July, 2024 storm event was not specifically modeled, as described in **Section 3.2.3 Precipitation**.

The analysis of the network suggests that it operates in an inefficient manner, highlighting the need for significant improvements. These improvements are necessary to fix the network's performance, increase its conveyance capacity, and provide safe passage at the underpass. By addressing these inefficiencies, the network will be better equipped to manage surface runoff and reduce the risk of flooding during storm events.

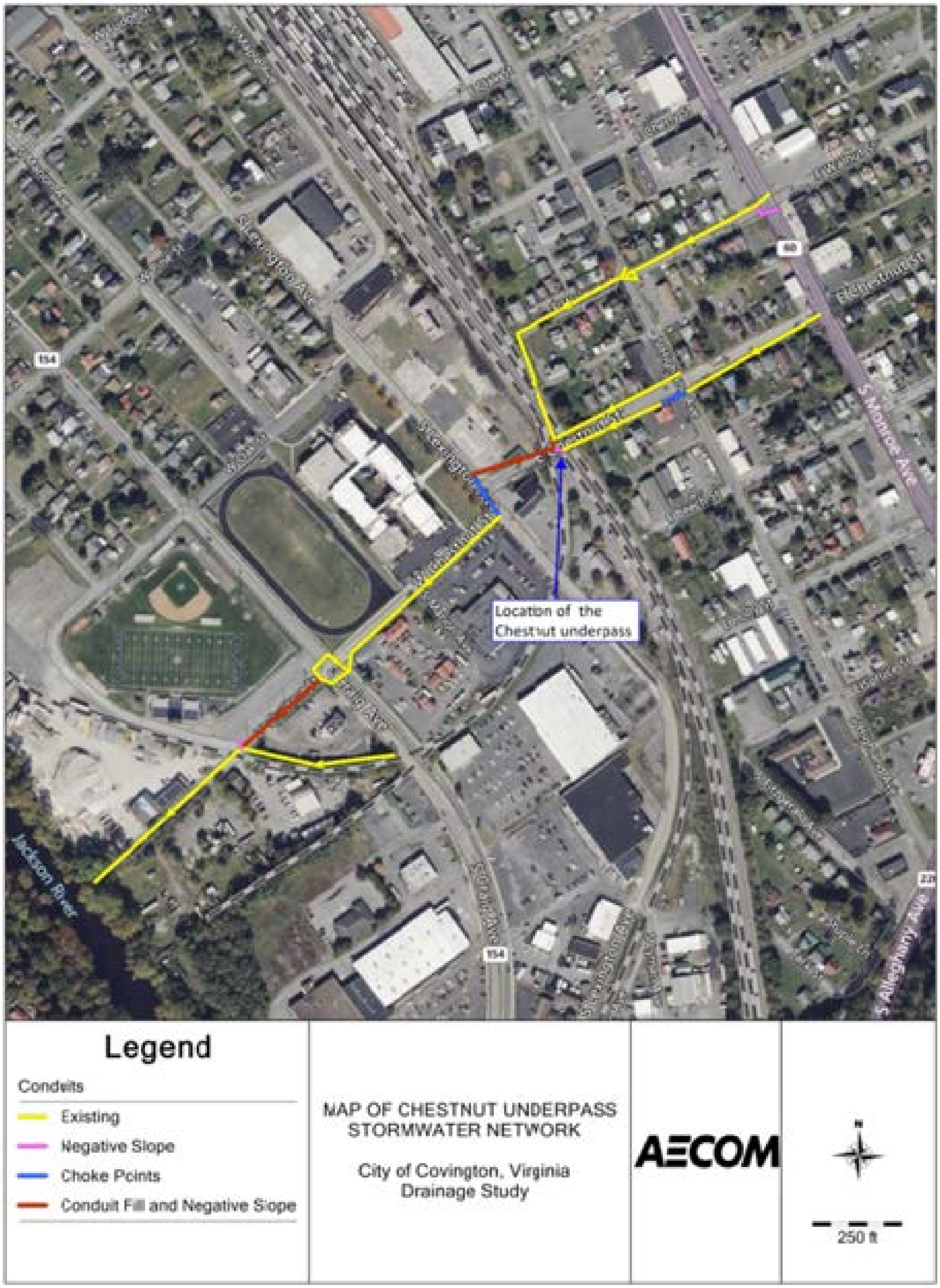

Figure 4-3: Chestnut Existing Stormwater Network Map

4.2.2 Monroe Existing Stormwater Network Analysis and Results

The Monroe network is composed of two distinct trunklines. The network diverges into the two trunklines at junction 7D76D, just downstream of the network's underpass and negative slope conduit, as illustrated in **Figure 4-6**. This network is located at the intersection of N Monroe Ave and CSX railroad. The network appears to contain conduit issues such as negative slopes and choke points. The initial observation is that the conduit downstream of the underpass has an adverse slope, inhibiting positive drainage. Additionally, the two trunklines operate in different stages due to a depth offset. Trunkline #1 is considered the primary trunkline due to Trunkline #2 having a conduit invert 1.13 feet above Trunkline #1's invert. Trunkline #2 begins to receive runoff only after the HGL of Trunkline #1 rises to the offset height. In the case of the Monroe underpass, this divergence is beneficial for the overall conveyance of runoff due to the simultaneous operation of both trunklines, improving the system's capacity.

The Monroe network features negatively sloped conduits (518A0, 5189A02, 519C3, 519C0) that significantly reduce its conveyance capacity due to standing water, thereby affecting the efficiency of the two trunklines within the network. The first adverse slope is located at the conduit (519C0) just downstream of the underpass, where the network remains a single trunkline. The conduit presents significant challenges due to the invert elevation at the underpass junction 7D7D6 being 1,233.30 feet, while the downstream junction 7D76D is at 1,234.33 feet, resulting in a 1.03 feet elevation difference. Consequently, during normal system operations, the HGL must rise by 1.03 feet before any discharge can occur from the underpass. This situation is particularly problematic because the underpass conduit is only 24 inches in diameter, leaving just less than a 12 inches of effective depth to manage the incoming runoff. Trunkline #2 includes two adversely sloped conduits (518A0 and 5189A02), while Trunkline #1 does not contain any further adverse slopes. Trunkline #1 profile is provided in **Figure 4-4**, while Trunkline #2 is provided in Error! Reference source not found., respectively.

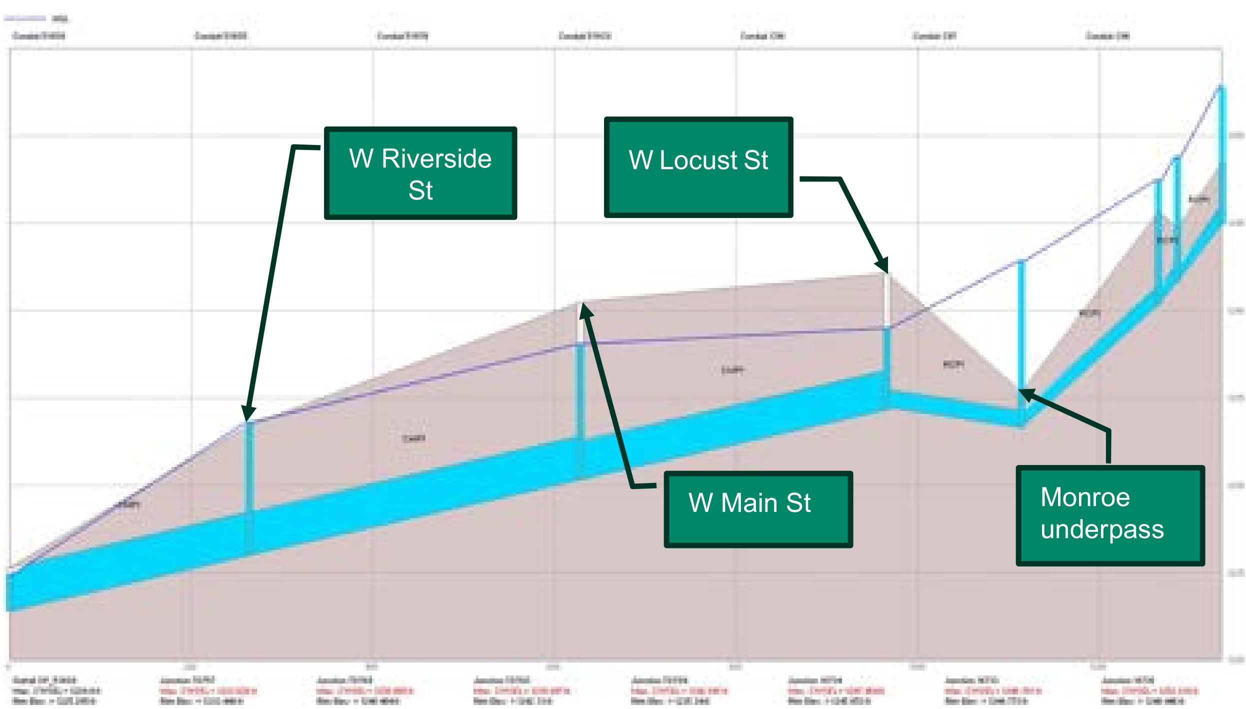

Figure 4-4: 100-yr 24-hr Existing Monroe Trunkline #1 Profile

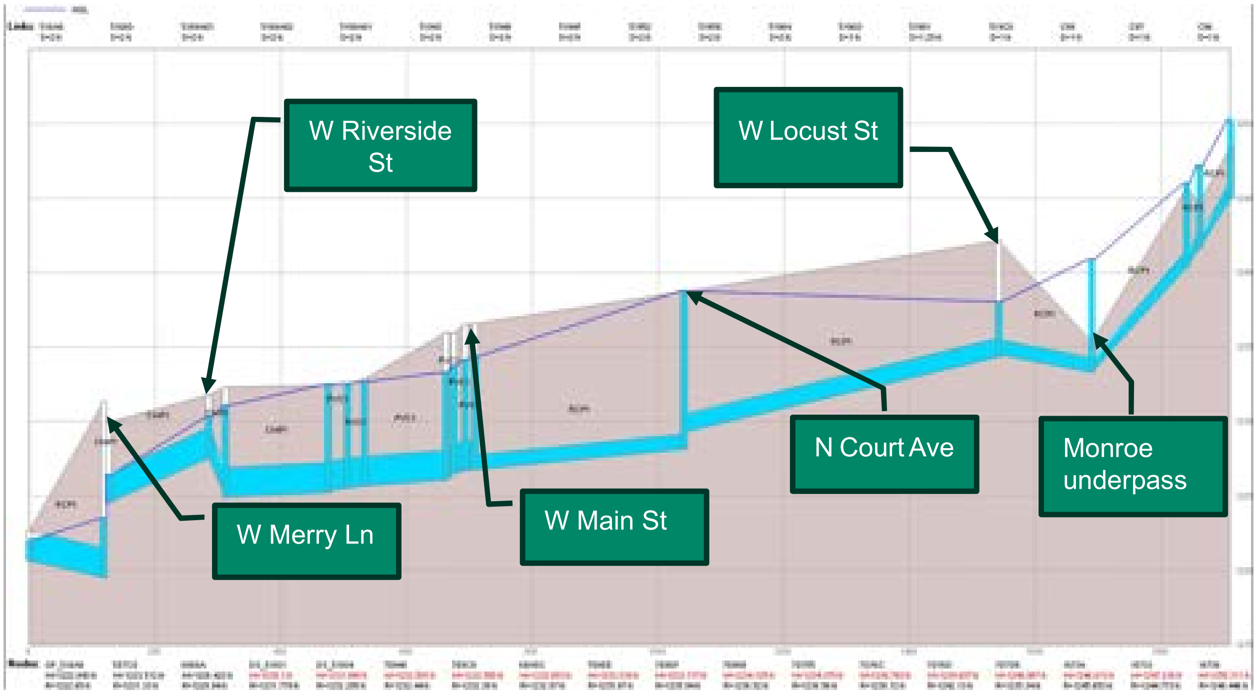

Figure 4-5: 100-yr 24-hr Existing Monroe Trunkline #2 Profile

Trunkline #2 contains a single choke point conduit (5196D) located downstream of junction 7D76C, as shown in **Figure 4-6**. This conduit has a reduced diameter of 12 inches compared to 15-inch diameter just upstream. Despite the minor reduction in diameter, it creates a bottleneck effect on the network, adversely affecting the conveyance capacity and efficiency of Trunkline #2. Downstream of the 12-inch diameter conduit, the trunkline increases to a 24-inch diameter and has no further choke points.

The 100-year, 24-hour storm event is provided for Trunkline #1 and Trunkline #2 in **Figure 4-4** and **Figure 4-5**, respectively. The Trunkline #1 flooding is observed to surcharge entirely with instances of flooding at 1 of 3 junctions downstream of the underpass. The Trunkline #2 is observed to surcharge throughout as well, while 4 of 13 junctions flood downstream of the underpass.

Trunklines #1 and #2 discharge into Jackson River at elevations of 1222.795 feet and 1220.65 feet, respectively, which are above the Action flood stage of 1219.94 feet. This observation suggests that the negative slopes are not necessarily needed at the most downstream conduits, however it is a generally good practice to provide such configuration to prevent the tailwater backflow into the stormwater network.

Analysis of the Monroe network, similar to the Chestnut stormwater network, indicates a failure to meet VDOT's underpass drainage guidelines. Improvements are required to provide larger conveyance capacity and ensure safe passage at the underpass. These improvements are crucial for preventing flooding and maintaining the infrastructure's integrity. Resolving these deficiencies will also help the network comply with regulations and improve overall performance.

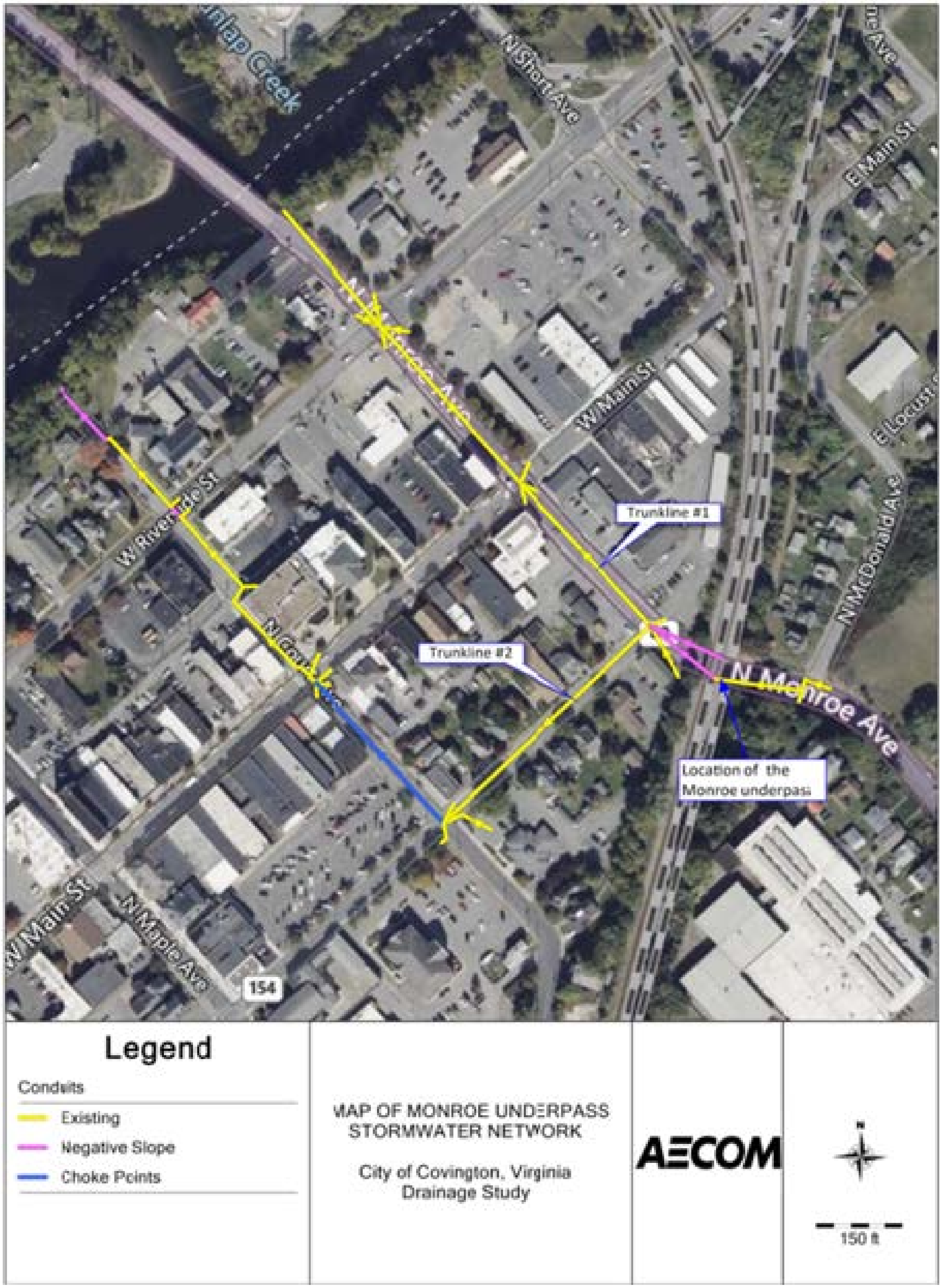

Figure 4-6: Monroe Existing Stormwater Network Map

4.2.3 Jackson River Flooding Impact Analysis and Results

Analysis of the Jackson River's FEMA floodplain, as depicted in **Figure 2-1**, reveals significant encroachment of the floodplain into the City's land areas, which raises concerns of fluvial flooding. The City is situated within flood zones AE and X, as detailed in Section **2.1 Watershed Description**. An assessment of the City's infrastructure was conducted to determine the number of structures inundated or impacted by fluvial flooding.

The results, illustrated in **Figure 4-7**, indicate inundation of stormwater junctions adjacent to Jackson River. Out of a total of 325 stormwater network nodes, 122 nodes could be affected, which represents 38% of the total. Specifically, Flood Zone X, representing the 0.2% annual chance flood, may impact up to 44 junctions, while Flood Zone AE may affect up to 78 junctions. This means that approximately 14% of the total inventory would be impacted within Flood Zone X, and 24% when Flood Zone AE is active.

Moreover, all City outfalls are situated within the AE flood zone, resulting in all 16 outfalls being affected. This widespread impact on the outfalls further exacerbates the flooding issues, highlighting the urgent need for comprehensive mitigation measures to protect the city's infrastructure and ensure effective drainage. Planning and mitigation strategies are essential to investigate whether fluvial flooding exacerbates the flooding within the City's stormwater networks. Additionally, an analysis of backflow prevention measures is necessary to determine their effectiveness within the study area.

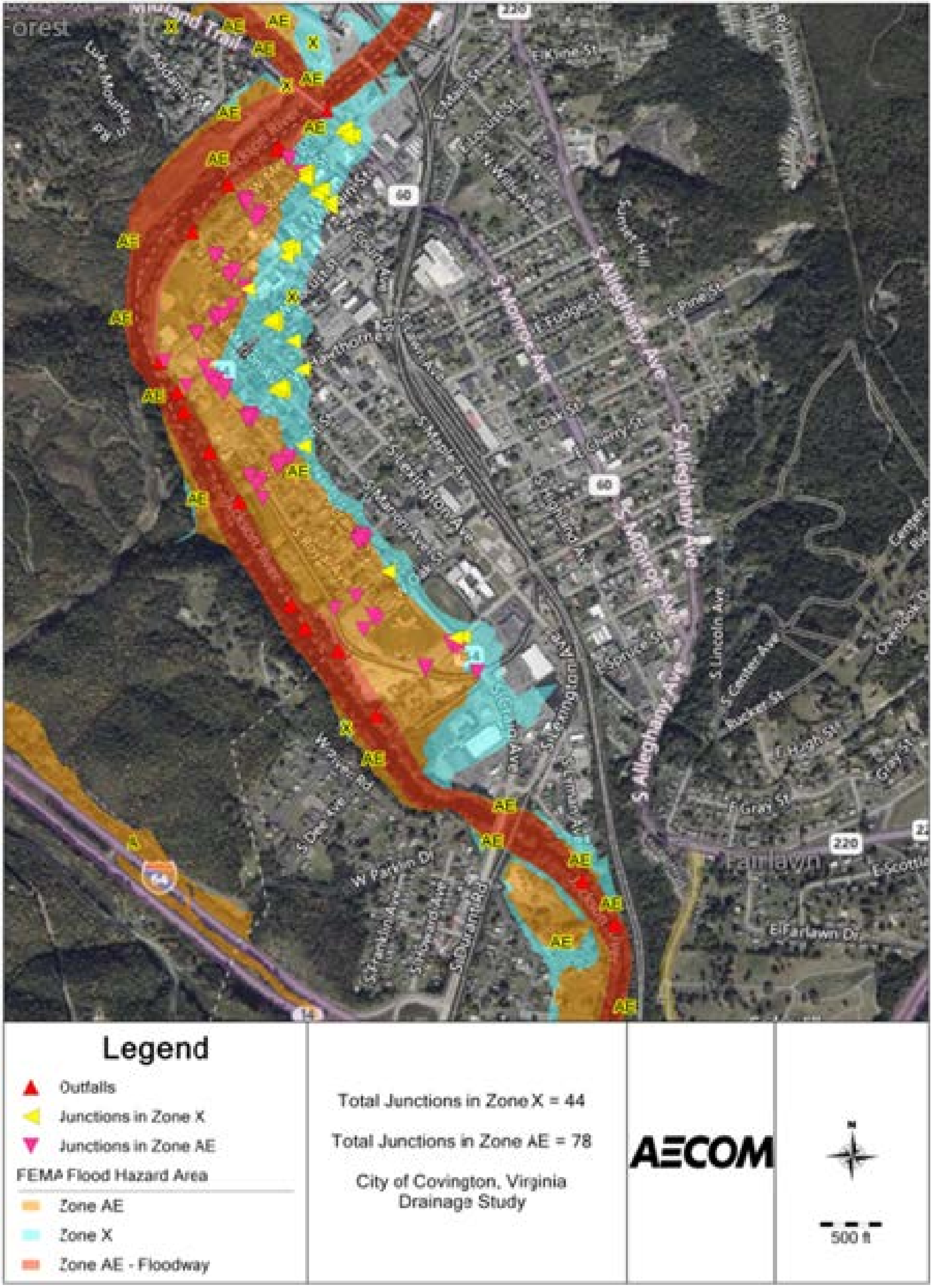

Figure 4-7: Existing Junctions Within the FEMA Floodplain Exhibit

5. Proposed Alternatives

5.1 Alternatives Narrative

The City's main objective for identifying the appropriate alternatives focuses mainly on mitigating floods from the two main underpasses that serve the public traffic to permit travel to and from the western and eastern sections of the City. The City is comprised of multiple individual stormwater network systems, so it is not feasible to provide a single improvement to remedy the entire study area. The proposed alternatives are mainly aimed at identifying individual improvements to Chestnut and Monroe stormwater network systems to allow safe vehicular passage to the local traffic, emergency responders, and escape routes. In general, improvements to the entire study area's stormwater network systems are recommended. The specific recommended improvements include several key actions to improve the stormwater network system's conveyance and capacity.

5.1 Scenario 1 Improvements Description

The first alternative is to clear 20 of 328 identified conduits, such as pipe and open channels, of debris, sediment accumulation, and overgrown vegetation to ensure unobstructed water flow. The anticipated benefit is to restore the storage capacity and flow conveyance to the existing stormwater network. While this may seem like an appropriate reactive measure to address flooding, it can sometimes result in overflow at downstream junctions due to quicker conveyance, compared to what has been experienced. Therefore, modeling this scenario is essential to understand the network's intended performance and operation. The results are discussed in **Section 6.1 Scenario 1 Model Results and Analysis**.

5.2 Scenario 2 Improvements Description

Scenario 2 builds upon Scenario 1. To address the issue of fluvial flooding, this scenario was developed to examine the performance of the stormwater networks with tide gates installed at each of the 16 outfalls. The locations of all outfalls is provided in **Figure 4-7**.Tide gates are backflow preventers installed at the end of a pipe network to block tailwater from entering the storm network. The network would retain the captured runoff within its storage capacity and release it once the tailwater conditions subside. There may still be capacity issues if the river level is too high, as this could prevent runoff from the system from being discharged. This analysis aims to determine the impact and benefits of implementing tide gates in the study area. The results are discussed in **Section 6.2 Scenario 2 Model Results and Analysis**.

5.3 Scenario 3 Improvements Description

The Scenario 3 builds upon Scenario 2, but this alternative's main objective is to mitigate the Chestnut underpass from flooding during the 100-year 24-hour storm event. The additional proposed improvements extents are illustrated in **Figure 5-2** and proposed profile in **Figure 5-1**.

These improvements are concentrated on the trunkline downstream of the underpass. The improvements include upsizing the trunkline to 72 inches, more specifically the conduits (C126, C112, C122, C61, C120, C124, C123, C128, C131), for a total length of approximately 1,963.0 feet. A total of 9 junctions will be replaced along the length of the trunkline to accommodate the upsized diameter and a new slope. Negative slopes within the trunkline will be eliminated, and a more uniform slope of 0.2% will be implemented to follow the minimum design standard allowed by the VDOT guidelines. The new slope was achieved without adjusting the underpass junction's invert elevation. Given the rim elevations of junctions 2408_X_04 and 2408_X_03, construction appears to be impacted or nearly impacted by the new trunkline. Therefore, a supplemental site survey and additional information are needed to verify constructability. As this study did not include a land survey, the conduit clearance with surface elevation should be investigated further. The utilities were not considered for the new trunkline's constructability. Additionally, a tide gate is provided at the outfall, as the outfall is located within the Action flood zone of Jackson River. The results are discussed in **Section 6.3 Scenario 3 Model Results and Analysis**.

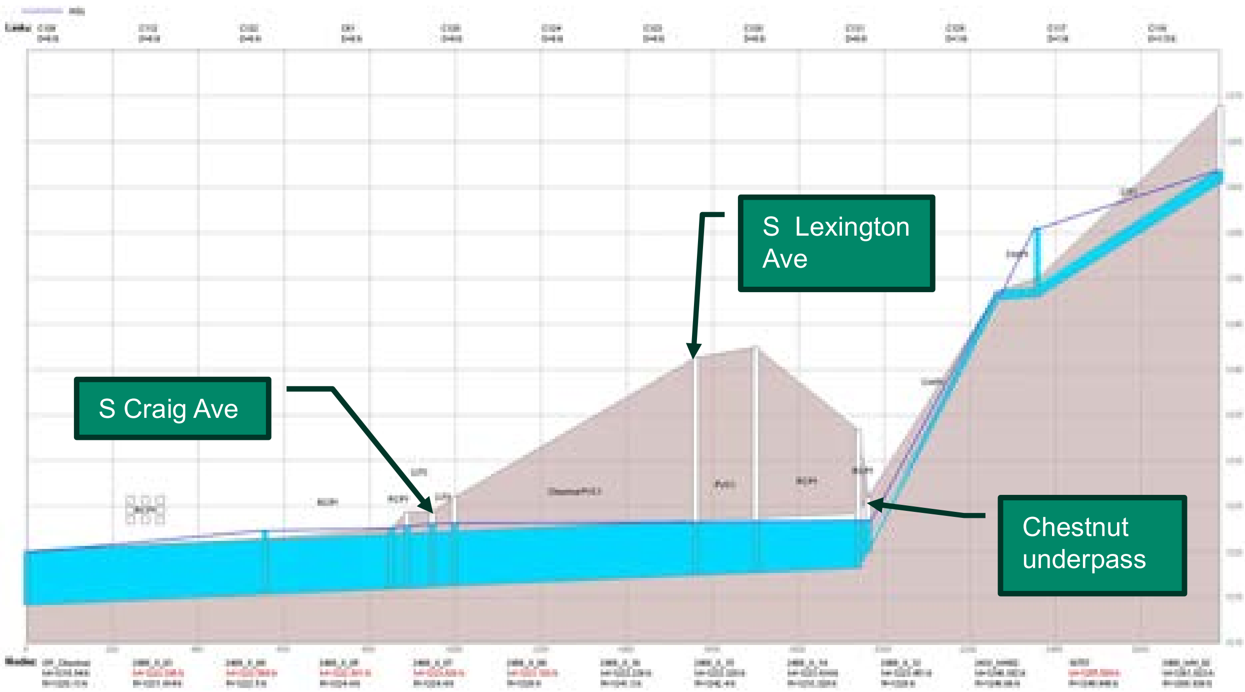

Figure 5-1: 100-Year 24-Hour Chestnut Proposed Stormwater Network Profile

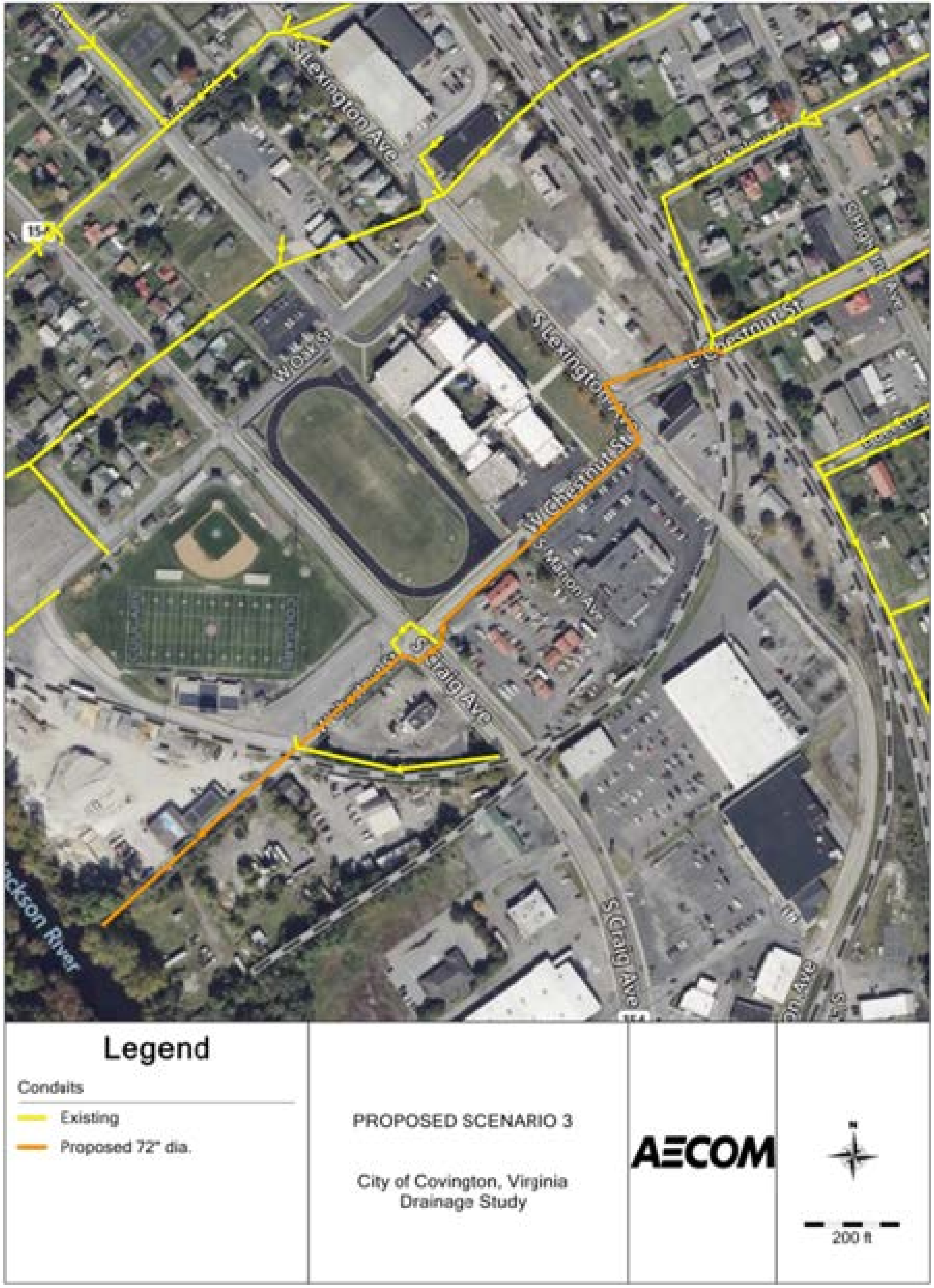

Figure 5-2: Chestnut Proposed Stormwater Network Map

5.4 Scenario 4 Improvements Description

Scenario 4 includes the improvements described in Scenario 1. The primary objective of this alternative is to mitigate flooding at the Monroe underpass during the 100-year 24-hour storm event. The additional proposed improvements extents are illustrated in **Figure 5-4** and proposed profile in **Figure 5-3**. These improvements are concentrated on the trunkline downstream of the underpass. The improvements include upsizing the Trunkline #1 to 60-inch diameter, more specifically the conduits (518D8, 518D5, 5197B, 519C0), for a total length of approximately 1,114.0 feet. A total of 4 junctions will be replaced along the length of the trunkline to accommodate the upsized diameter. To rectify the absence of a consistent slope, a uniform slope of 0.4% will be implemented. The 0.4% slope is greater than the minimum design standard allowed by VDOT. The proposed underpass junction 7D7D6 should be lowered to an invert elevation of 1,227.25 feet to accommodate the new uniform slope. This slope appears feasible in terms of clearance; however, utilities were not considered in the selection of this grade for design purposes. Although the existing outfall discharges into Jackson River above the Action flood elevation, a tide gate is included in the model as it may still help to mitigate riverine flooding. These measures are designed to improve conveyance capacity and ensure effective stormwater management.

By implementing the improvements to Trunkline #1 as described above, Trunkline #2 is anticipated to see improvements as well, thanks to decreased flows from the Monroe underpass. This will likely enhance conveyance capacity and expand available network storage, providing overall benefits to the network. The results are assessed in **Section 6.4 Scenario 4 Model Results and Analysis**.

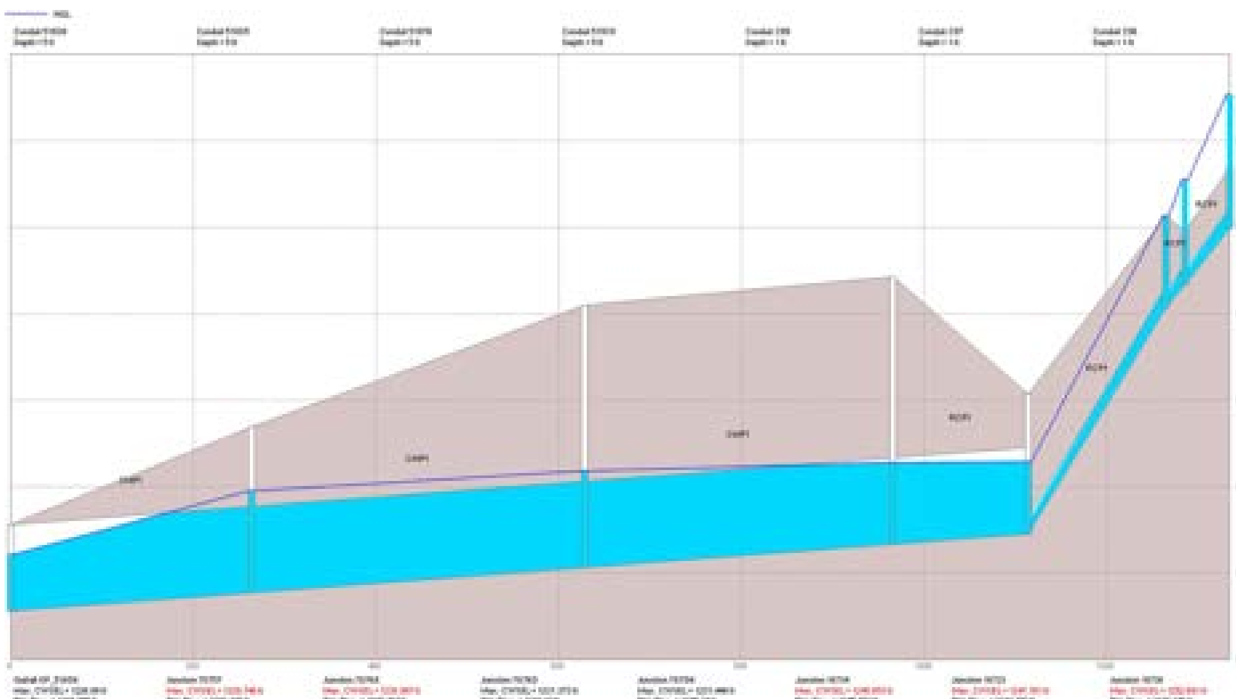

Figure 5-3: 100-Year 24-Hour Monroe Proposed Trunkline #1 Profile

Figure 5-4: Proposed Scenario 4 Map

5.5 Scenario 5A Improvements Description

Scenario 5A incorporates the improvements described in Scenario 1. The objective of Scenario 5 improvements was focused on identifying a simpler recommendation for future improvements to the City's stormwater network systems, specifically those involving storm sewer pipe replacement. By determining a recommended minimum conduit diameter, the goal is to provide the City with a simpler guideline to address and mitigate flooding. The scenario 5A proposes a minimum conduit diameter of 15 inches to be incorporated within the study area. This improvement requires 11,547.0 feet of pipe to be replaced, or 31% of total modeled circular conduits. The results are provided in **Section 6.5 Scenario 5A Model Results and Analysis**.

5.6 Scenario 5B Improvements Description

Scenario 5B includes Scenario 1. This alternative was developed to aid the City in decision-making for the minimum conduit size for future storm sewer replacement to reduce flooding within the study area, as described in Scenario 5A. This Scenario 5B recommends a minimum conduit size of 18 in throughout the study area. This improvement requires 14,484.53 feet of pipe to be replaced, or 39% of total modeled circular conduits. Results are presented in **Section 6.6 Scenario 5B Model Results and Analysis**.

5.7 Scenario 5C Improvements Description

Scenario 5C, is comparable to Scenario 5B, as this Scenario is designed to assist the City in determining the optimal minimum conduit size for future storm sewer replacements to reduce flooding within the study area. This scenario utilizes a uniform conduit size of 24 in across the entire study area and the Scenario 1 improvements. This improvement requires 27,882.40 feet of pipe to be replaced, or 74% of total modeled circular conduits. The results are provided in **Section 6.7 Scenario 5C Model Results and Analysis**.

6. Alternatives Evaluation

The alternatives evaluation is conducted by reviewing the model results with the results of the base model flooding impacts during all the selected storm events. The base model results are provided in **Table 4-1**.

6.1 Scenario 1 Model Results and Analysis

The results of Scenario 1 are presented in **Table 6-1**. The results of implementing improvements described in **Section 5.1 Scenario 1 Improvements Description** reveal that as the severity of storm events increases, the number of flooded junctions and the percentage of the total inventory flooded also rise. This alternative is not recommended on its own due to the adverse results observed during return periods exceeding the 25-year, 24-hour event. Specifically, the total junctions flooded range from 154 units for the 2-year 24-hour storm event to 225 units for the 100-year 24-hour storm event. Correspondingly, the percentage of the total inventory flooded increases from 47% to 69%. However, the percent reduction or gain from the base model shows a decreasing trend, starting at 3% for the 2-year event and dropping to -2% for the 100-year event. This indicates that the system's performance in managing more severe storm events worsens, highlighting the need for additional flooding mitigation alternatives.

Table 6-1: Proposed Scenario 1 Results

Description	Total Junctions Flooded				
	2yr-24hr	10yr-24hr	25yr-24hr	50yr-24hr	100yr-24hr
Scenario 1	154	195	210	220	225
Percent of Total Inventory Flooded	47%	60%	65%	68%	69%
Percent Reduction or Gain from the Base Model	3%	1%	0%	-3%	-2%

6.2 Scenario 2 Model Results and Analysis

The results of Scenario 2 are presented in **Table 6-2.** The results of implementing improvements described in **Section 5.2 Scenario 2 Improvements Description** show a consistent increase in the number of flooded junctions and the percentage of the total inventory flooded as the severity of storm events increases. This alternative is not advisable, as the results indicate poor performance during storm events exceeding the 25-year, 24-hour return period. The total junctions flooded range from 154 for the 2-year, 24-hour storm event to 226 for the 100-year, 24-hour storm event. The percentage of the total inventory flooded rises from 47% to 70%. The percent reduction or gain from the base model starts at 3% for the 2-year event and decreases to -2% for both the 50-year and 100-year events. This suggests that the system's hydraulic performance degrades under more intense storm events, indicating that the tide gates are likely not a great factor attributable to the study area’s flooding.

Table 6-2: Proposed Scenario 2 Results

Description	Total Junctions Flooded				
	2yr-24hr	10yr-24hr	25yr-24hr	50yr-24hr	100yr-24hr
Scenario 2	154	196	209	219	226
Percent of Total Inventory Flooded	47%	60%	64%	67%	70%

Percent Reduction or Gain from the Base Model	3%	1%	0%	-2%	-2%

6.3 Scenario 3 Model Results and Analysis

The results of Scenario 3 are presented in **Table 6-3**. The results of implementing improvements described in **Section 5.3 Scenario 3 Improvements Description** show a consistent mitigation result during each of the return periods modeled.

This alternative shows an increase in the number of flooded junctions and the percentage of the total inventory flooded as the severity of storm events increases. The total junctions flooded range from 146 for the 2-year 24-hour storm event to 213 for the 100-year 24-hour storm event. Similarly, the percentage of the total inventory flooded rises from 45% to 66%. The percent of junction flooding reduction or gain from the base model starts at 8% for the 2-year event and decreases to 4% for the 100-year event. This indicates that while the system performs relatively well under less severe storm events, its effectiveness diminishes as the severity of the storm events increases, highlighting the need for further improvements to handle large storm events effectively. While the proposed improvements in this section specifically target flooding within the Chestnut underpass, the number of flooded junctions is less critical. The primary focus is on ensuring the underpass remains functional and free from flooding, which is the main objective of these enhancements.

The proposed trunkline analysis of the 100-year 24-hour storm event shows that a 66-inch diameter RCP trunkline provides 0.77 feet of freeboard at the underpass junction 2408_X_12. To accommodate for unforeseen circumstances and general good practice a 1-foot of freeboard is recommended to avoid any future flooding due to increases to rainfall depth, such as the findings described in **Section 3.2.4 Climate Change Impact on Precipitation**. Since the invert elevation is approximated from the DEM, the freeboard is likely below the recommended 1-foot depth. Upsizing the trunkline to a 72-inch diameter increases the freeboard to about 1.52 feet, which is more favorable.

This trunkline mitigates the flooding at the underpass, however 2 junctions within the proposed section of trunkline still flood. Junction 2408_X_03 and 2408_X_04 continue to flood due to the close proximity to Jackson River; this section is not mitigated via this scenario to contain the 100-year storm. Modeling efforts revealed that increasing the conduit downstream of junction 2408_X_03 to a double barrel and adding a tide gate may mitigate junction flooding within this trunkline.

To address the flatter slope required to accommodate the elevation differences between the underpass's junction invert elevation and the outfall invert elevation, it is recommended to implement a tide gate in this network. This measure will prevent the Jackson River from backing into the stormwater system and will hold the runoff until the flood elevation decreases. Once the flood elevation has decreased, the network will begin to release the stored runoff water.

Table 6-3: Proposed Scenario 3 Results

Description	Total Junctions Flooded				
	2yr-24hr	10yr-24hr	25yr-24hr	50yr-24hr	100yr-24hr
Scenario 3	146	183	198	203	213
Percent of Total Inventory Flooded	45%	56%	61%	62%	66%
Percent Reduction or Gain from the Base Model	8%	7%	6%	5%	4%

6.4 Scenario 4 Model Results and Analysis

The results of Scenario 4 are presented in **Table 6-4**. The results of implementing the improvements of **Section 5.4 Scenario 4 Improvements Description** show positive results for the study area across all return periods, except for the 100-year, 24-hour storm event. This indicates that this alternative is generally favorable.

The percent reduction or gain from the base model starts at 6% for the 2-year event and decreases to 0% for the 100-year event. This shows that the system works well during less severe storms but loses effectiveness as storm intensity increases, indicating a need for other improvements to handle higher rainfall events. The proposed improvements are targeted to prevent flooding in the Monroe underpass, with the main goal being to improve the network section downstream of the underpass, rather than focusing on the number of flooded junctions within the study area.

The trunkline #1 was the only proposed area of improvements as described in Section **5.4 Scenario 4 Improvements Description**. The Base model indicated that a 54-inch trunkline can convey the 100-year 24-hour storm event without causing flooding at the underpass junction 7D7D6. However, this configuration does not achieve the recommended 1-foot of freeboard from the top of the HGL to the rim elevation. In light of the increased rainfall depths discussed in Section **3.2.4 Climate Change Impact on Precipitation** it is recommended to adjust the elevation accordingly. Upsizing the trunkline to 60 inches significantly improves the situation, providing 3.89 feet of freeboard. The stormwater network's system improves with the proposed 60-inch Trunkline #1, resulting in only two downstream conduits experiencing surcharging. It was observed that to further mitigate conduit surcharge, the Trunkline #1 can be upsized to 72 inches.

Table 6-4: Proposed Scenario 4 Results

Description	Total Junctions Flooded				
	2yr-24hr	10yr-24hr	25yr-24hr	50yr-24hr	100yr-24hr
Scenario 4 and Scenario 1	149	186	202	212	220
Percent of Total Inventory Flooded	46%	57%	62%	65%	68%
Percent Reduction or Gain from the Base Model	6%	6%	4%	1%	0%

6.5 Scenario 5A Model Results and Analysis

The results of Scenario 5A, as presented in **Table 6-5**, indicate that the 15-inch minimum pipe diameter yields a percent reduction or gain from the base model starting at 5% for the 2-year event and decreasing to -2% for the 100-year event.

This analysis, detailed in **Section 5.5 Scenario 5A Improvements Description**, shows that implementing a 15-inch minimum diameter throughout the network provides improvements for the 2-year, 10-year, and 25-year, 24-hour storm events. However, for larger storms, the performance deteriorates compared to the base model. This suggests that while the 15-inch diameter is effective for smaller storm events, it is not recommended for mitigating flooding during more severe storms

Table 6-5: Proposed Scenario 5A Results

Description	Total Junctions Flooded				
	2yr-24hr	10yr-24hr	25yr-24hr	50yr-24hr	100yr-24hr
Scenario 5A and Scenario 1	150	191	204	216	226

Percent of Total Inventory Flooded	46%	59%	63%	66%	70%
Percent Reduction or Gain from the Base Model	5%	3%	3%	-1%	-2%

6.6 Scenario 5B Model Results and Analysis

The results of Scenario 5B, as presented in **Table 6-6**, indicate that the 18-inch diameter upsizing alternative shows a reduction from the base model. This reduction starts at 6% for the 2-year event and decreases to 1% for the 100-year event, as analyzed in **Section 5.6 Scenario 5B Improvements Description**.

Scenario 5B also illustrates that no reduction or gain occurs at the 50-year 24-hour event, which suggests additional diameter upsize is required to achieve a more uniform reduction across the study area. This indicates that this alternative is favorable for implementation within the study area as a guideline for future improvements.

Table 6-6: Proposed Scenario 5B Results

Description	Total Junctions Flooded				
	2yr-24hr	10yr-24hr	25yr-24hr	50yr-24hr	100yr-24hr
Scenario 5B and Scenario 1	149	183	201	215	219
Percent of Total Inventory Flooded	46%	56%	62%	66%	67%
Percent Reduction or Gain from the Base Model	6%	7%	4%	0%	1%

6.7 Scenario 5C Model Results and Analysis

The results of Scenario 5C are presented in **Table 6-7**. The analysis of the **Section 5.7 Scenario 5C Improvements Description**, which examines the improvements using a 24-inch minimum diameter alternative, shows a percent reduction from the base model starting at 31% for the 2-year event and decreasing to 1% for the 100-year event. These results indicate a uniform improvement across all storm events modeled.

These results indicate the most consistent improvements to the City's stormwater networks. Consequently, it is recommended that the City implement a guideline mandating a minimum conduit diameter of 24 inches for storm sewer replacement projects.

Table 6-7: Proposed Scenario 5C Results

Description	Total Junctions Flooded				
	2yr-24hr	10yr-24hr	25yr-24hr	50yr-24hr	100yr-24hr
Scenario 5C and Scenario 1	109	169	191	204	218
Percent of Total Inventory Flooded	34%	52%	59%	63%	67%

Percent Reduction or Gain from the Base Model	31%	14%	9%	5%	1%

7. Conclusion and Recommendations

The City of Covington faces significant challenges due to recurrent flooding, which endangers the health and safety of residents and roadways, particularly at the two underpasses located at North Monroe Avenue and East Chestnut Street intersections with the CSX railroad. The frequent flooding of these underpasses obstructs safe vehicle passage, exacerbating risks during emergency situations. Additionally, riverine flooding of the Jackson River contributes to hazardous conditions throughout the year. The existing conditions PCSWMM model revealed that a significant portion of the stormwater infrastructure in the City of Covington is prone to flooding. Specifically, 49% of the infrastructure nodes flood during the 2-year 24-hour storm event, and this increases to 68% during the 100-year 24-hour storm event. This highlights the urgent need for improvements to the stormwater management system to mitigate flooding risks and enhance the resilience of the City's infrastructure.

To address these challenges, several recommendations are proposed. Creating a detailed GIS map of the City's stormwater network will help better understand its condition and identify areas requiring maintenance or upgrades. Establishing a proactive maintenance schedule to clear debris and sediment from conduits will ensure unobstructed water flow and reduce the likelihood of blockages. Although, the observation of only clearing out conduits is overall negative as certain junctions receive the runoff quicker, leading to adverse effects.

Tide gates at the outfalls generally have a positive effect on lower intensity storms, such as those with return periods less than the 25-year, 24-hour event. During these lower intensity storms, a reduction in total flooded junctions is observed. However, for greater storms, the number of flooded junctions increases compared to existing conditions, indicating that tide gates may not be as effective in mitigating flooding during more severe storm events. Tide gates are recommended for both of the Monroe and Chestnut networks.

The critical upgrades that are recommended to improve the underpass flooding of Chestnut and Monroe intersections with the CSX railroad are increasing the diameter of the Chestnut trunkline to 72-inch and the Monroe trunkline to 60 inches, while adjusting the trunkline slopes. This will improve the stormwater network’s performance and reduce flooding at both underpasses. The Chestnut trunkline should also include a tide gate to discourage fluvial flooding into the proposed trunkline.

Furthermore, implementing minimum recommended conduit diameter guidelines, such as applying a minimum diameter of 24 inches across the stormwater network, will reduce the number of flooded junctions and enhance overall system capacity. The observed results for smaller diameters such as 15 inches or 18 inches are generally favorable to the study area’s flooding mitigation, however the most consistent flooding mitigation was observed within the 24-inch diameter alternative.

Implementing these recommendations will enable the City of Covington to significantly enhance its stormwater management capabilities, bolster infrastructure resilience, and safeguard the health and safety of its residents from the adverse effects of flooding. To proactively address future flooding risks, **Appendix E** provides a depiction of the approximate future impacts of climate change on rainfall depth increases. Complementing future data analysis with field verifications will validate findings and address discrepancies between datasets and real-world conditions. Finally, consulting with experts in hydrology, geology, and environmental science will ensure accurate data interpretation and meaningful insights for better decision-making.

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Appendix A Maintenance Items Along the CSX Railroad

0 15 30 60 90 120 US Feet

Storm Water System Mapping Problem Areas

Covington, Virginia

September 03, 2024

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0 15 30 60 90 120 US Feet

Storm Water System Mapping
Problem Areas

Covington, Virginia
September 03, 2024

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Storm Water System Mapping
Problem Areas

Covington, Virginia
September 03, 2024

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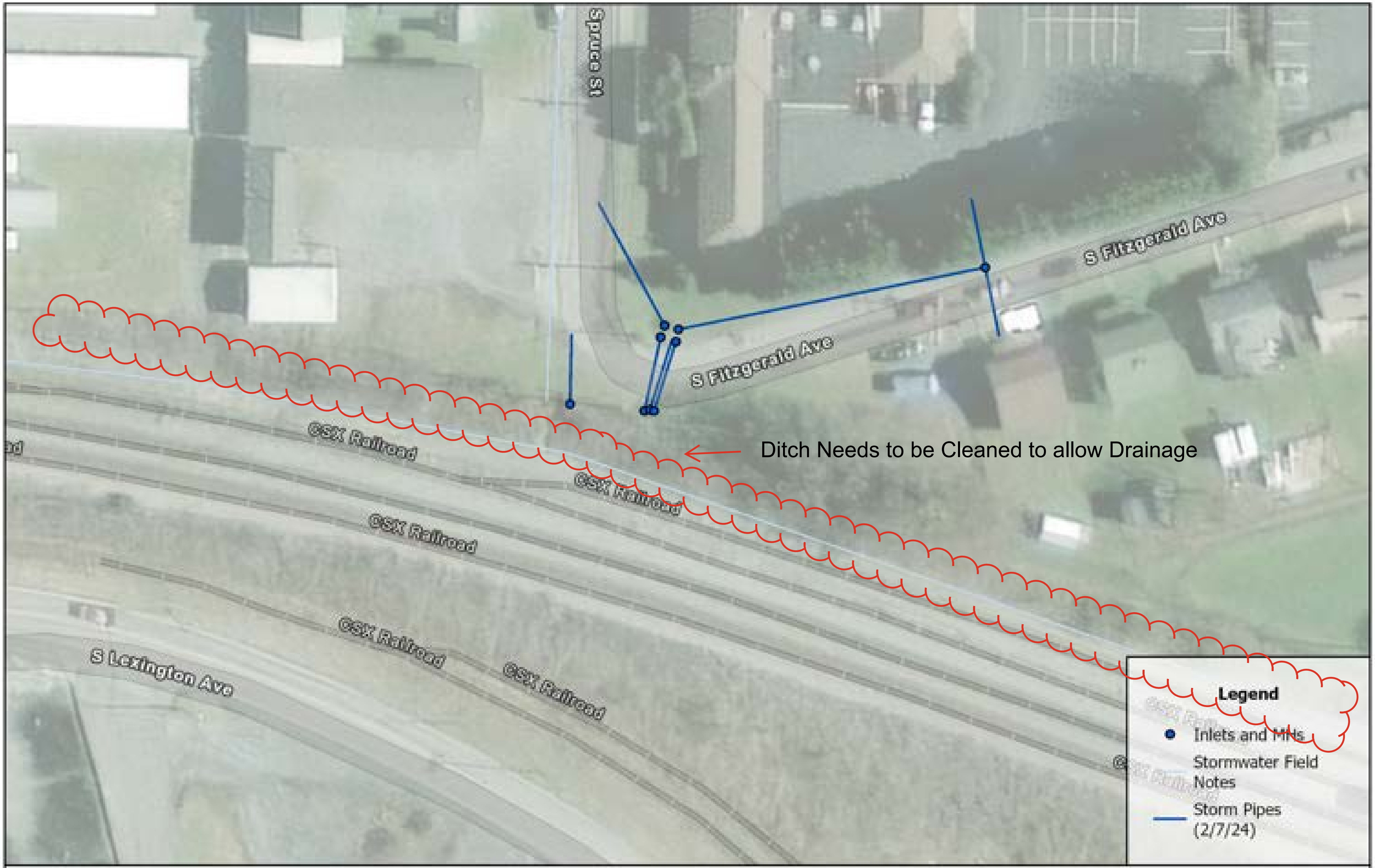

Storm Water System Mapping
Problem Areas

Covington, Virginia
September 03, 2024

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US Feet

Storm Water System Mapping
Problem Areas

Covington, Virginia
September 03, 2024

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US Feet

Storm Water System Mapping
Problem Areas

Covington, Virginia
September 03, 2024

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Appendix B Regional Rainfall Study

Regional Rainfall Study

prepared for Covington, VA

March 18, 2024

Delivering a better world

Revision History

Revision	Revision date	Revision Notes
1	March 18, 2024	Original Version

Prepared for:

City of Covington, VA
As proposed in Community Flood Preparedness Fund Round 3 Grant Application

Prepared by:

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Virginia Beach, VA 23452
aecom.com

Table of Contents

Figures

Tables

1. Introduction

1.1 Background

The City of Covington, Virginia (City), is in the southwestern part of the Commonwealth, situated in Alleghany County. It lies in the Allegheny Highlands region along the Jackson River, which is a tributary of the James River. The City is subject to riverine flooding from the Jackson River as well as pluvial flooding due to inadequate and antiquated storm sewer systems. The recurrence of higher intensity storms in recent years has stressed the stormwater conveyance system throughout the City causing local urbanized flooding that is detrimental to the residents and critical infrastructure.

1.2 Purpose

In order to quantify recent trends of rainfall intensity and estimate the future increase in precipitation, an analysis of available rainfall data in the vicinity of Covington, VA was performed. Climate change and its effects on rainfall frequency and intensity have been widely acknowledged by localities and agencies across the country as a complicating factor in designing new stormwater systems and drainage projects. Similar analyses have been conducted, even within Virginia[1], showing significant recent increases in storm intensities and rainfall depths. However, there is a lack of such analyses for the mountainous region of western Virginia, in which Covington, VA is situated. Therefore, the following analysis focused on the local and regional rainfall trends around Covington to determine whether there is evidence to support increasing the design storm rainfall depths that should be used when designing new or upgraded stormwater drainage systems.

2. Study Method

1.3 Data Sources

The primary data source for this analysis was the Climate Data Online (CDO) database provided by NOAA and the National Centers for Environmental Information (NCEI). Daily summaries for local and regional rain gages were downloaded between October 25 and November 2, 2023. For each, the longest available time frames were obtained; the data was later filtered to include only the more relevant and reliable records.

1.4 Stations Analyzed

The first portion of the analysis included only local stations, defined as those within 5 miles of Covington, VA. This resulted in three available stations: Covington (USC00442041), Covington Filter Plant (USC00442044), and Covington 4.1 E

[1] "Analysis of Historical and Future Heavy Precipitation" City of Virginia Beach (2018)

(US1VAAL0003). These stations included daily summaries for 1937-1999, 1960-2023, and 2007-2023 respectively. To ensure the patterns found in the local analysis were reflected in the Southwest VA/Southeast WV area, a regional analysis was performed with stations within a 30-mile radius of Covington, VA. As expected, this included significantly more rain gages. To narrow the analysis, I only selected gages which had data for the day of initial download, October 30, 2023. Those additional gages are shown in Figure 1 and described in Table 1.

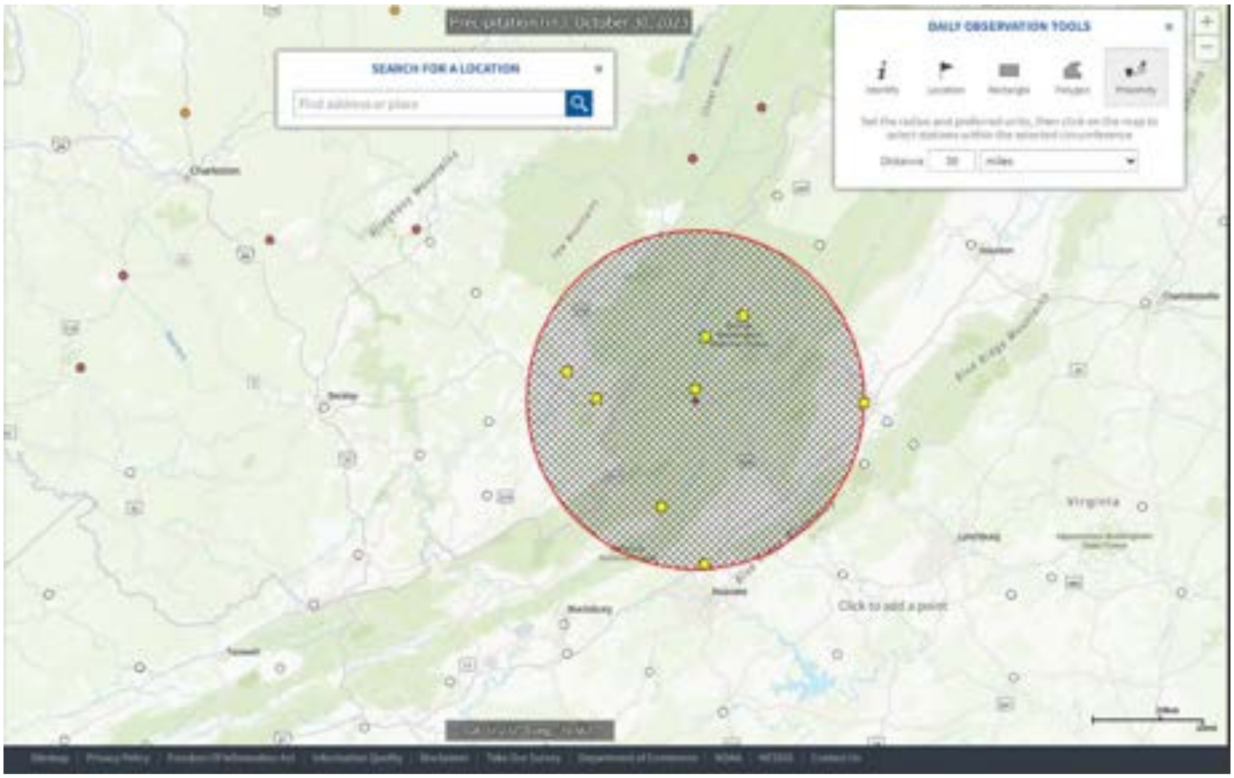


Figure 1 - Range of Regional Analysis

Table 1. Gages used in regional analysis

Name	Gage ID	Start Year[1]
Gathright Dam, VA	USC00443310	1980
Hot Springs, VA	USC00444128	1893
Lewisburg 3 N, WV	USC00465224	1851
Lexington, VA	USC00444876	1889
New Castle, VA	USC00446012	1907
Roanoke 8 N, VA	USC00447278	1998
White Sulfur Springs, WV	USC00469522	1888

[1] *A start year of 1900 was applied to all data sets with older starts due to excessive download sizes when attempting to capture the entire available ranges.*

1.5 Local Analysis

All three of the local gages were blended into one data set, using the average of all available rainfall data for each day. This was implemented to reduce the effects of outlier data. Furthermore, the data was cut off at 1/1/1947 because, from that point to present year, the coverage was above 99% (See Figure 2). Data after 12/31/2022 since we do not yet have a full year of data for 2023. Coverage, here, is defined as the percentage of days after the cutoff date for which at least one of the sources is not blank.

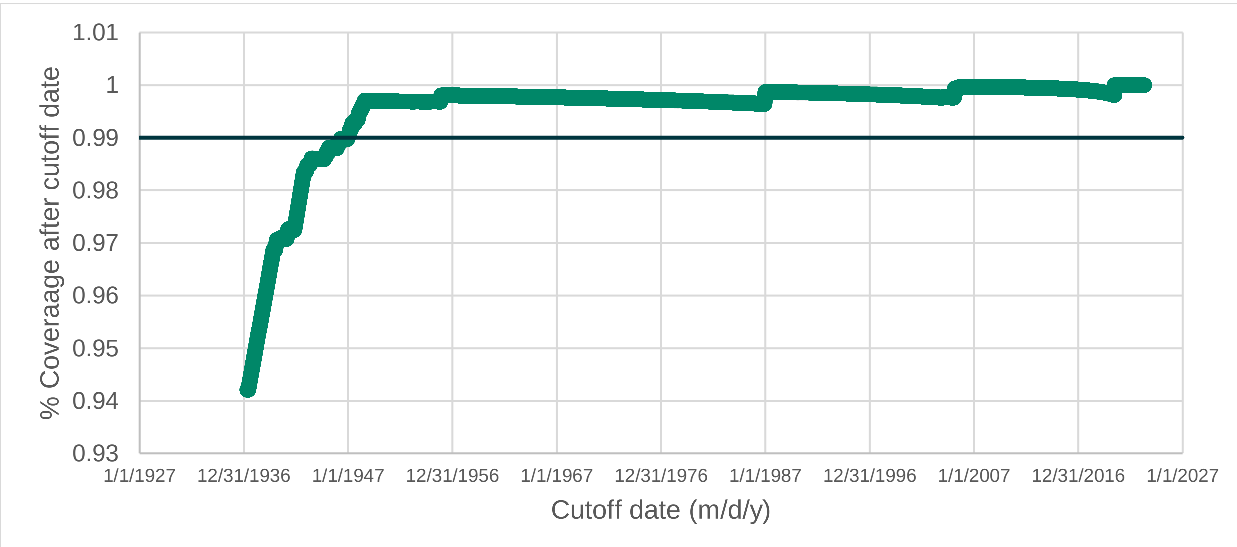


Figure 2 – Percent Coverage After Cutoff Date

Using the daily average values, annual maximum series (AMS) and peaks over threshold (POT) analyses were carried out. As shown in Figure 3, the AMS results show an increase of 0.15 in/century. This is a relatively minor but still positive trend in the local rain gages (for comparison, a Dewberry analysis of Norfolk precipitation trends found an increase of 1.98 in/century over a similar time period[2]). With a mean value of 2.3 in, that represents a growth of 0.65% every decade. The small increase is likely due to the significant outliers seen in the 1950s and 1980s. The variance in the data makes it difficult to draw a direct conclusion from the AMS alone.

[2] "Analysis of Historical and Future Heavy Precipitation" City of Virginia Beach (2018), pg. 8

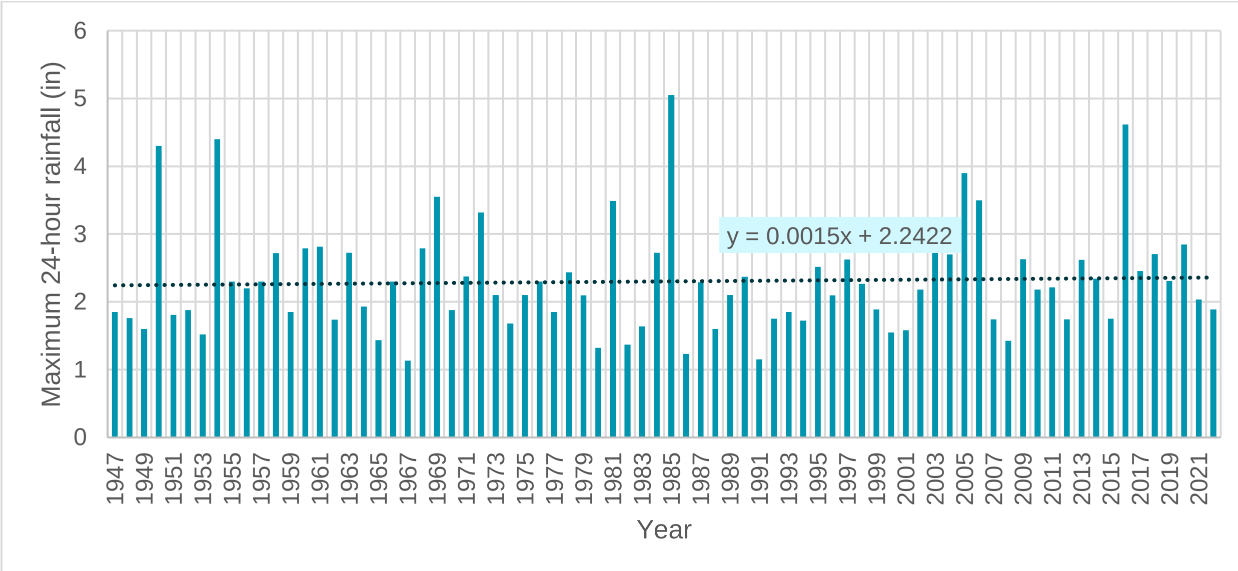


Figure 3 – Local AMS Analysis

A much more significant and obvious upward trend can be seen in the POT analysis (Figure 4), which shows the number of 24-hour storm events that surpassed 1.25 in. The trendline of this graph shows an average increase of 5.2 days/century. With a mean value of 4.0 days/year in the existing data, that represents a 13% increase every decade. Together, this appears to show the number of significant storm events per year is increasing more substantially than the magnitude of the largest annual storm events. However, the precipitation frequency graph (Figure 5) was determined using the AMS due to ease of calculation.

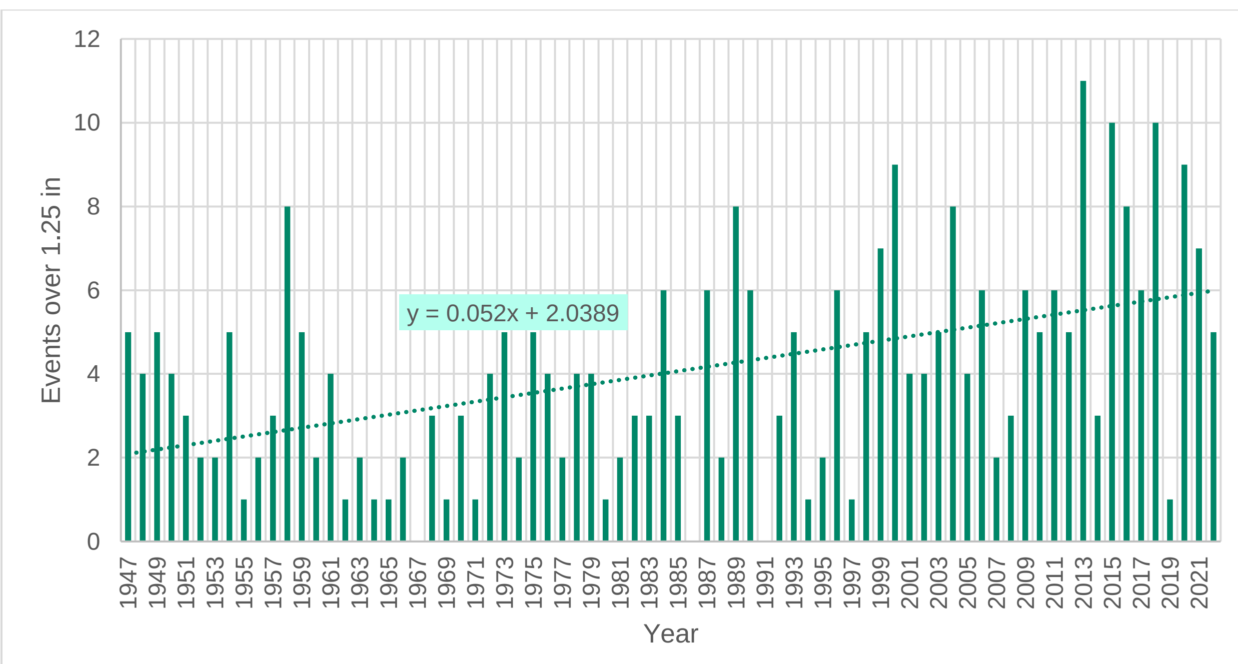


Figure 4 – Local POT Analysis

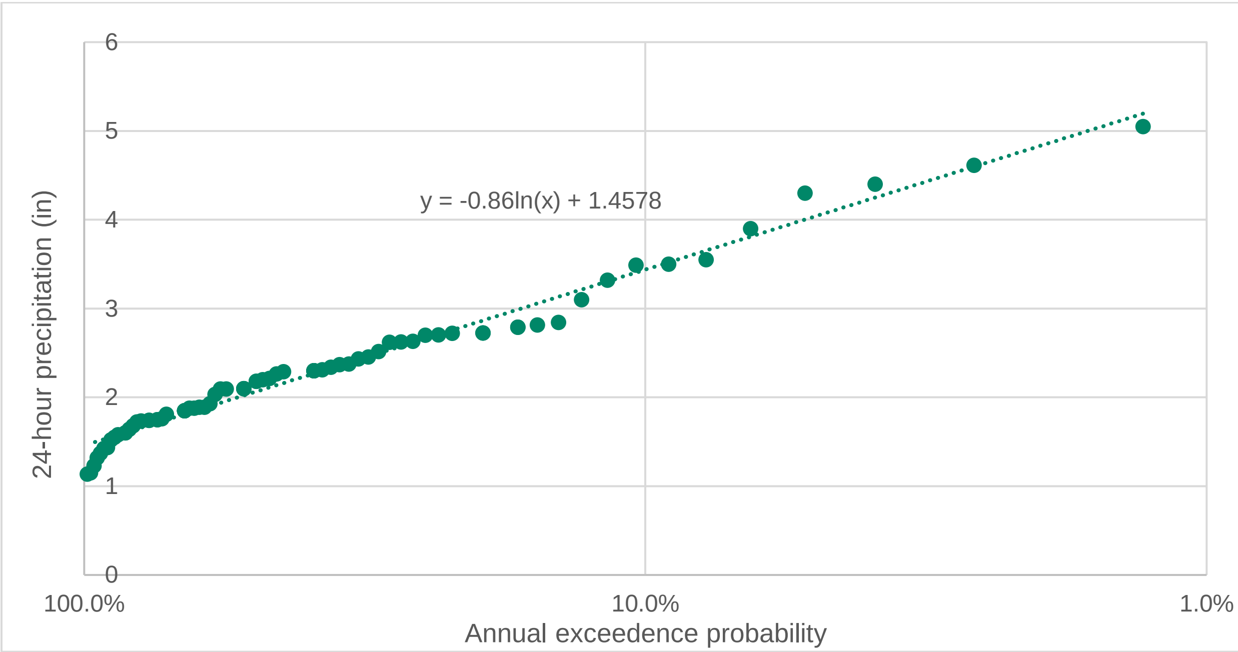


Figure 5 – Local Precipitation Frequency Analysis (from AMS)

The precipitation frequency data yielded a logarithmic trendline with an R^2 of 0.98. The formula of that trendline, shown in Figure 5, was used to find the 24-hour storm intensities of certain return periods, which are shown in table 2. Table 2 also shows the results from splitting the data at 1984, such that both before and after (1947-1984 and 1985-2022) represented 38 years of data. As seen in Figure 6, there is a significant increase between the older and newer sets, about 8.7% for each storm event.

Table 2. 24-Hour Precipitation Depths for Various Return Periods

Return Period (years)	Probability	24-hr Precipitation			% Increase
		1947-2022	1947-1984	1985-2022	
2	0.5	2.05	2.03	2.20	8.6%
5	0.2	2.84	2.82	3.07	8.6%
10	0.1	3.44	3.43	3.72	8.6%
25	0.04	4.23	4.22	4.59	8.7%
50	0.02	4.82	4.82	5.24	8.7%
100	0.01	5.42	5.42	5.90	8.7%

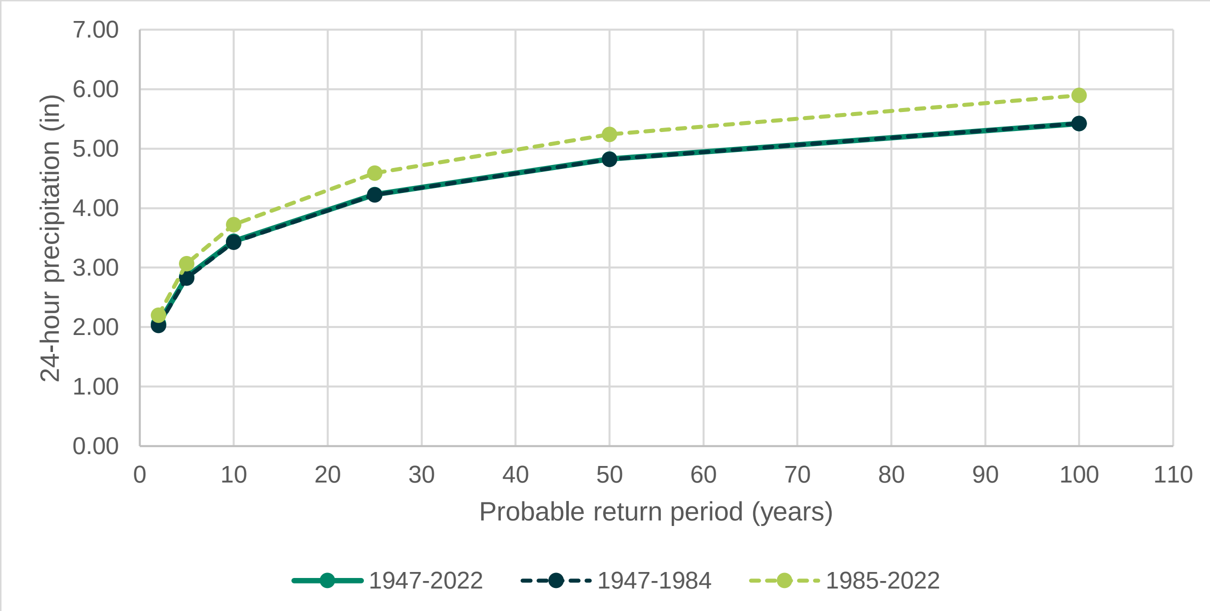


Figure 6 – 24-Hour Precipitation Depths for Various Return Periods

Unexpectedly, the precipitation levels for the various return periods found using the method outlined above are slightly lower than the NOAA Atlas 14 (volume 2 version 3) point precipitation frequency estimates (PPFE). When taking the average of the Covington and Covington Filter Plant PPFE's, the 24-hour precipitation levels are roughly 7% higher than the values derived from AMS. This may be due to differences in the methodology (ex: the inclusion of snowfall in the precipitation data; when that data was incorporated into the AMS analysis, it drastically overestimated the NOAA data and so was not included). However, since most of the storm intensities fell within the 90% confidence intervals for their respective recurrence intervals, it can still be considered reliable for the purposes of this analysis.

1.6 Regional Analysis

Regional analysis was carried out in a similar manner to the local analysis with only a few methodological changes. Since there were more rain gages to pull from and wider timeframes for many of them, it was less computationally effective to average the daily precipitation values before determining the AMS. Instead, each rain gage was individually assessed first, and the AMS results were averaged out afterward. Rain gages which did not include at least half of a year's worth of data for each calendar year were discounted from that year's AMS. As mentioned earlier, a cutoff of 1900 was imposed due to limits of data size, and between then and 2022, there were at least 3 rain gages which were included in each year's analysis, as shown in Figure 7. However, there appeared to be a high number of unusually large storm events between 1900 and 1915, when coverage was the most sparse, so the data was further limited to 1916-2022.

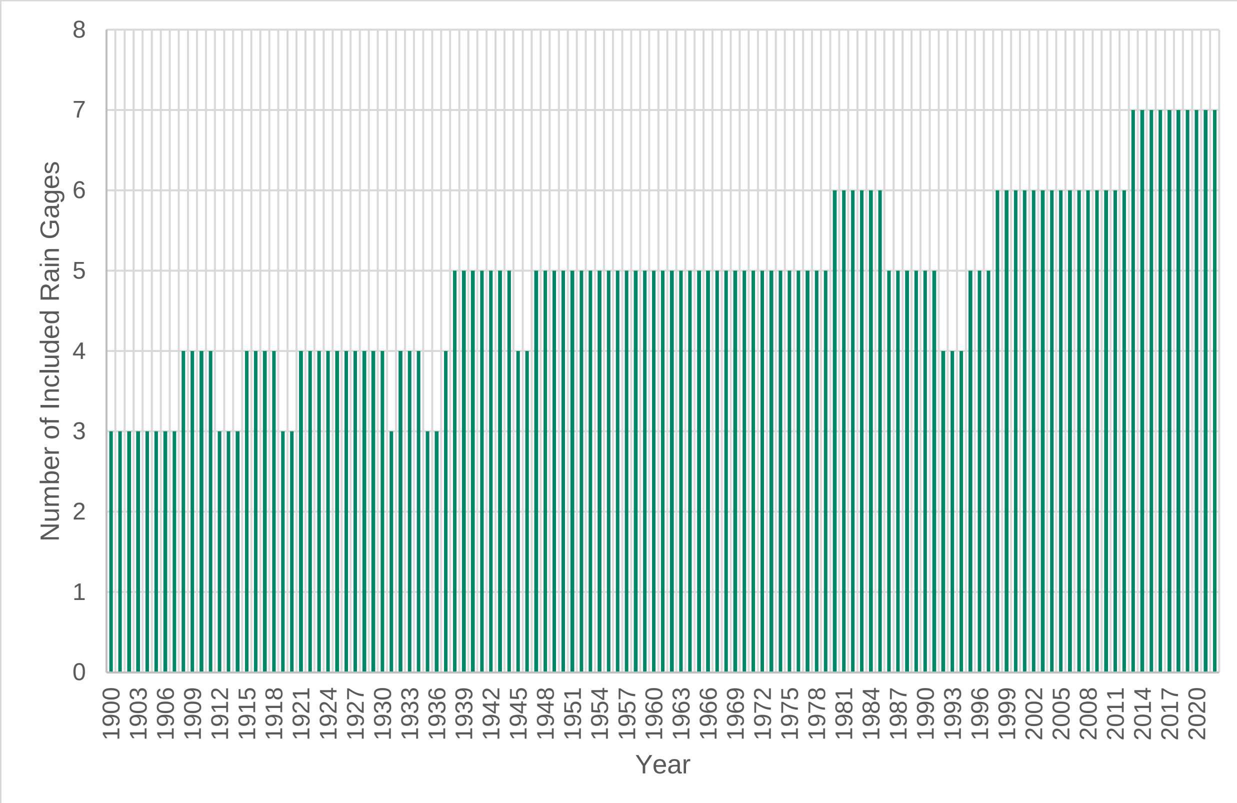


Figure 7 – Number of Rain Gages Included in AMS Analysis Per Year

The AMS of the regional rain gages shows a slight positive increase over time (Figure 8), similar to the results of local analysis. According to the trendline, there has been an average increase of 0.3 in/century, a mere 1.26% increase per decade using a mean value of 2.37 in. Like in the local analysis, the outlier events in 1954 and 1985 here are likely skewing the data.

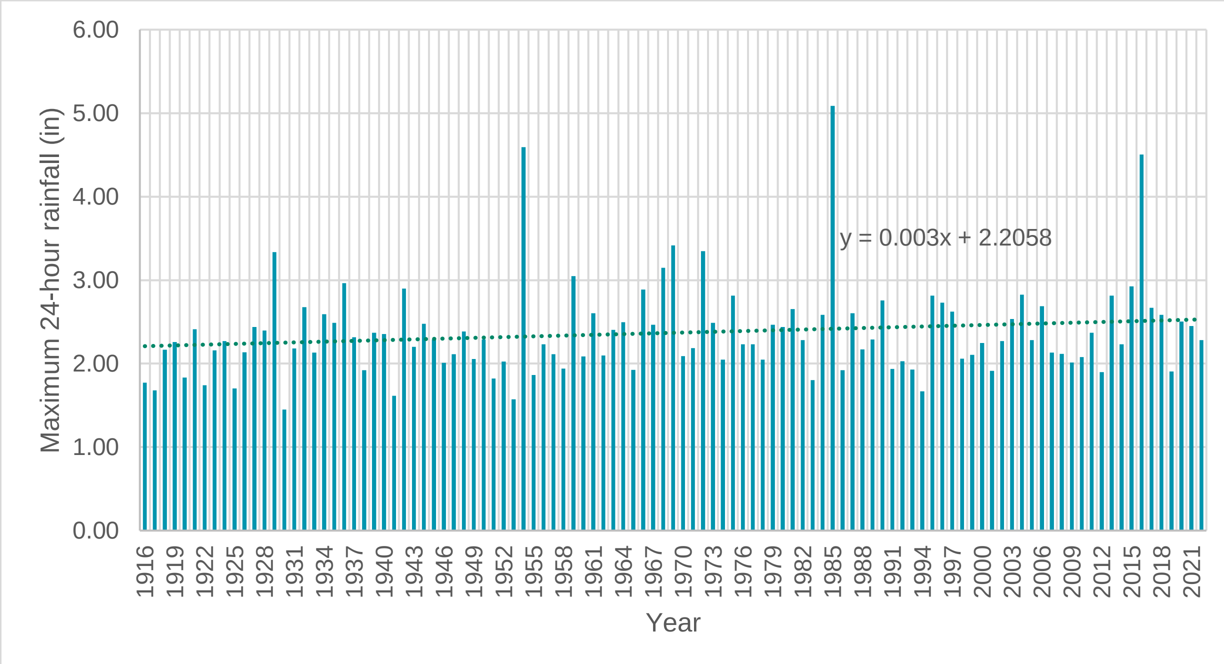


Figure 8 – Regional AMS Analysis

There was again a significantly more noticeable trend in the POT analysis, in this case an average of 2.13 days/century. That represents a 4.63% increase per decade over the existing mean value of 4.6 days per year. While this is less of an effect than seen in the local analysis, the upwards trend is still visible.

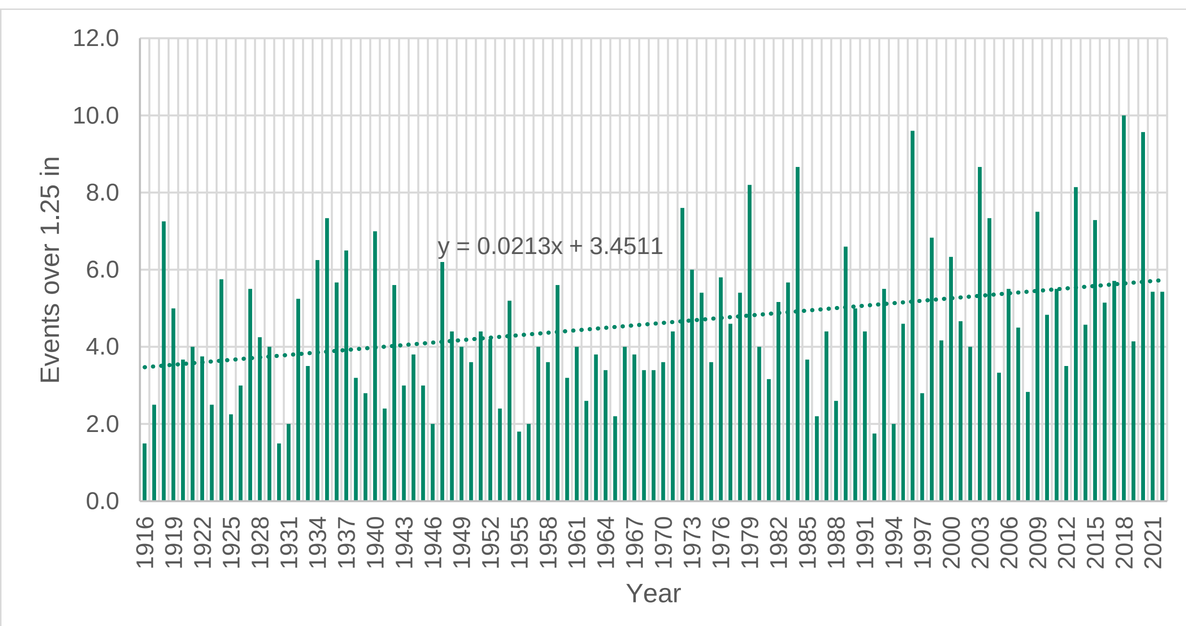


Figure 9 – Regional POT Analysis

3. Results of Local and Regional Analyses

Due to the limitations of the available data and the requirements of AMS and POT analysis, which become less reliable in shorter timescales, the best use of the findings above is to note the broad trends present. Both POT and annual exceedance analysis show significant increasing trends (12.9% per decade and 8.7% between analysis windows, respectively). Thus, it seems reasonable to expect similar or more accelerated trends as climate change continues.

Those trends were mostly reflected in the regional data. While none of the trends were as extreme as those found in other regions (such as the earlier mentioned Dewberry report of Norfolk, VA), they are nonetheless significant. A more in-depth analysis would be needed to determine the efficacy of changes to design hyetographs and specific stormwater design standards, but the findings of this report are sufficient to warrant a forecast of at least a 10% increase in expected rainfall over the next decade.

aecom.com

Appendix C Existing Condition Map Exhibits

Legend

Junctions

- Flooded Junctions
- Junctions

10-YEAR 24-HOUR STORM EVENT
BASE MODEL

TOTAL FLOODED
JUNCTIONS = 197

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft

Legend

Junctions

- Flooded Junctions
- Junctions

2-YEAR 24-HOUR STORM EVENT
BASE MODEL

TOTAL FLOODED
JUNCTIONS = 158

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft

Legend

Junctions

- Flooded Junctions
- Junctions

25-YEAR 24-HOUR STORM EVENT
BASE MODEL

TOTAL FLOODED
JUNCTIONS = 210

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft

Legend

Junctions

- Flooded Junctions
- Junctions

50-YEAR 24-HOUR STORM EVENT
BASE MODEL

TOTAL FLOODED
JUNCTIONS = 214

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft

Legend

Junctions

- Flooded Junctions
- Junctions

100-YEAR 24-HOUR STORM EVENT
BASE MODEL

TOTAL FLOODED
JUNCTIONS = 221

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft

Appendix D Proposed Alternatives Map Exhibits

Legend

Junctions

- Junctions
- Flooded Junctions

2-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 1

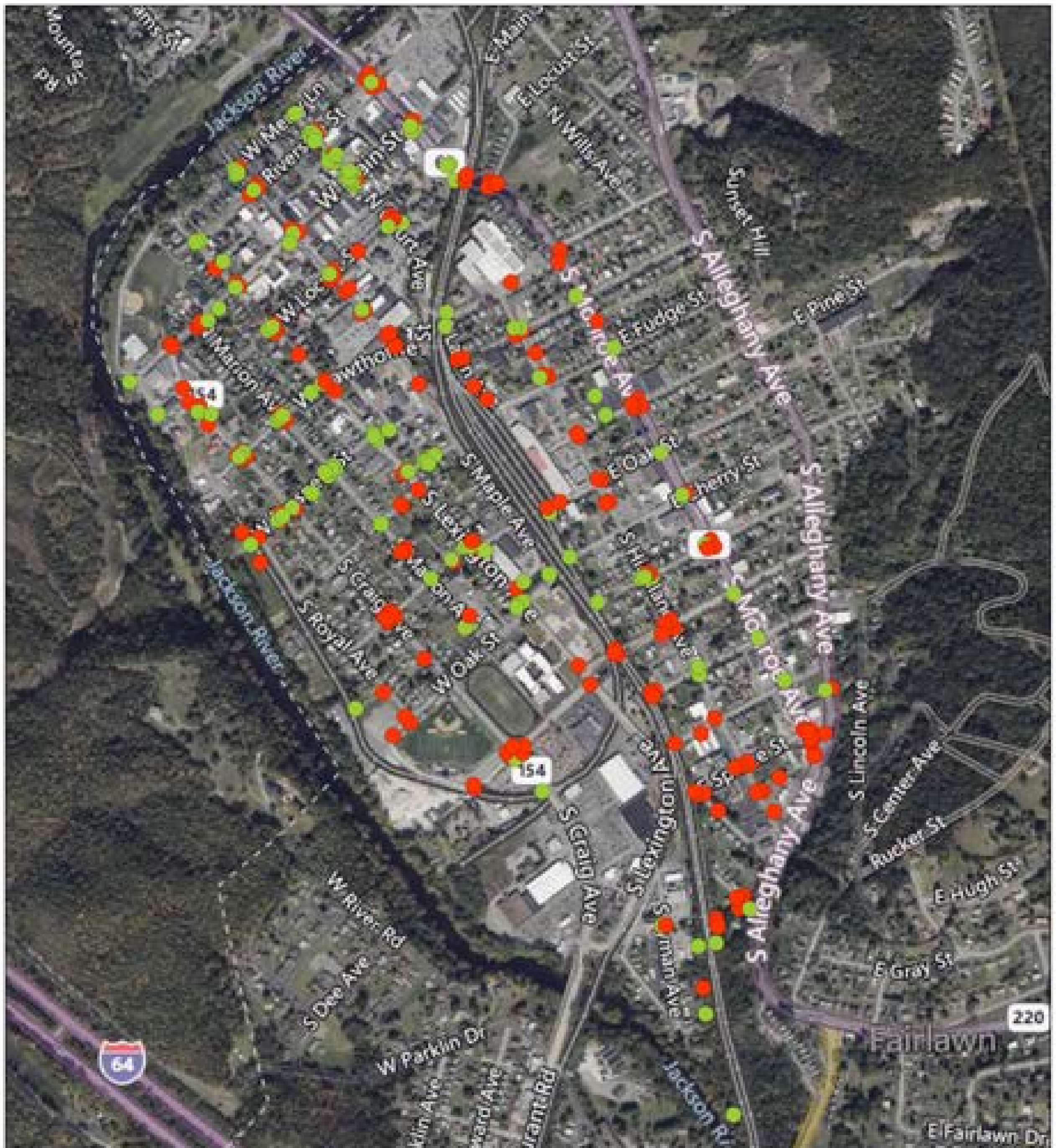
TOTAL FLOODED
JUNCTIONS = 154

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

10-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 1

TOTAL FLOODED
JUNCTIONS = 195

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft

Legend

Junctions

- Junctions
- Flooded Junctions

25-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 1

TOTAL FLOODED
JUNCTIONS = 210

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft

Legend

Junctions

- Junctions
- Flooded Junctions

50-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 1

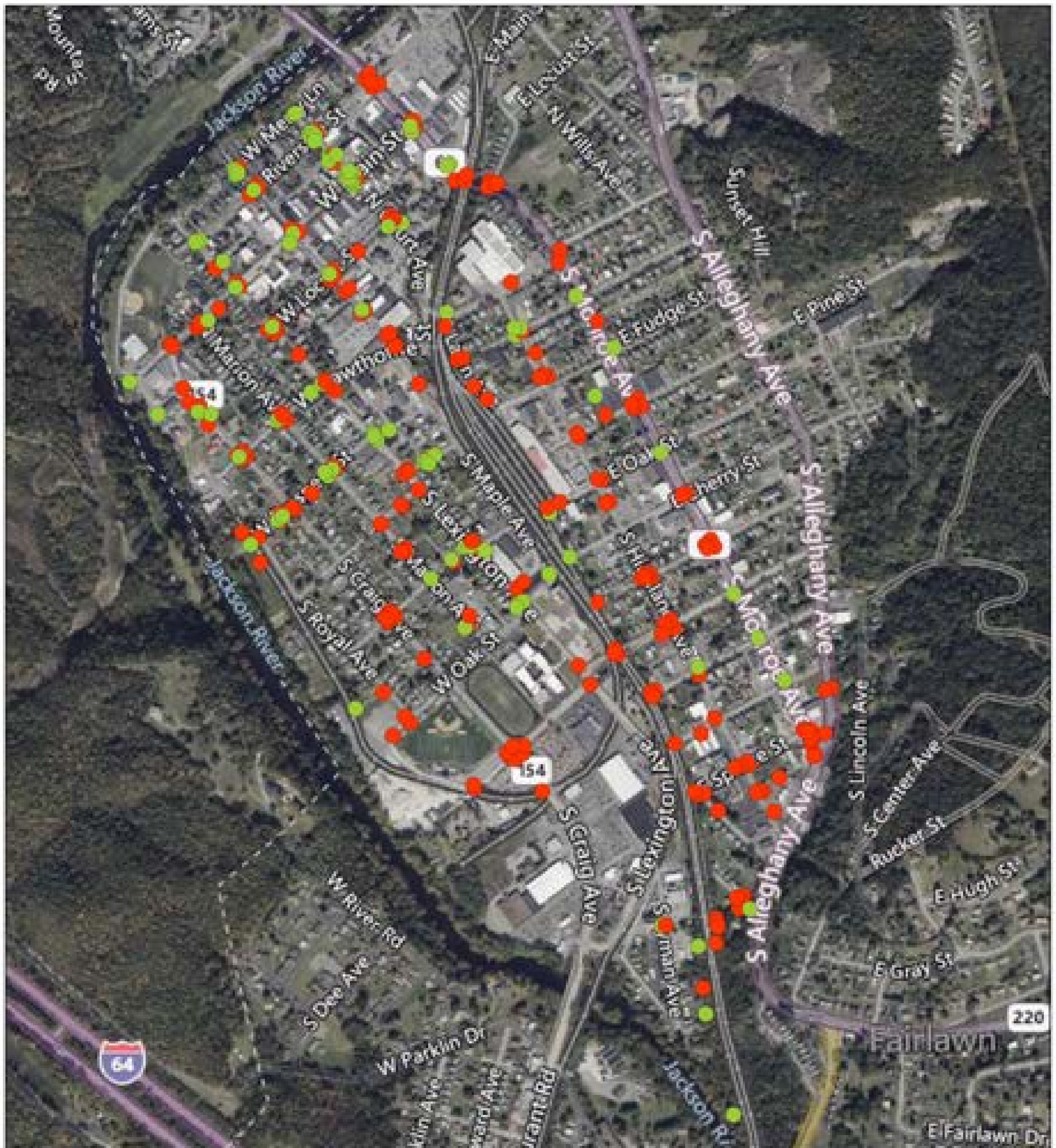
TOTAL FLOODED
JUNCTIONS = 220

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

100-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 1

TOTAL FLOODED
JUNCTIONS = 225

City of Covington, Virginia
Drainage Study

N

500 ft

Legend

Junctions

- Junctions
- Flooded Junctions

2-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 2

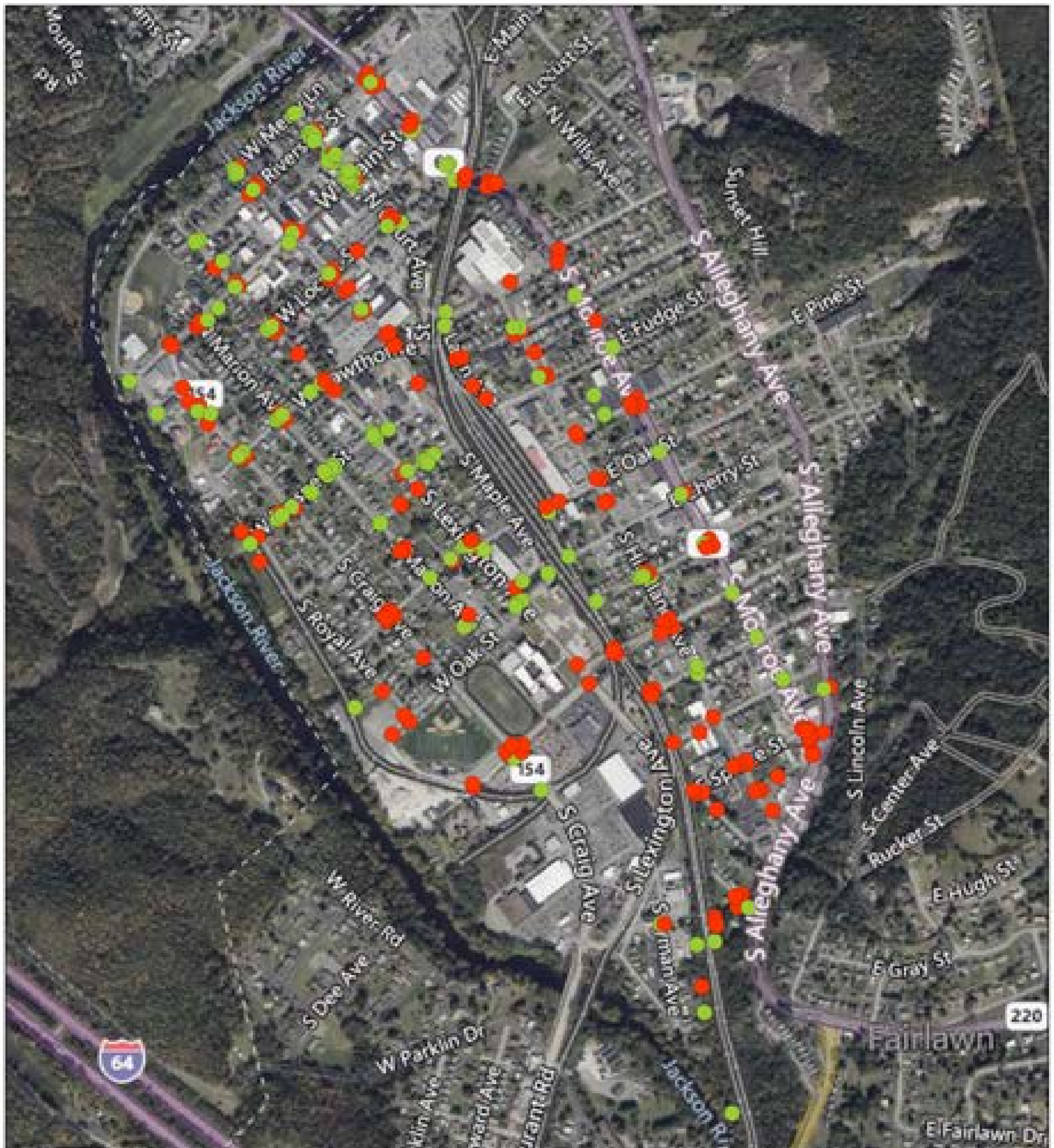
TOTAL FLOODED
JUNCTIONS = 154

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

10-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 2

TOTAL FLOODED
JUNCTIONS = 196

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft

Legend

Junctions

- Junctions
- Flooded Junctions

25-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 2

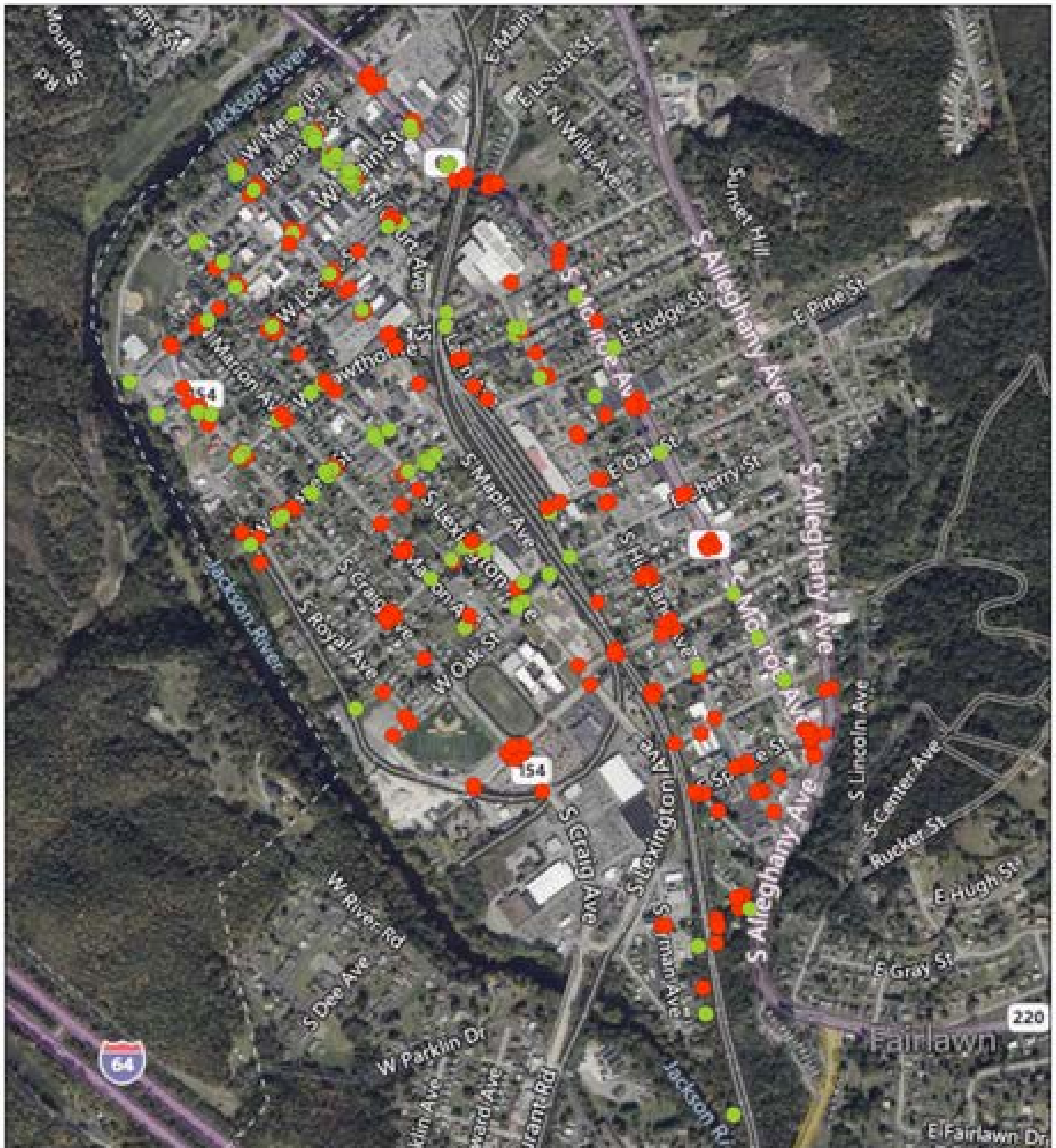
TOTAL FLOODED
JUNCTIONS = 209

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

50-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 2

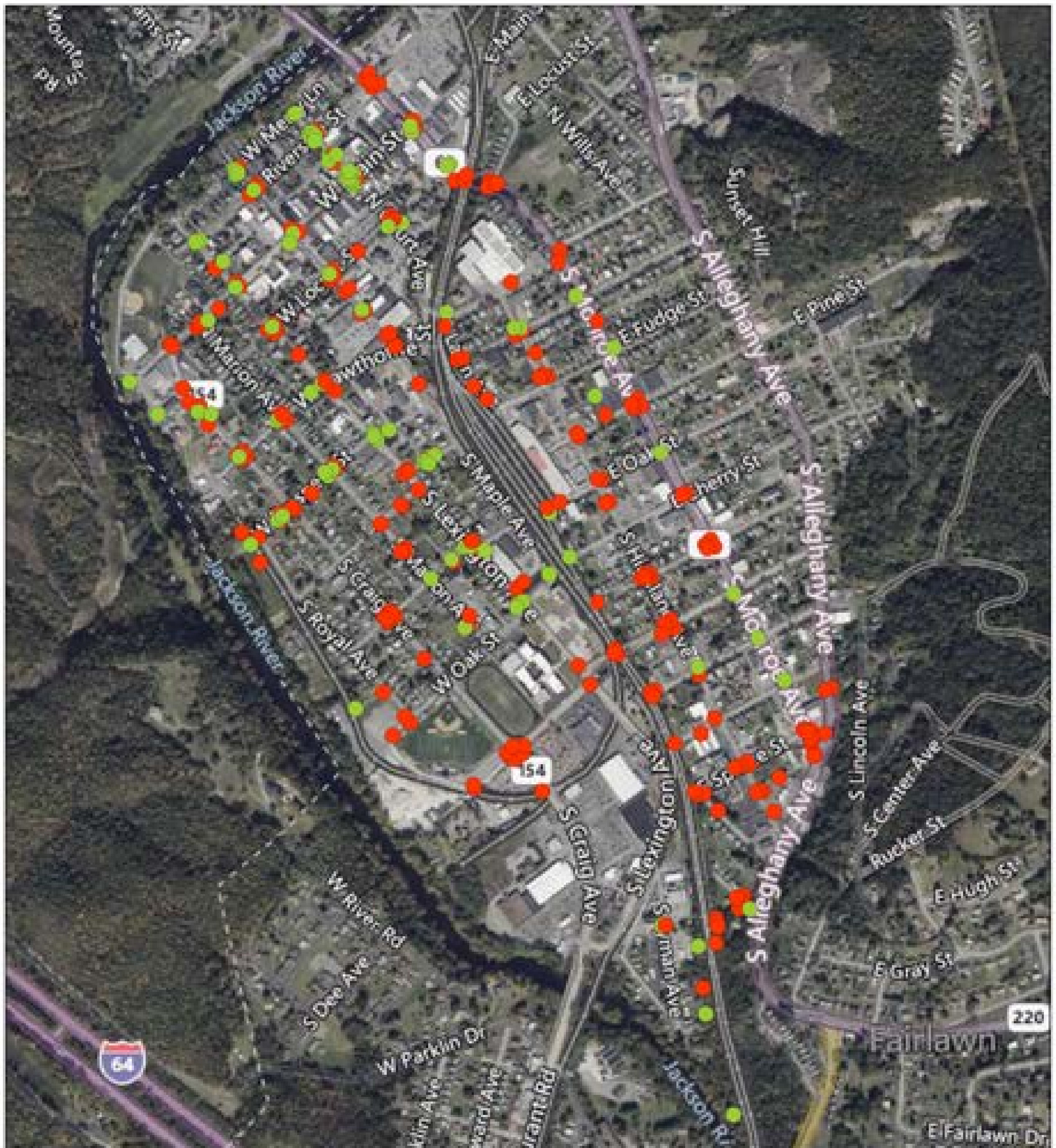
TOTAL FLOODED
JUNCTIONS = 219

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

100-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 2

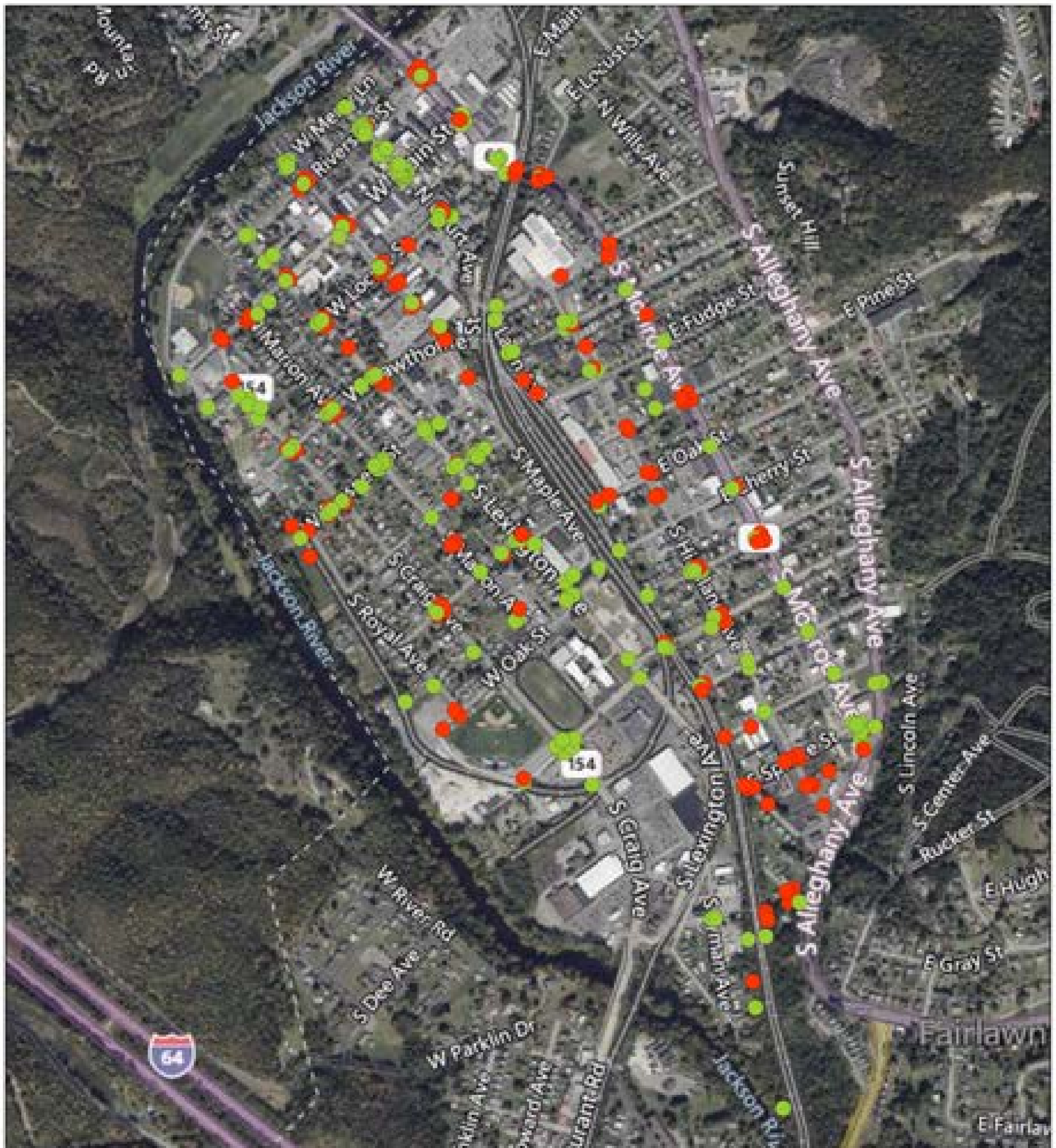
TOTAL FLOODED
JUNCTIONS = 226

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

2-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 3

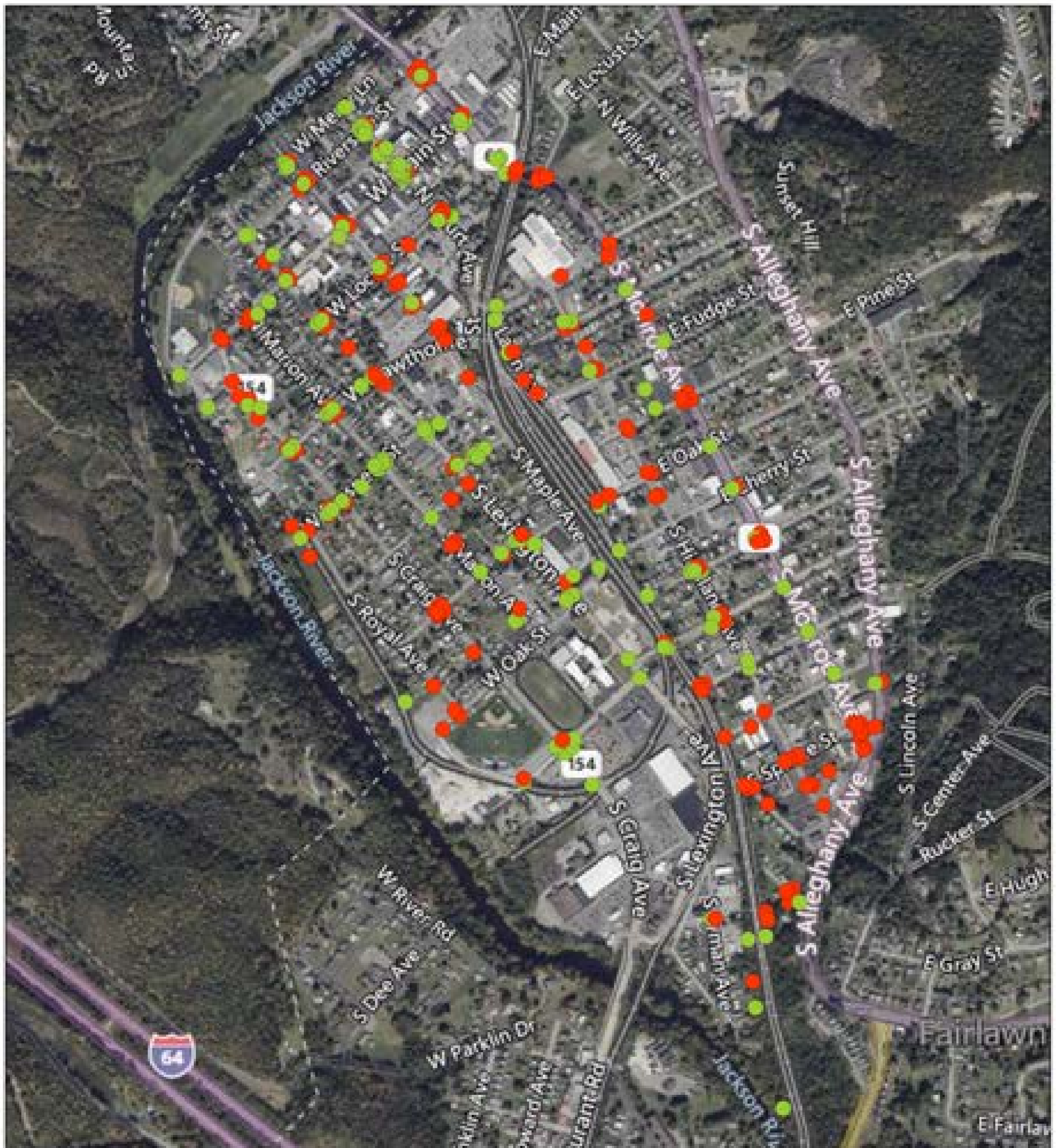
FLOODED JUNCTIONS
TOTAL = 146

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

10-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 3

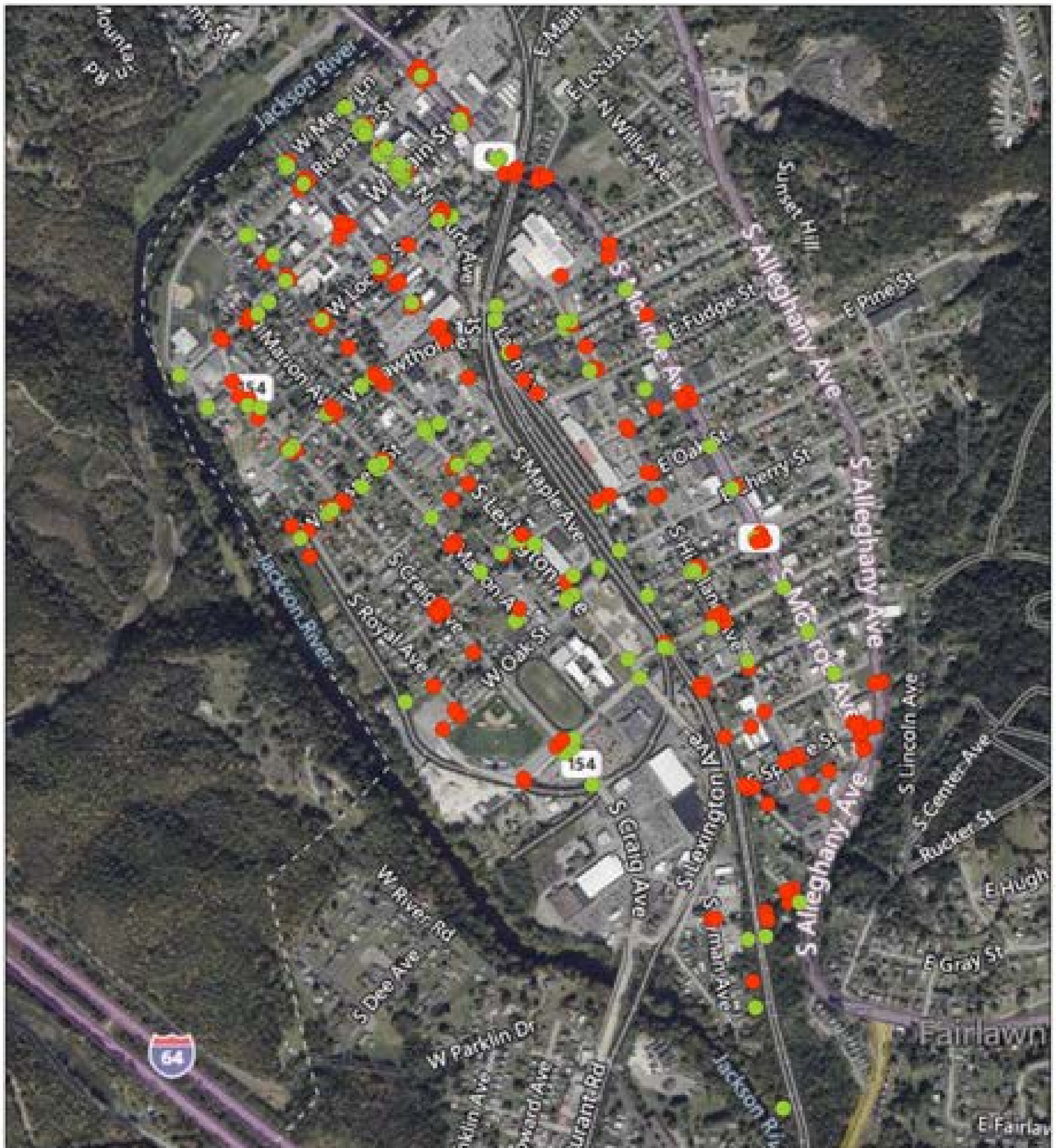
FLOODED JUNCTIONS
TOTAL = 183

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

25-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 3

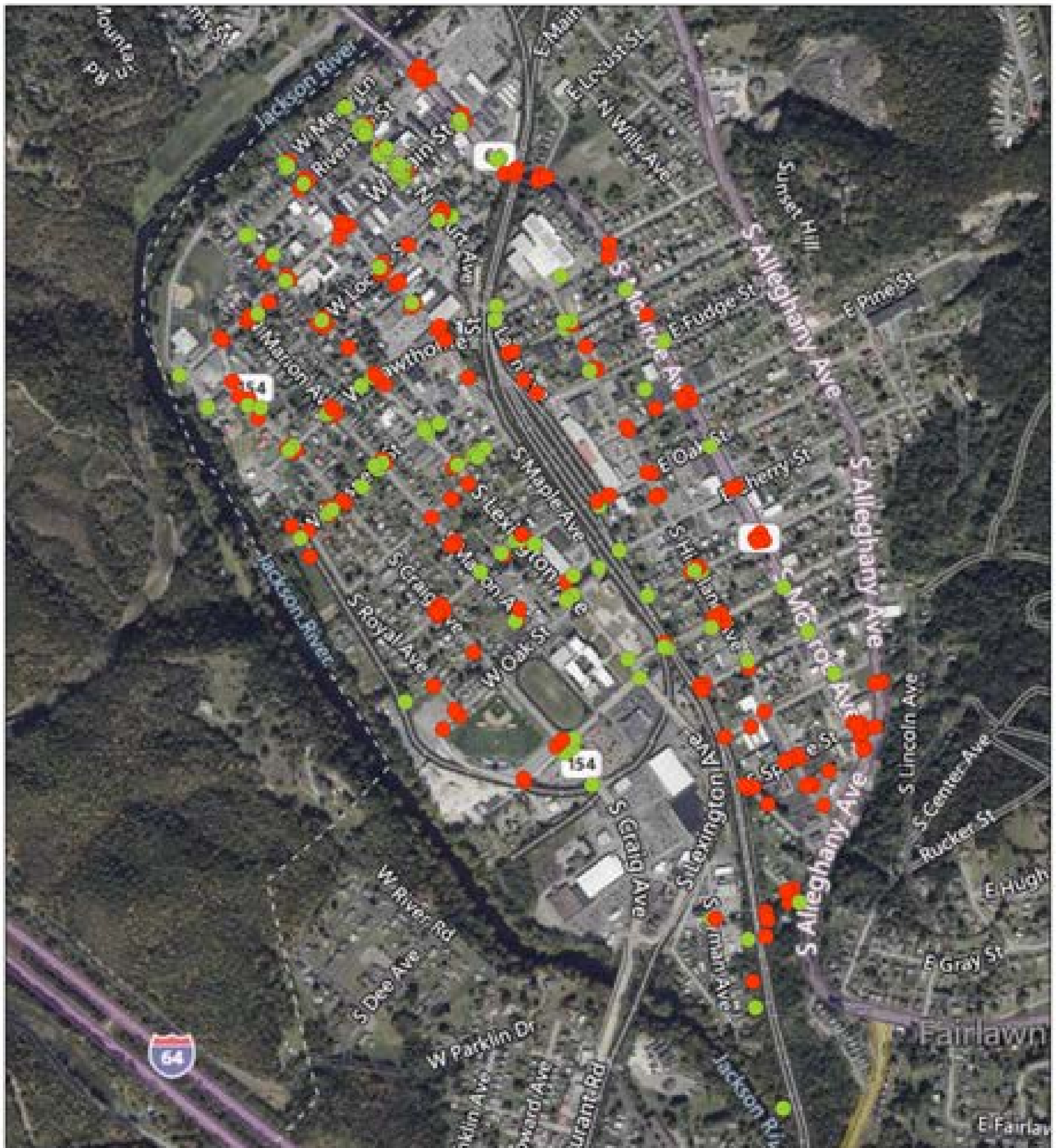
FLOODED JUNCTIONS
TOTAL = 198

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

50-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 3

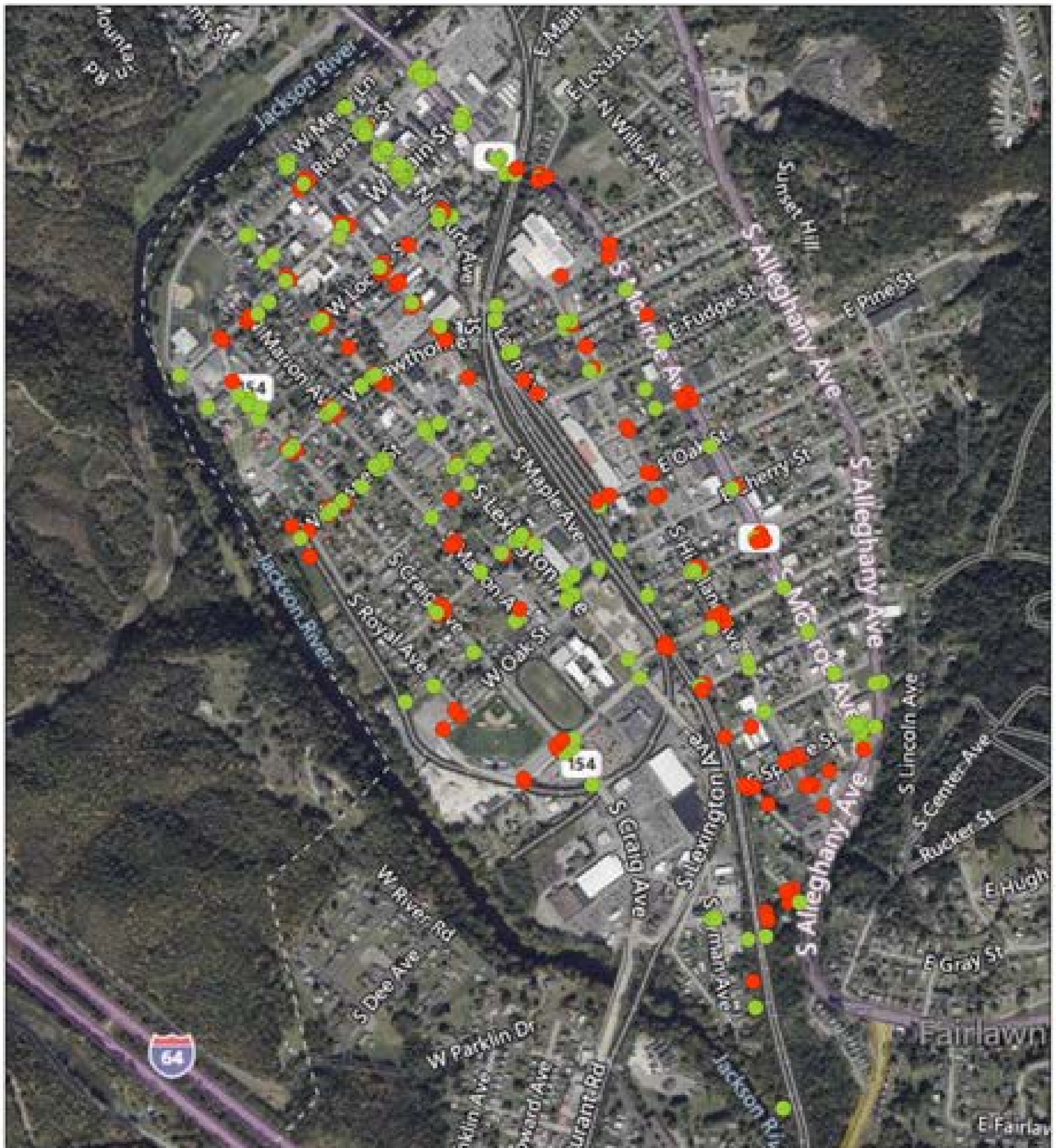
FLOODED JUNCTIONS
TOTAL = 203

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

2-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 4

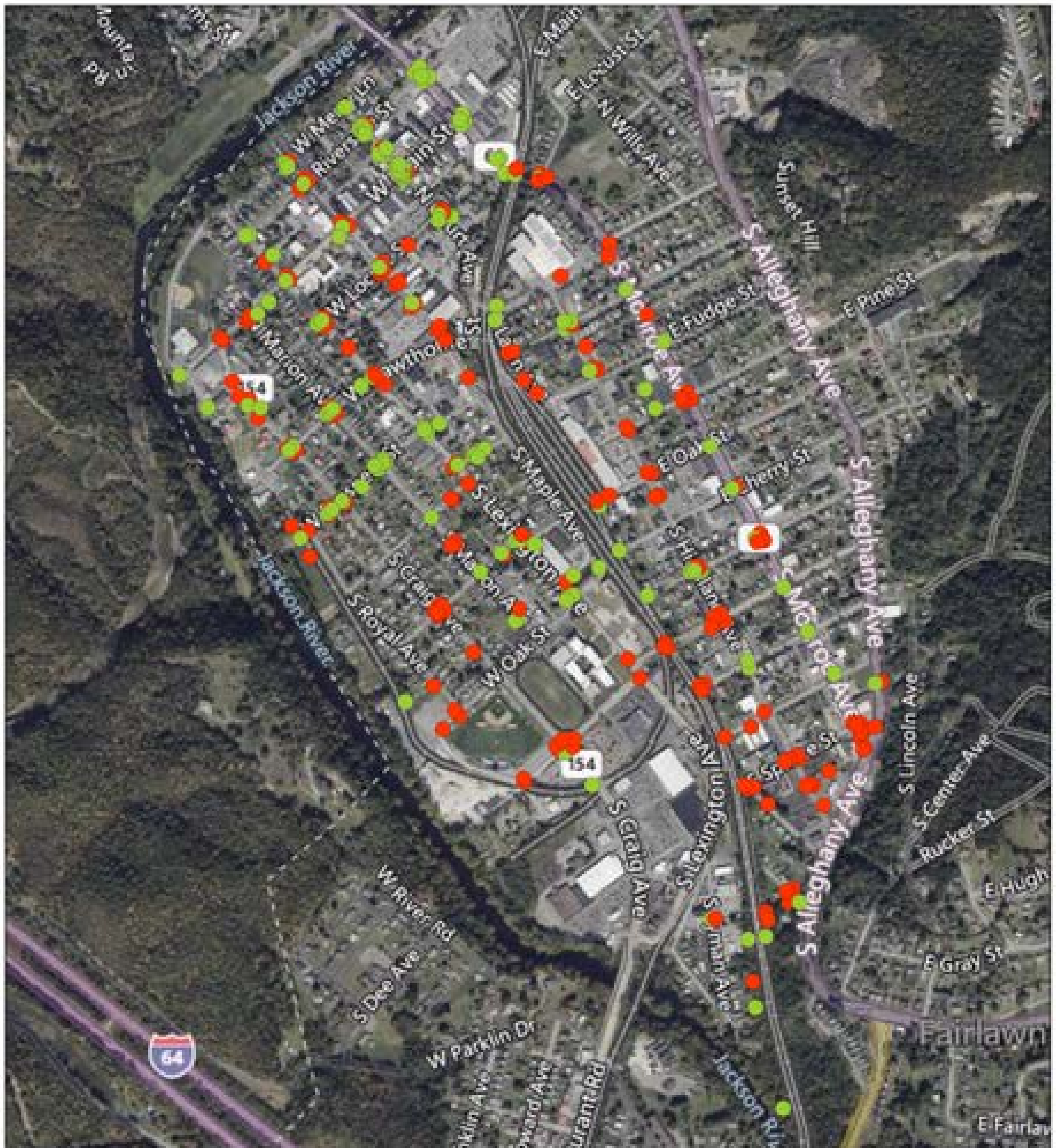
FLOODED JUNCTIONS
TOTAL = 149

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

Junctions

Flooded Junctions

10-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 4

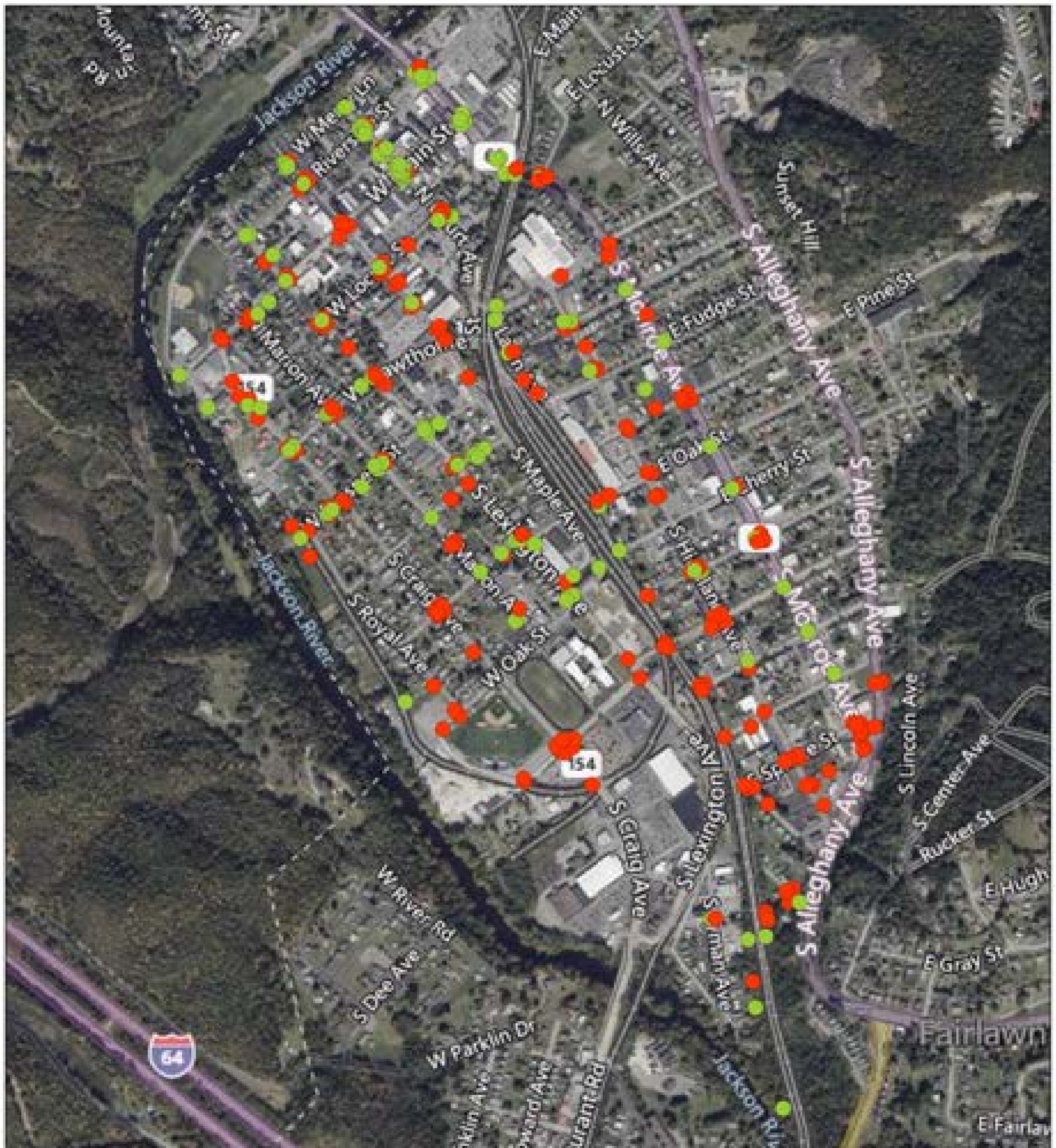
FLOODED JUNCTIONS
TOTAL = 186

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

25-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 4

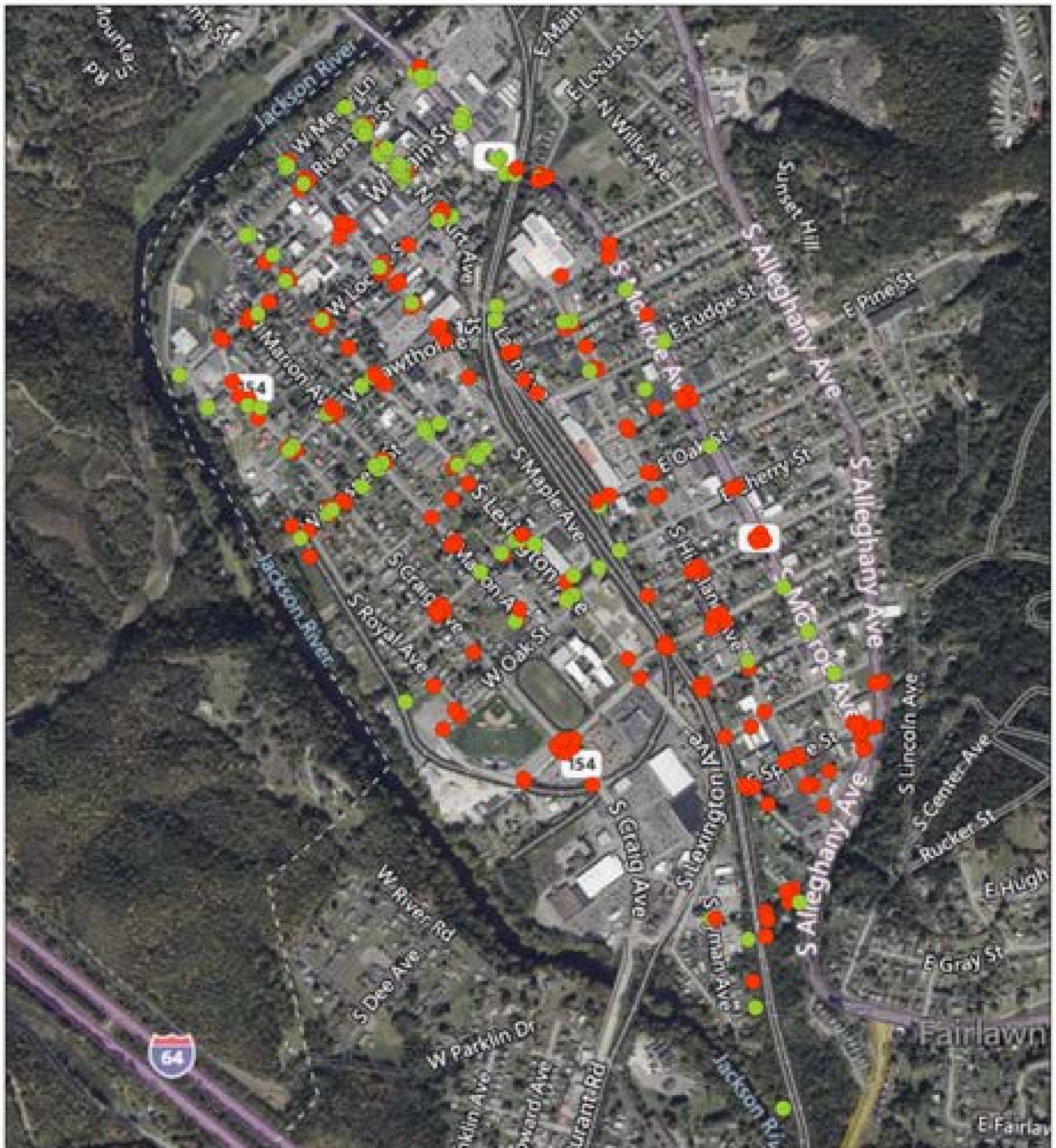
FLOODED JUNCTIONS
TOTAL = 202

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

50-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 4

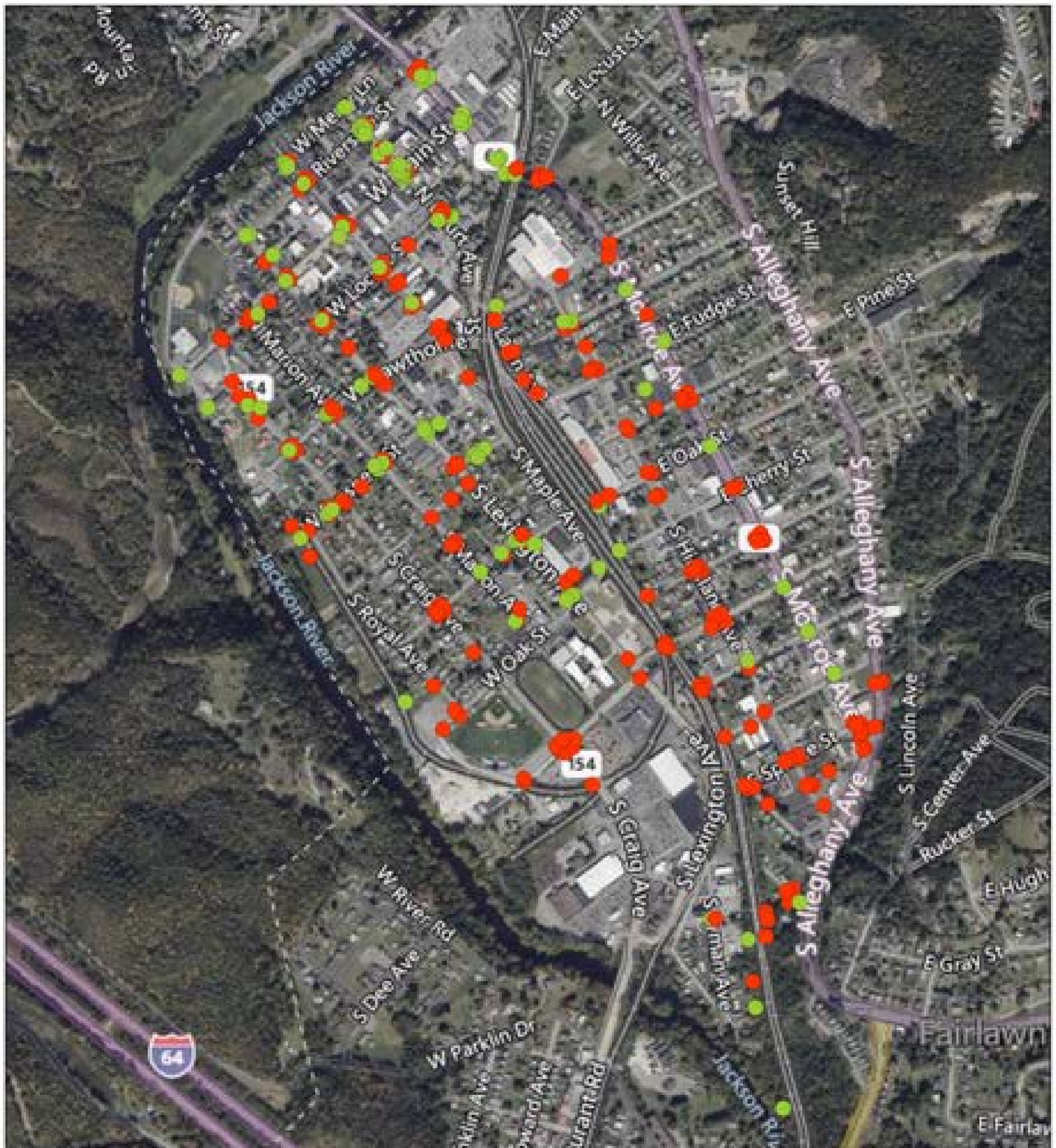
FLOODED JUNCTIONS
TOTAL = 212

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

100-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 4

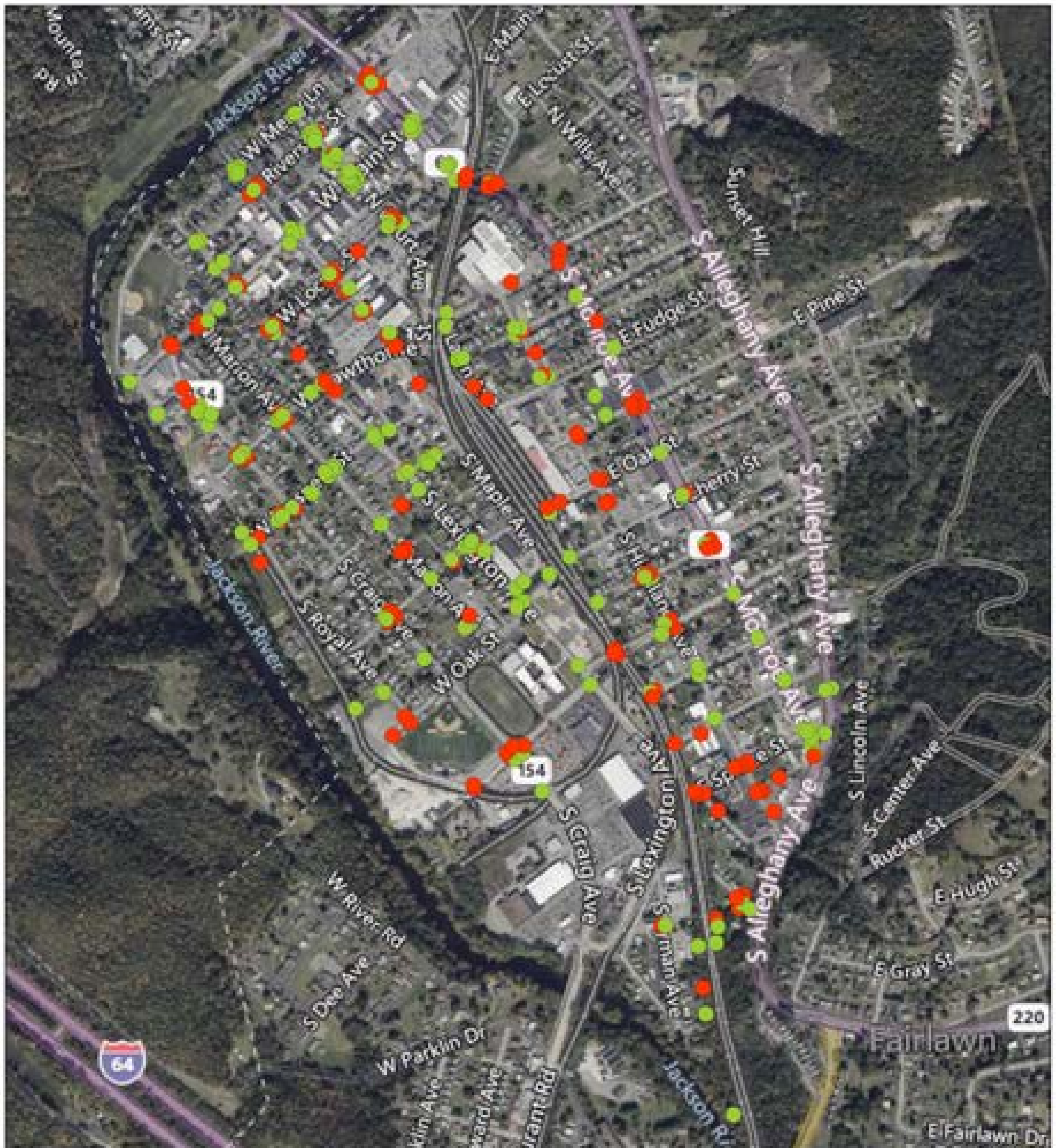
FLOODED JUNCTIONS
TOTAL = 220

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

2-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5A

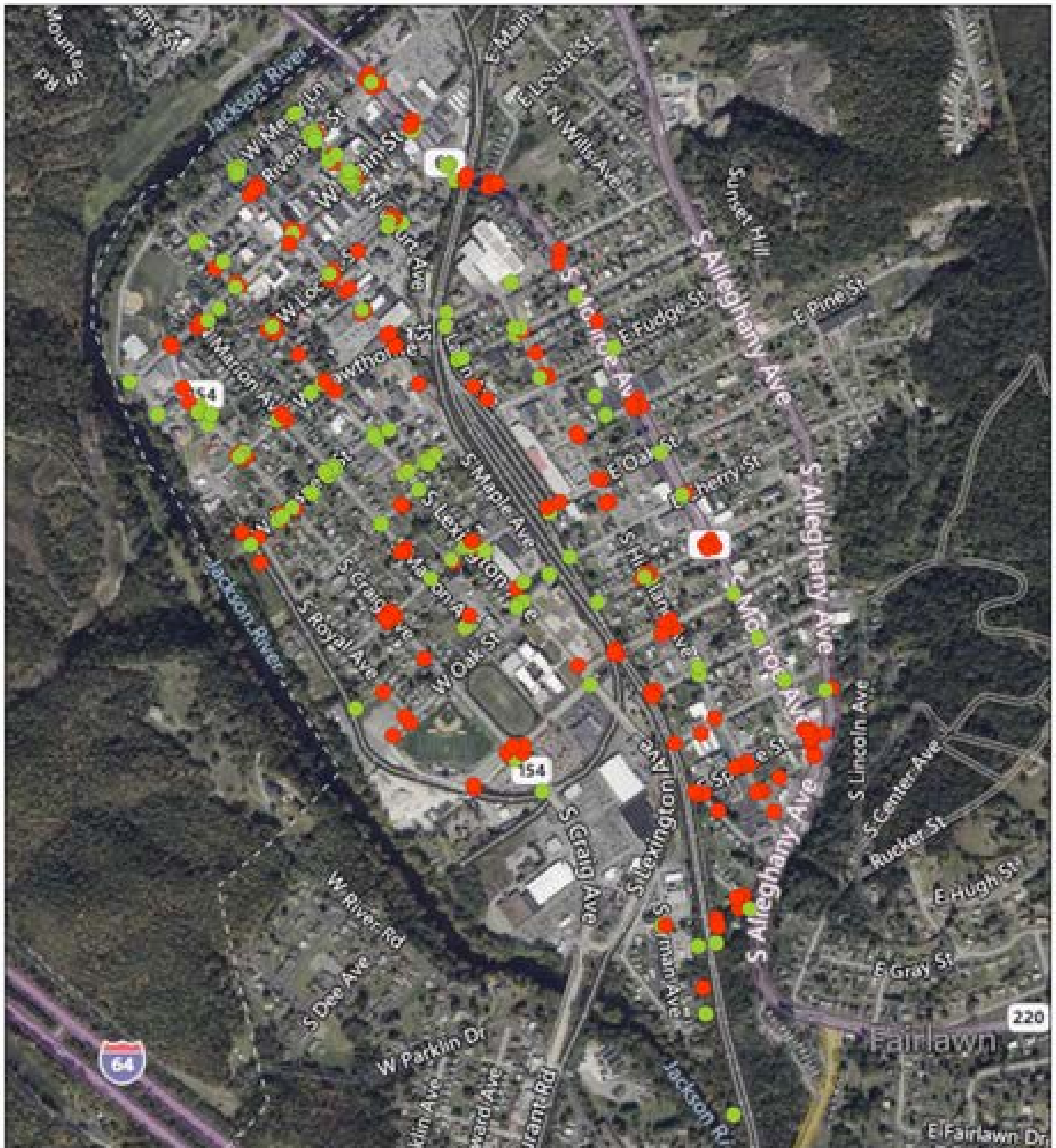
TOTAL FLOODED
JUNCTIONS = 150

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

10-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5A

TOTAL FLOODED
JUNCTIONS = 191

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft

Legend

Junctions

- Junctions
- Flooded Junctions

25-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5A

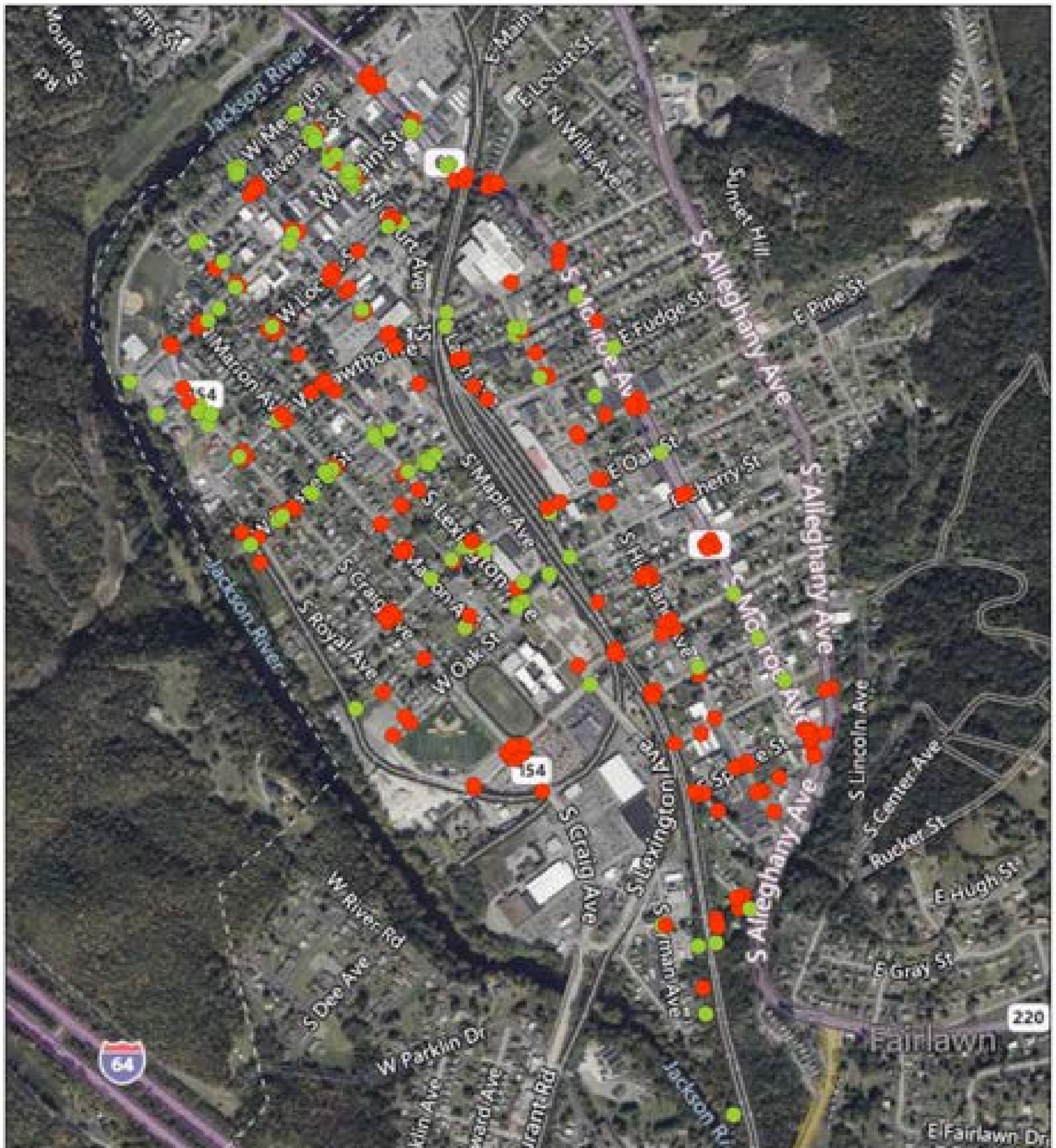
TOTAL FLOODED
JUNCTIONS = 204

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

50-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5A

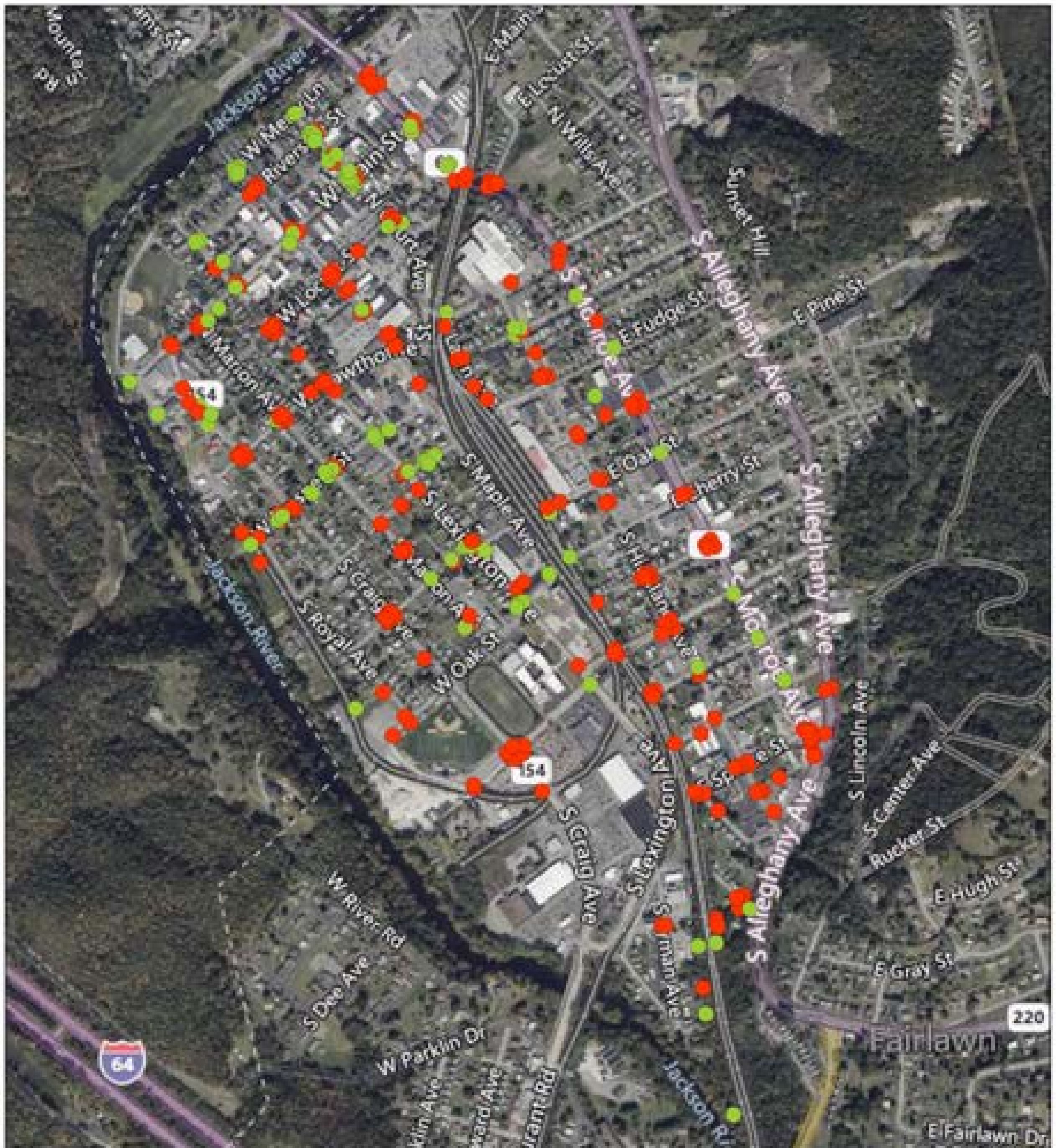
TOTAL FLOODED
JUNCTIONS = 216

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

100-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5A

TOTAL FLOODED
JUNCTIONS = 226

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft

Legend

Junctions

- Junctions
- Flooded Junctions

2-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5B

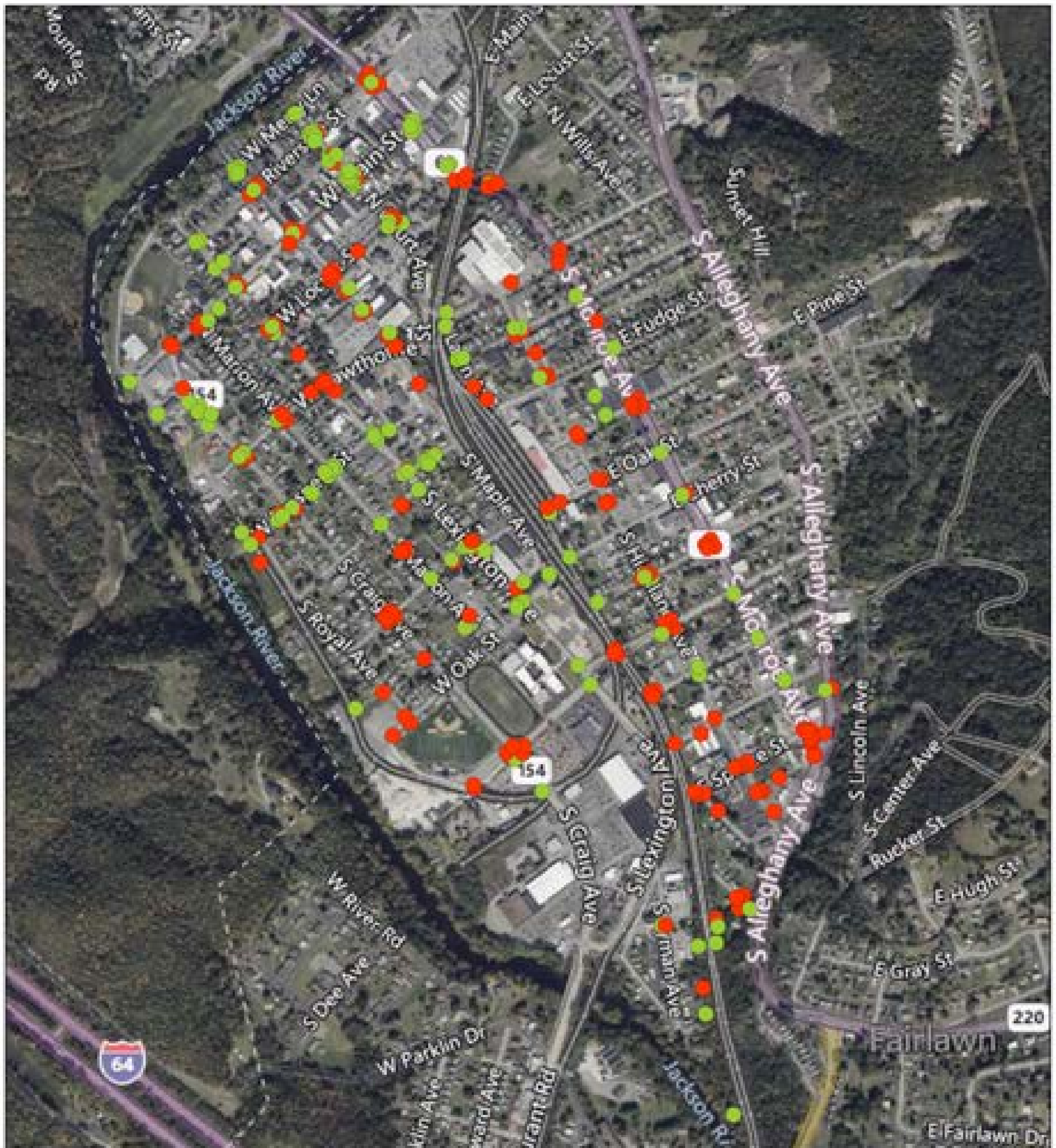
TOTAL FLOODED
JUNCTIONS = 149

City of Covington, Virginia
Drainage Study

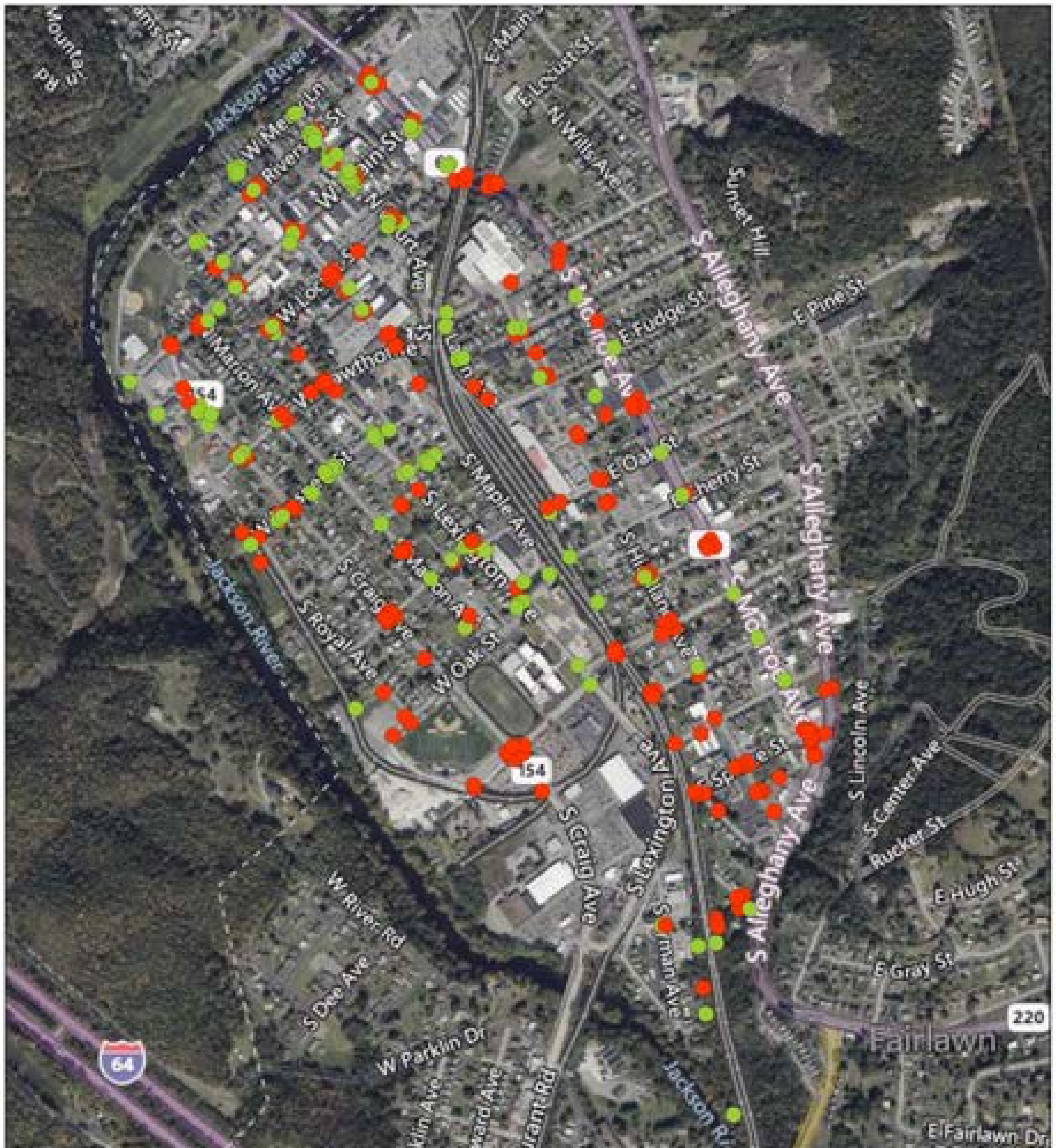
AECOM

N

500 ft



Legend			
Junctions ● Junctions ● Flooded Junctions	10-YEAR 24-HOUR STORM EVENT PROPOSED SCENARIO 5B TOTAL FLOODED JUNCTIONS = 183 City of Covington, Virginia Drainage Study	AECOM	N 500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

25-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5B

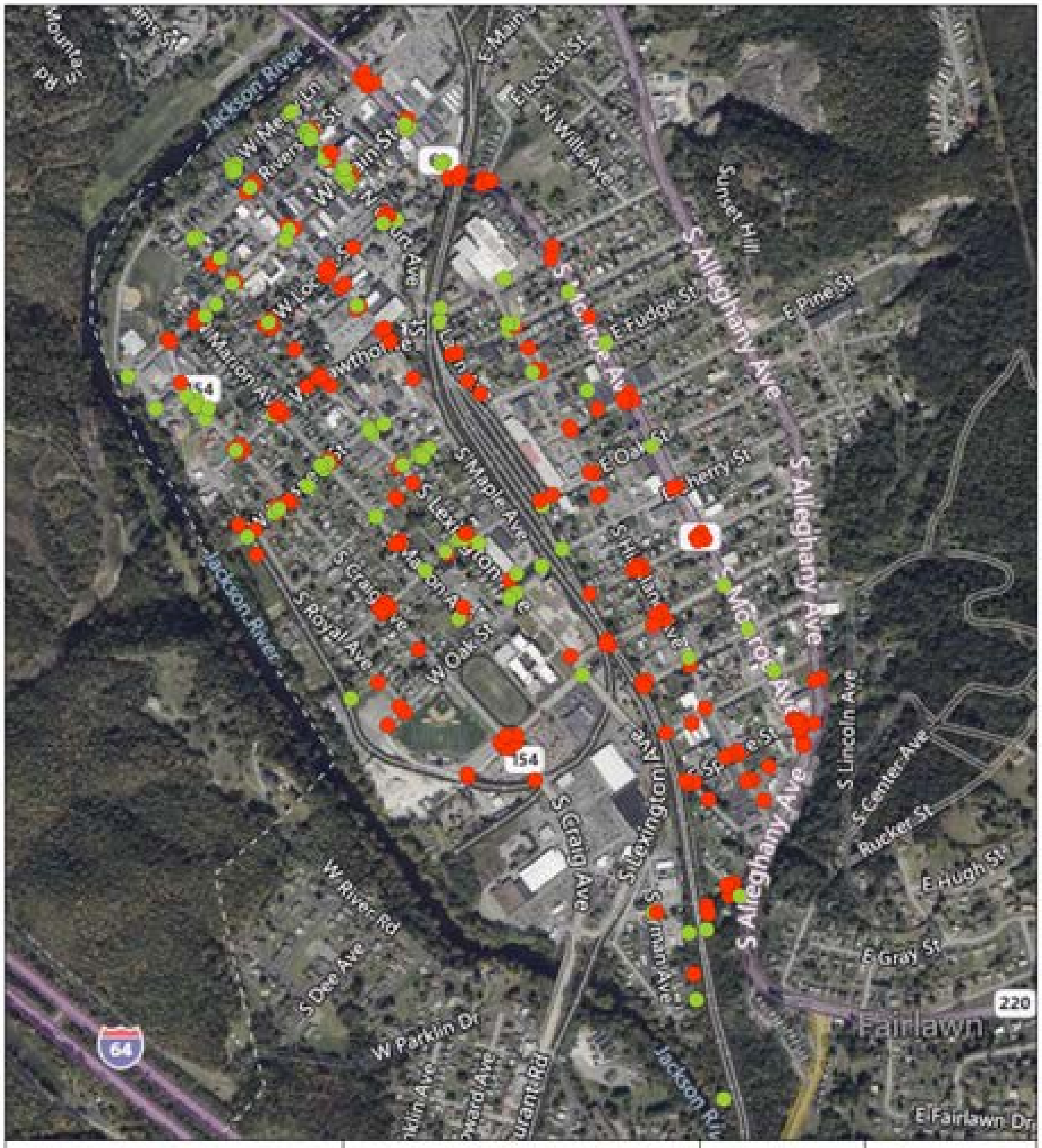
TOTAL FLOODED
JUNCTIONS = 201

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

50-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5B

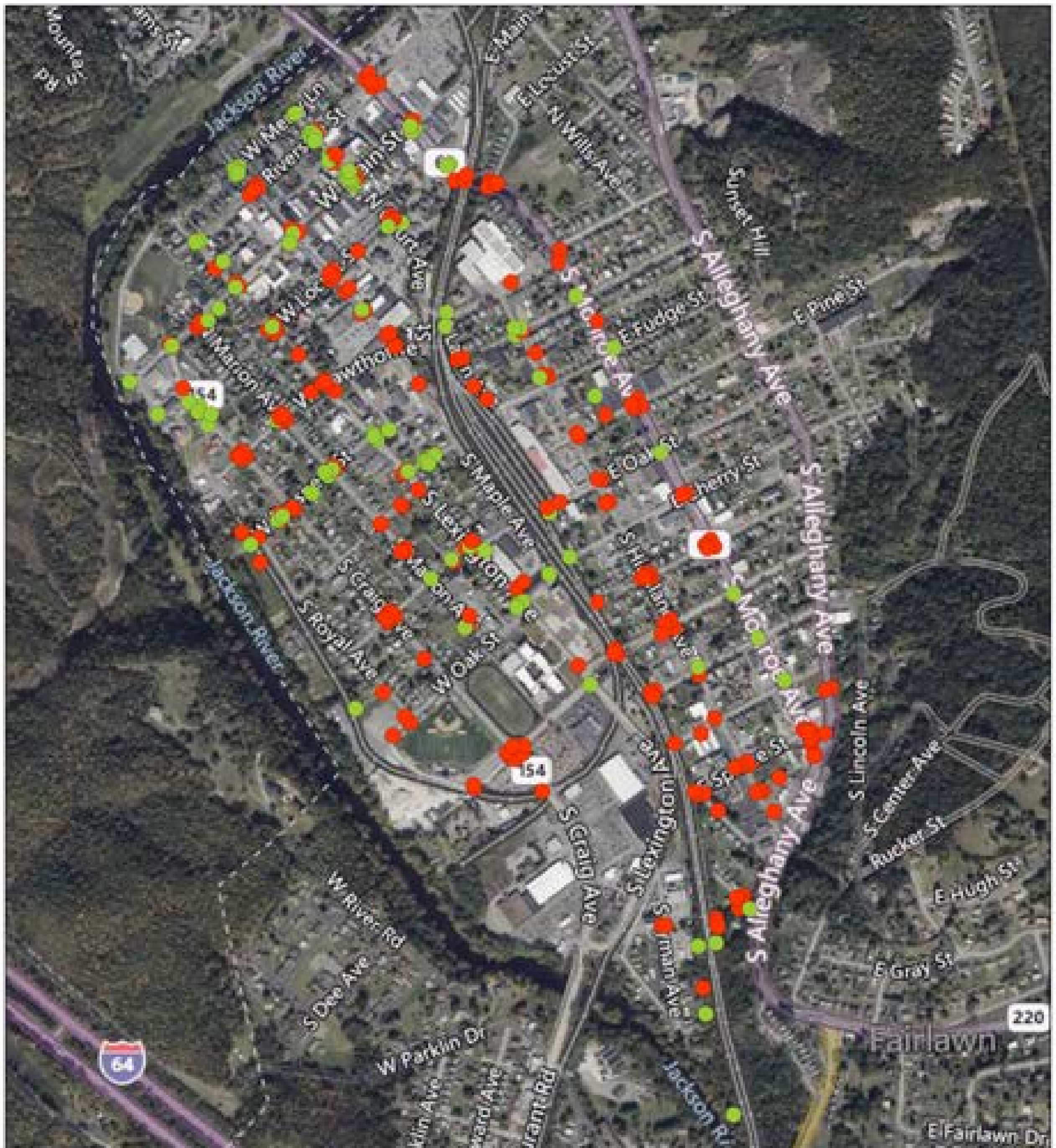
TOTAL FLOODED
JUNCTIONS = 215

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

100-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5B

TOTAL FLOODED
JUNCTIONS = 219

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft

Legend

Junctions

- Junction
- Flooded Junctions

2-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5C

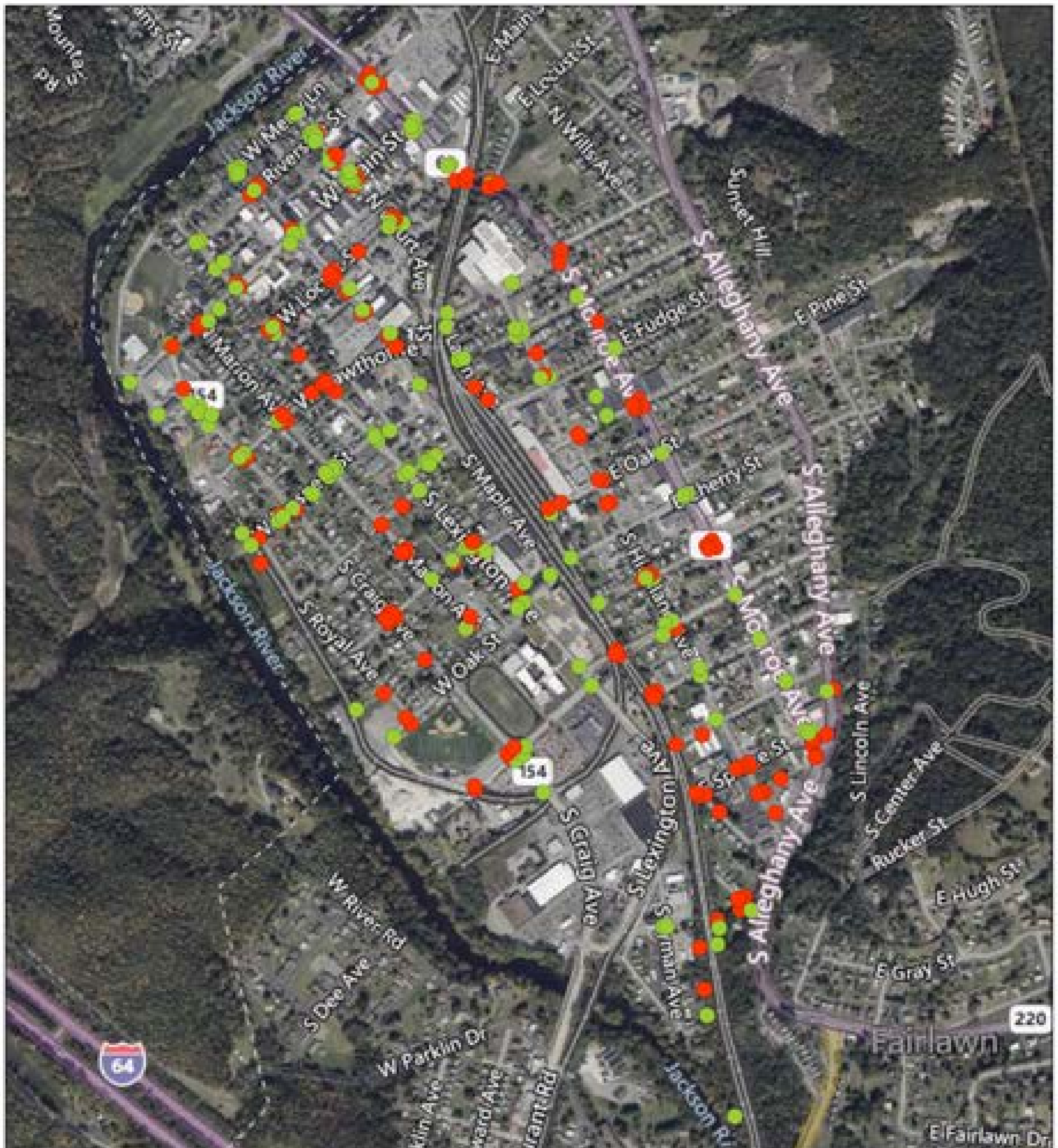
TOTAL FLOODED
JUNCTIONS = 109

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junction
- Flooded Junctions

10-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5C

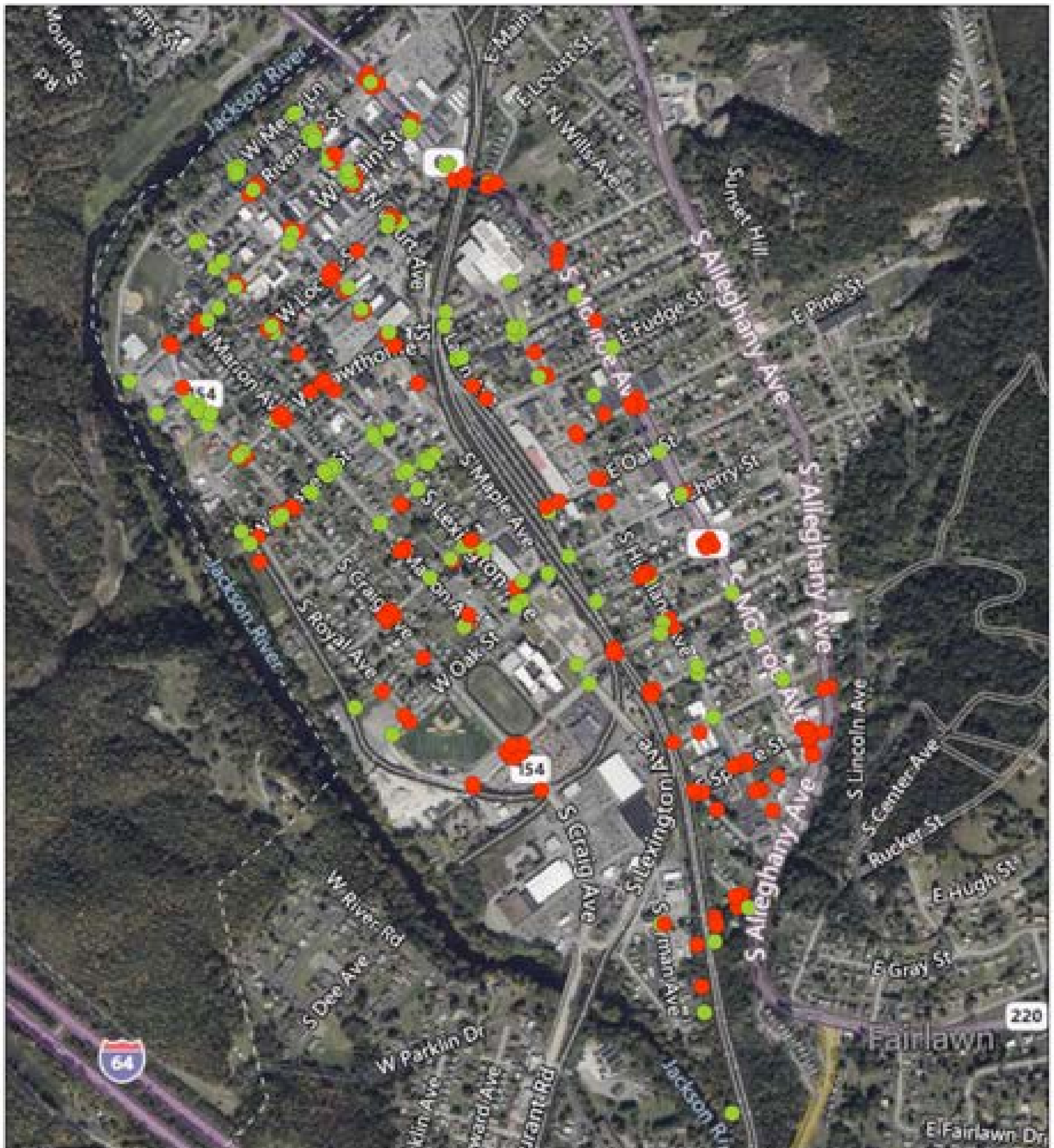
TOTAL FLOODED
JUNCTIONS = 169

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junction
- Flooded Junctions

25-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5C

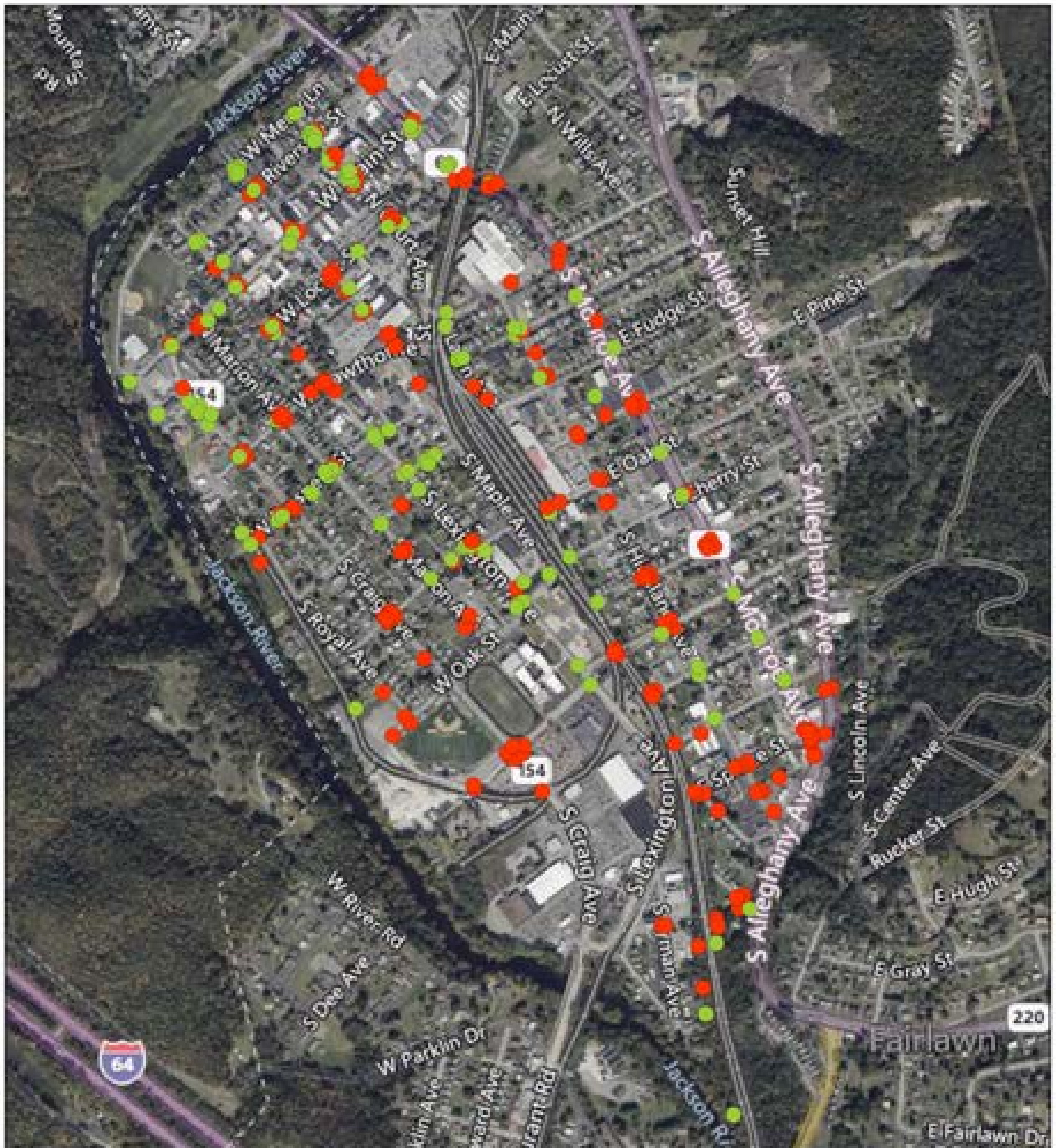
TOTAL FLOODED
JUNCTIONS = 191

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junction
- Flooded Junctions

50-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5C

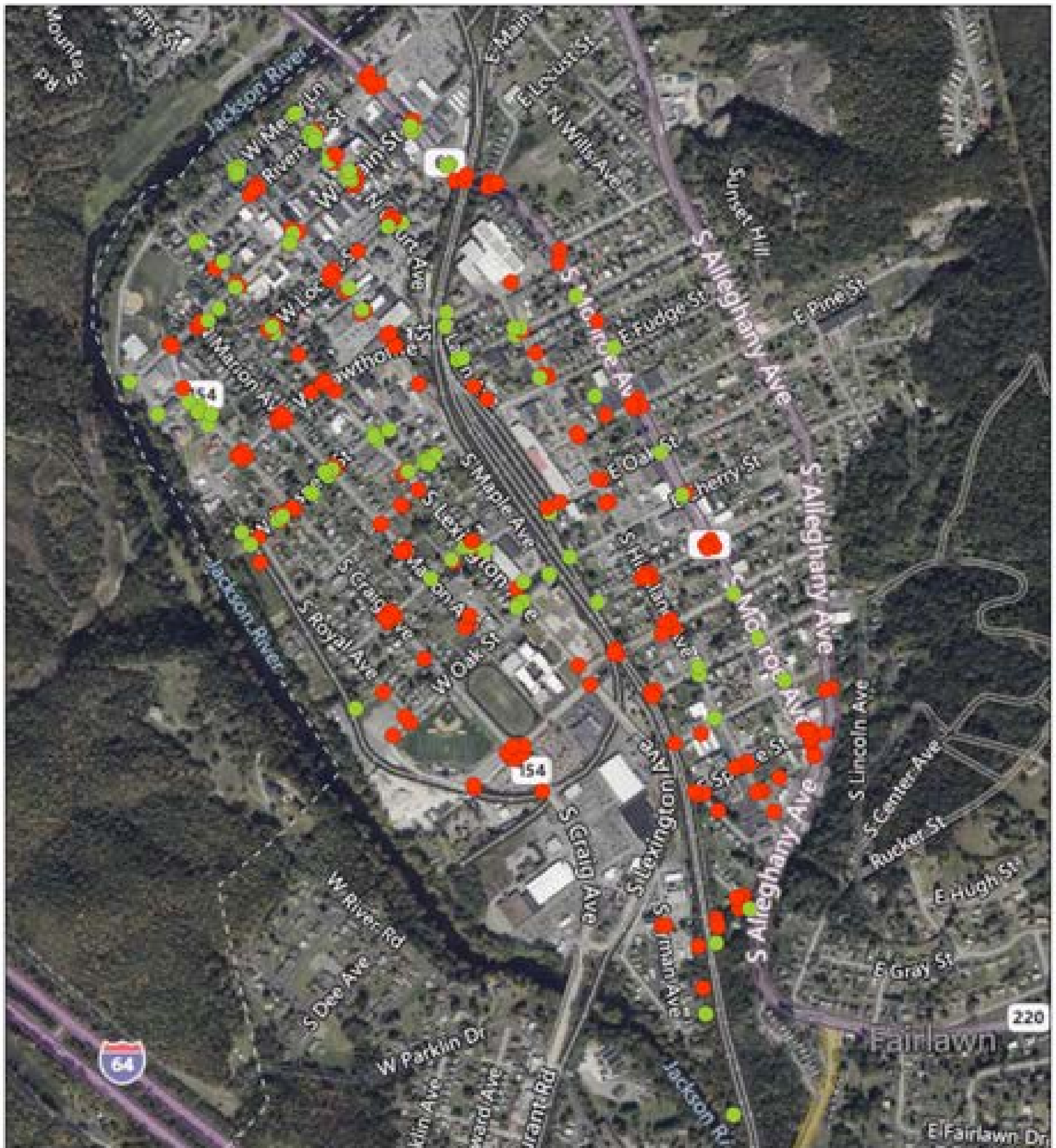
TOTAL FLOODED
JUNCTIONS = 204

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junction
- Flooded Junctions

100-YEAR 24-HOUR STORM EVENT
PROPOSED SCENARIO 5C

TOTAL FLOODED
JUNCTIONS = 218

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft

Appendix E Climate Change Impact on Stormwater Networks Map Exhibits

Legend

Junctions

Flooded Junctions

Junctions

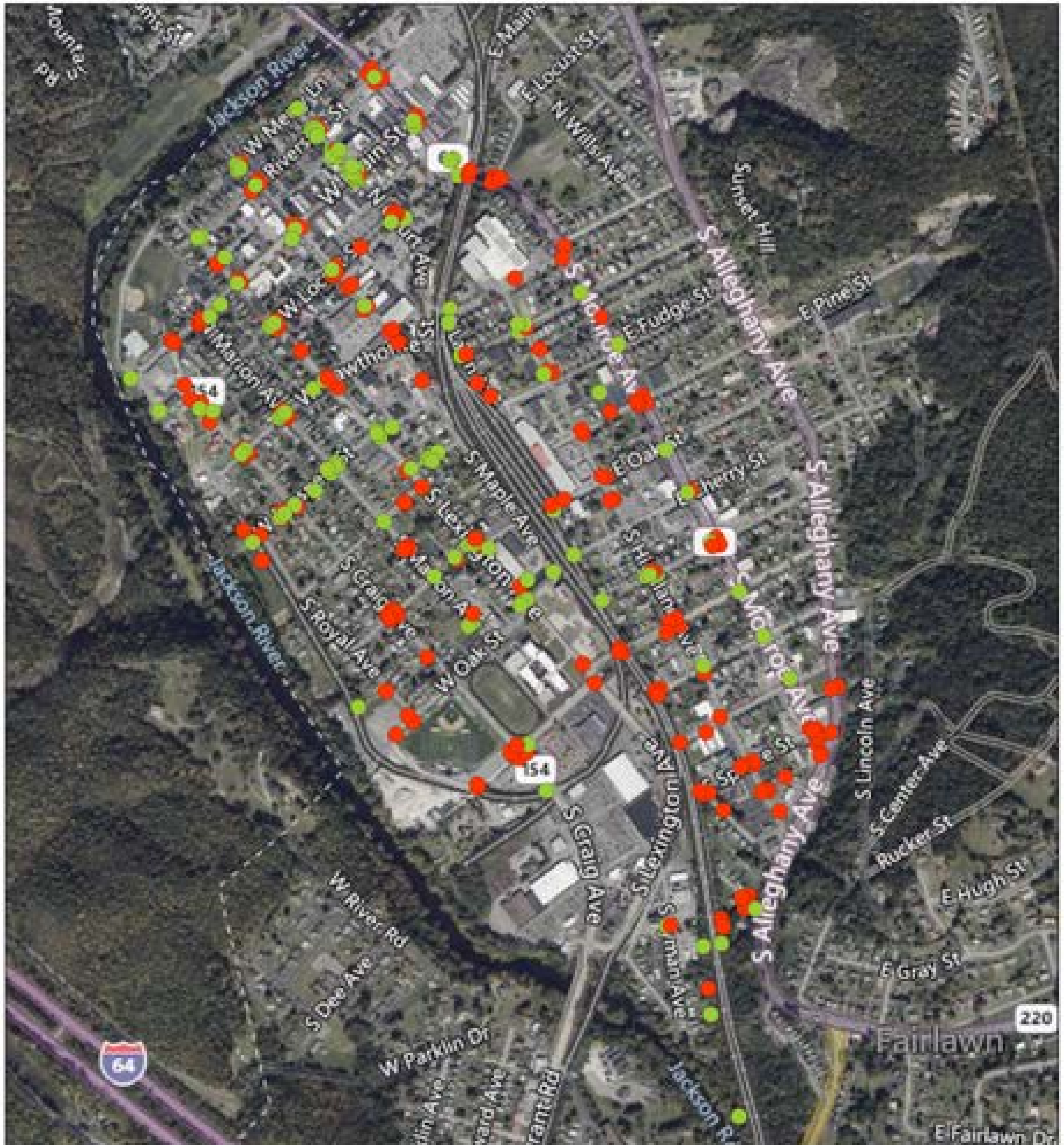
10-YEAR 24-HOUR STORM EVENT
CLIMATE CHANGE SCENARIO
BASE MODEL

TOTAL FLOODED JUNCTION = 200

City of Covington, Virginia
Drainage Study

AECOM

500 ft



Legend

Junctions

- Junction
- Flooded Junction

10-YEAR 24-HOUR STORM EVENT
CLIMATE CHANGE SCENARIO
SCENARIO 1

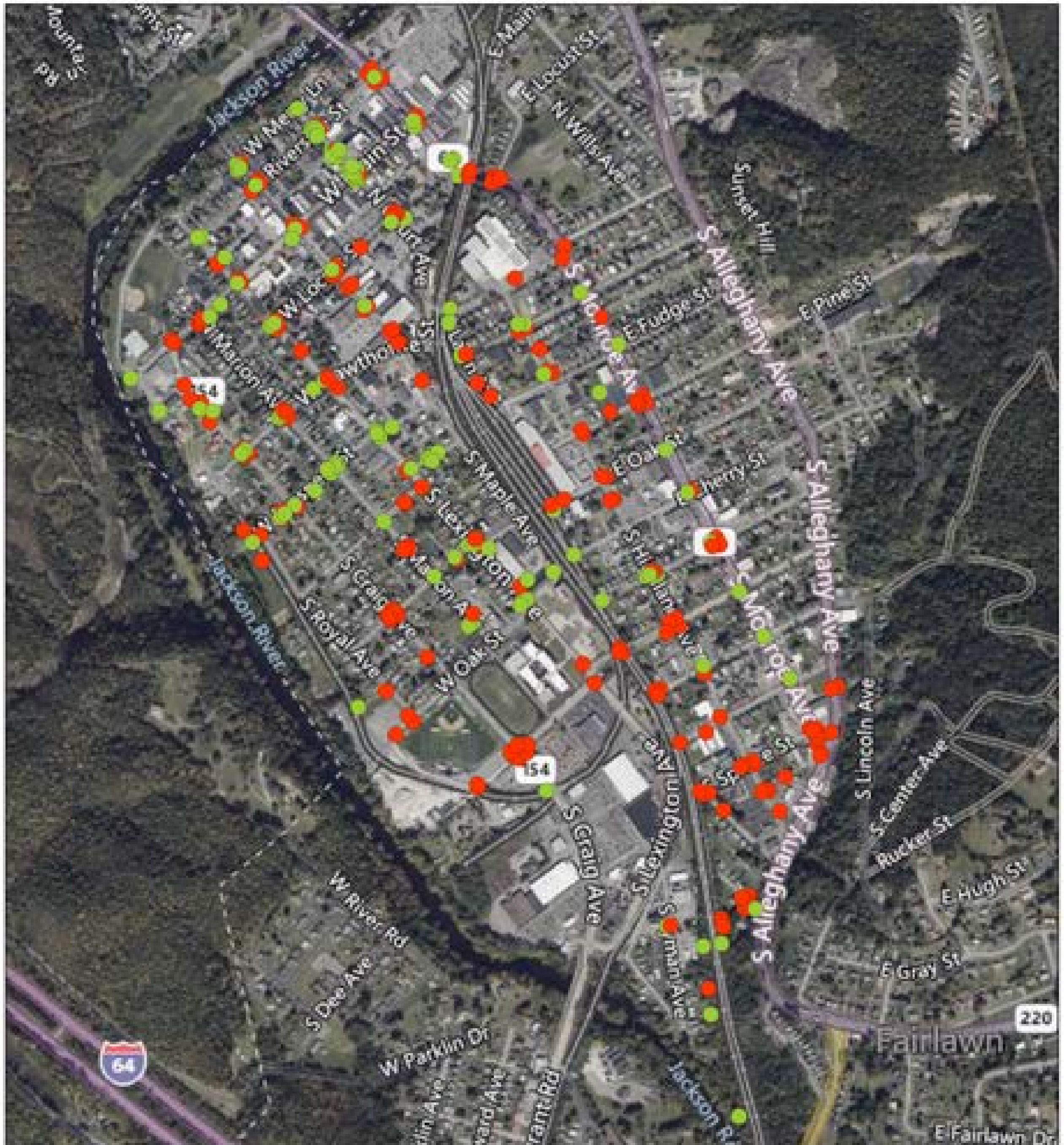
TOTAL FLOODED JUNCTIONS = 197

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junction
- Flooded Junction

10-YEAR 24-HOUR STORM EVENT
CLIMATE CHANGE SCENARIO
SCENARIO 2

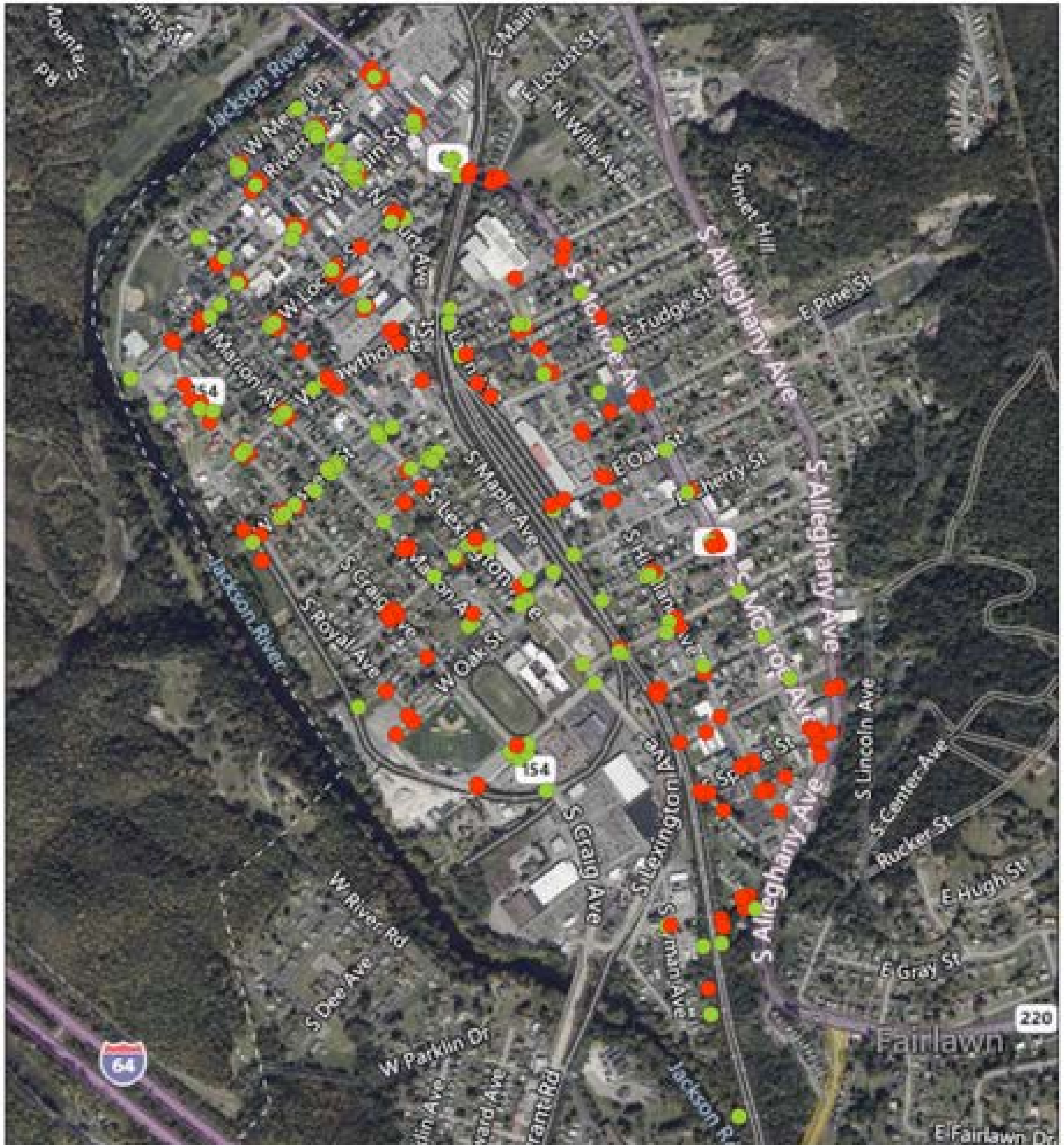
TOTAL FLOODED JUNCTIONS = 198

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

10-YEAR 24-HOUR STORM EVENT
CLIMATE CHANGE SCENARIO
SCENARIO 3

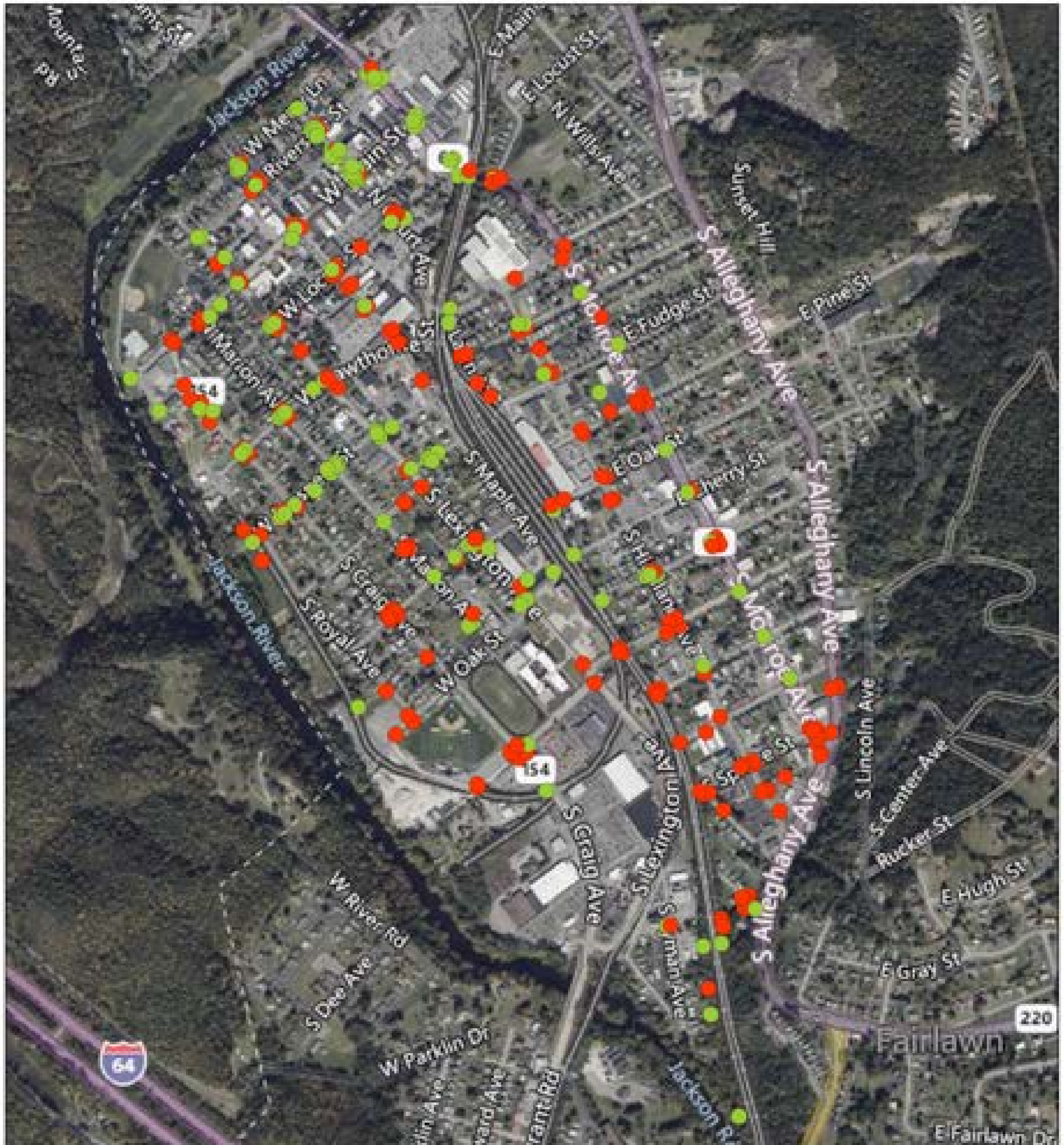
TOTAL FLOODED JUNCTIONS = 188

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

10-YEAR 24-HOUR STORM EVENT
CLIMATE CHANGE SCENARIO
SCENARIO 4

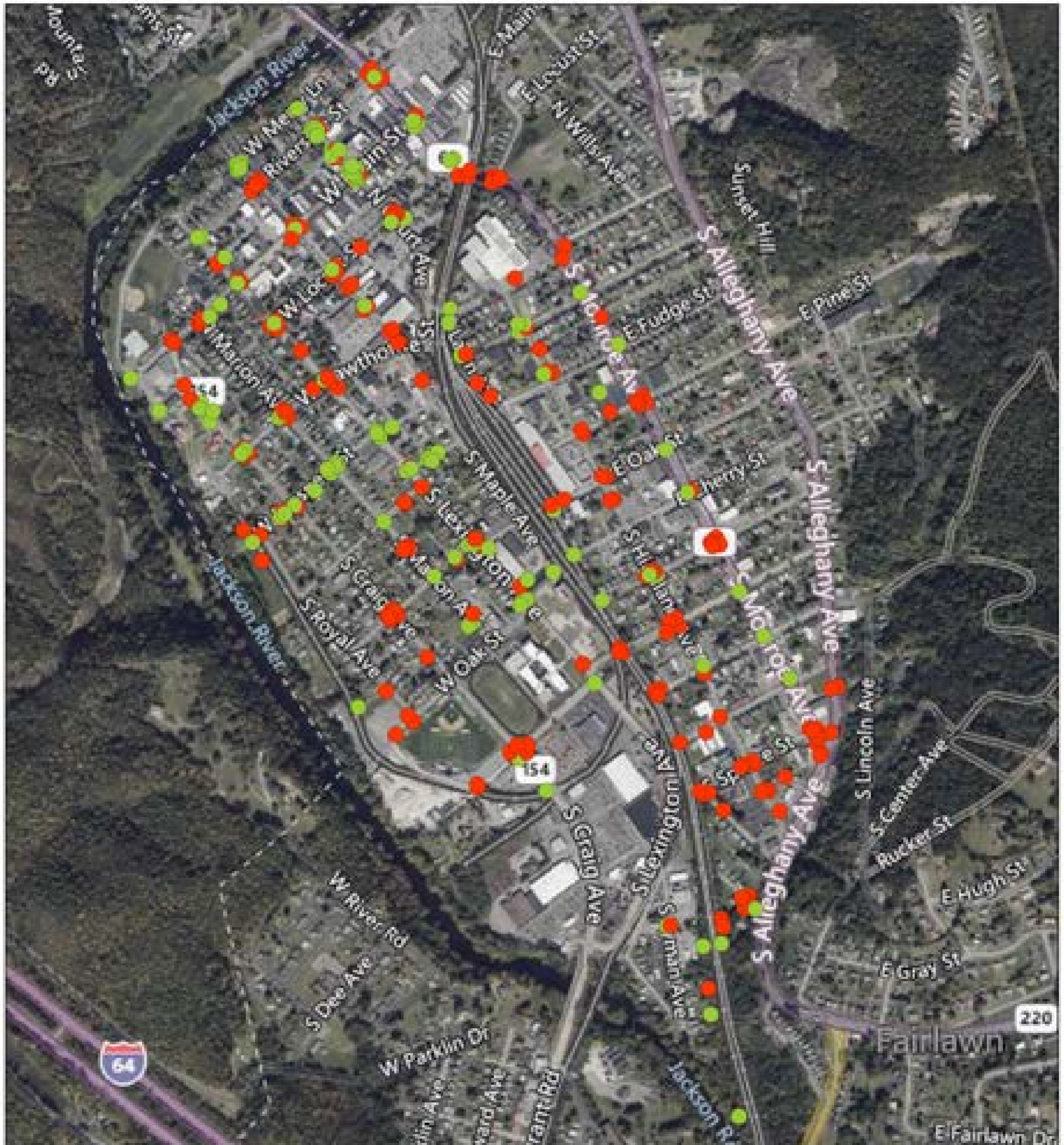
TOTAL FLOODED JUNCTIONS = 192

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

10-YEAR 24-HOUR STORM EVENT
CLIMATE CHANGE SCENARIO
SCENARIO 5A

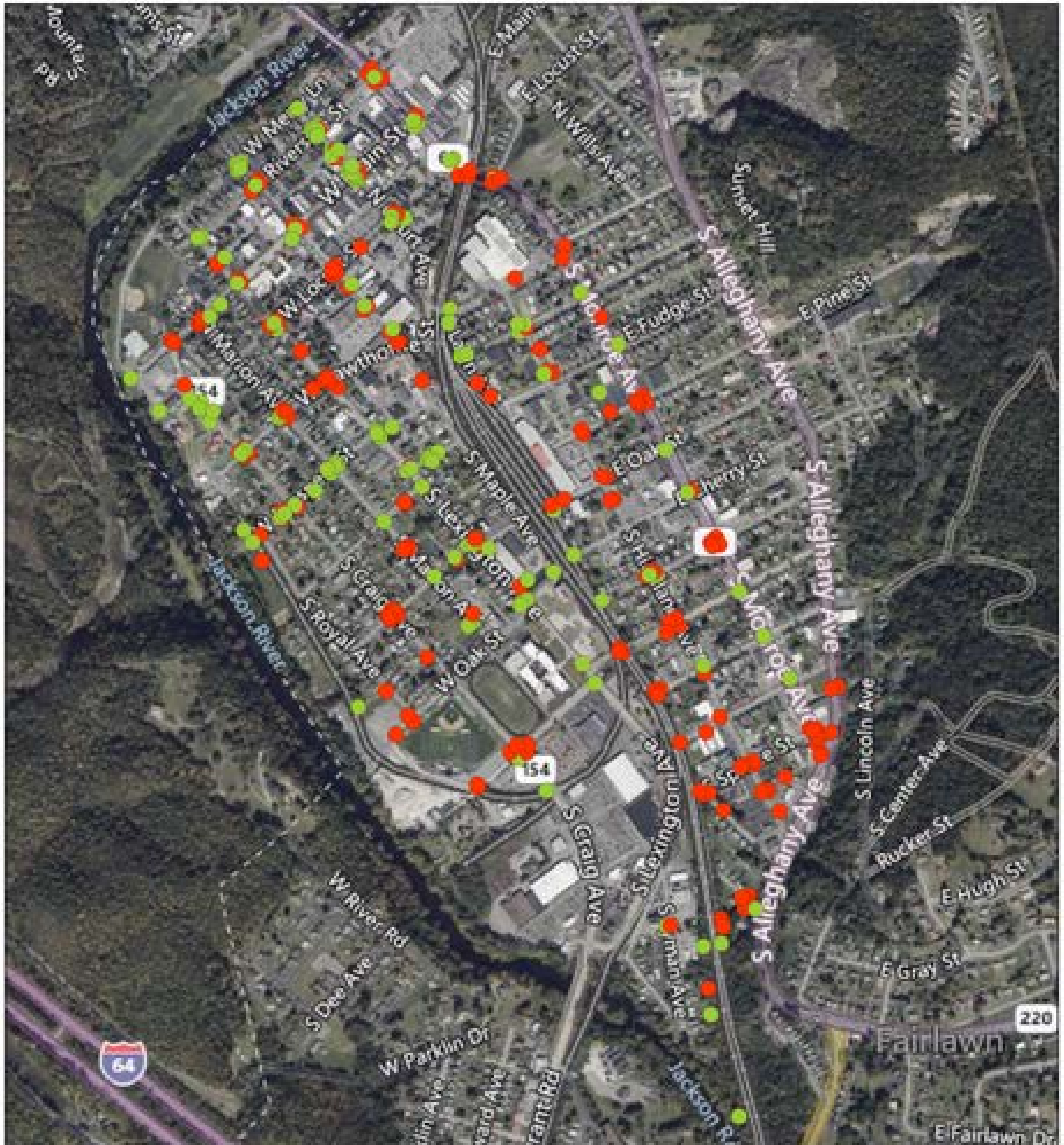
TOTAL FLOODED JUNCTIONS = 200

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junctions
- Flooded Junctions

10-YEAR 24-HOUR STORM EVENT
CLIMATE CHANGE SCENARIO
SCENARIO 5B

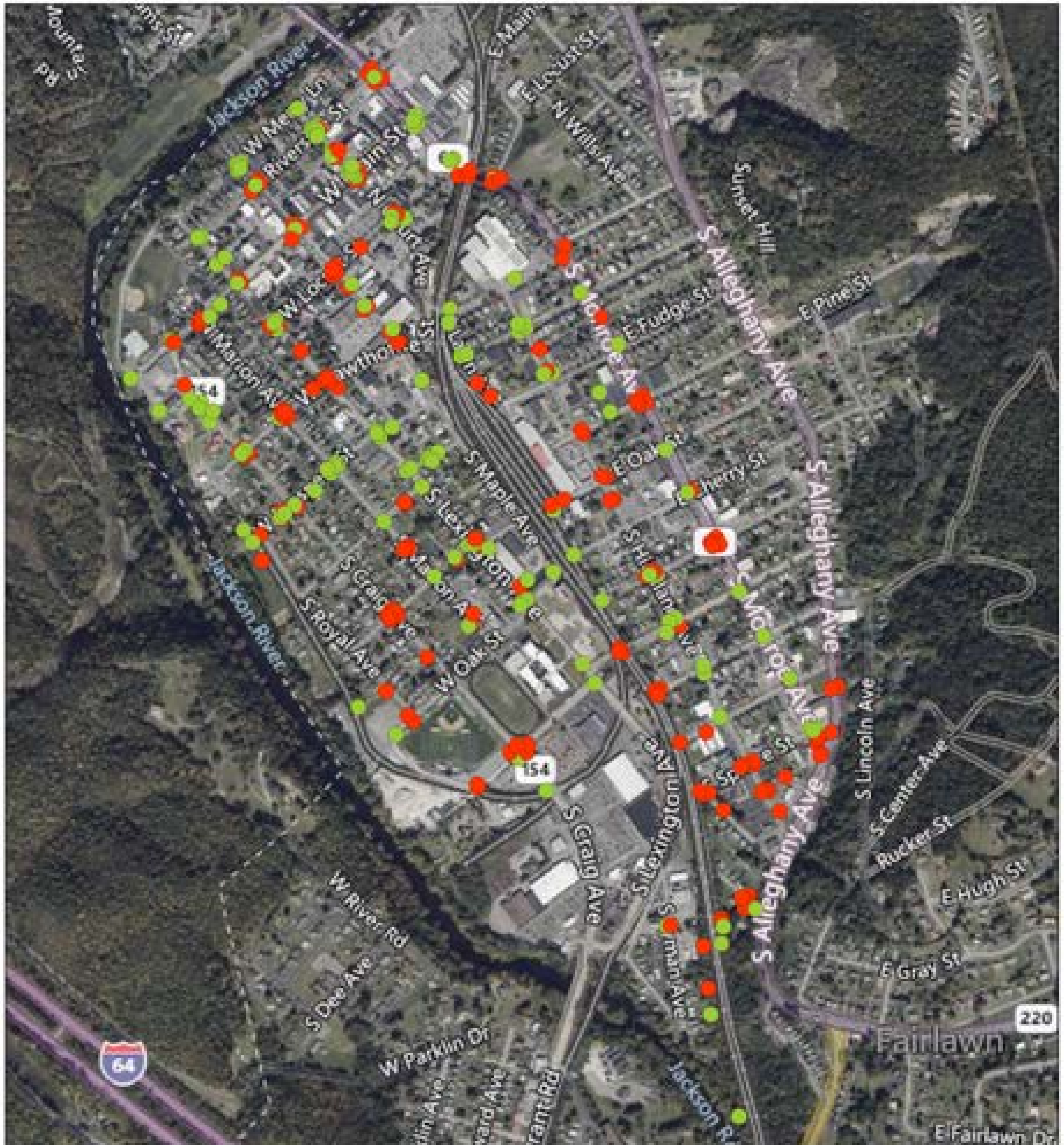
TOTAL FLOODED JUNCTIONS = 189

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft



Legend

Junctions

- Junction
- Flooded Junctions

10-YEAR 24-HOUR STORM EVENT
CLIMATE CHANGE SCENARIO
SCENARIO 5C

TOTAL FLOODED JUNCTIONS = 176

City of Covington, Virginia
Drainage Study

AECOM

N

500 ft

aecom.com