Harrington Drainage Basin Study

Volume II of II

Prepared For:

City of Memphis
Division of Engineering

Prepared By:

Smith Seckman Reid, Inc.
Geodesy Professional Services, LLC
Younger and Associates, LLC

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Smith Seckman Reid, Inc.
2650 Thousand Oaks Boulevard, Suite 4200, Memphis, Tennessee 38118
Tel: 901.683.3900 · Fax: 901.683.3990 · www.ssr-inc.com

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Abbreviation:	**Meaning:**
(CEASER)	Center for Applied Earth Science and Engineering Research
(CIP)	Capital Improvement Project
(CMP)	Corrugated Metal Pipe
(DEM)	Digital Elevation Model
(DTV)	Difference in Total Volume
(FHWA)	Federal Highway Administration
(GIS)	Geographic Information System
(GPS)	Global Positioning System
(HDS-5)	Hydraulic Design Series Number 5
(HERCP)	Horizontal Elliptical Reinforced Concrete Pipe
(HGL)	Hydraulic Grade Line
(H&H)	Hydrologic and Hydraulic
(LID)	Low Impact Development
(LiDAR)	Light Detection and Ranging
(MLSE)	Mean Least Square Error
(NRCS)	National Resource Conservation Service
(RCAP)	Reinforced Concrete Arch Pipe
(RCP)	Reinforced Concrete Pipe
(ROW)	Right-of-Way
(R-Square)	R^2
(SCS)	Soil Conservation Service
(SSURGO)	Soil Survey Geographic Database
(SWM)	Storm Water Management
(UMRF)	University of Memphis Research Foundation
(US-EPA)	U.S. Environmental Protection Agency

LIST OF APPENDICIES

VOLUME II:

1.0 PROJECT INTRODUCTION

1.1 Project Background and Purpose

Over the last 200 years the City of Memphis has grown in both size and population. This growth has resulted in an increased rate of development and a higher percentage of pavement and rooftop impervious area within the City. As the impervious area increases so does the stormwater runoff during rainfall events. That said, much of the City stormwater infrastructure has not been improved to match the increased rate of stormwater runoff. As a result, numerous portions of the City stormwater infrastructure have inadequate conveyance capacity and repetitive flooding problems occur. This is particularly true for older areas of the City.

In fiscal year 2014, the Division of Engineering began utilizing accumulated stormwater utility fees to fund a series of master plan drainage studies throughout 7 identified major drainage areas corresponding to City council districts. These 7 major drainage areas have been subdivided into 37 smaller study areas. The original plan was to systematically analyze these 37 smaller study areas at a rate of one small study area per year in each of the 7 major drainage areas. The overall goal is to analyze each study area in the City based on priority and undertake specific projects to mitigate the impacts of future storm events on the public infrastructure and private property throughout the City.

The City of Memphis hired the SSR team to perform a drainage analysis on the Harrington Creek drainage basin by developing a hydrologic and hydraulic (H&H) model of the stormwater infrastructure located throughout the basin. The model was constructed based on the combination of Geographic Information System (GIS) data and field survey data and was utilized to identify areas with flooding problems and recommend infrastructure improvements that reduce or eliminate the amount of flooding in the selected improvement areas. The estimated cost of each improvement was also quantified along with the impacts of each improvement solution. This data will be utilized by the Division of Engineering to prioritize Capital Improvement Projects (CIPs) throughout the study area and throughout the City. The field surveyed stormwater infrastructure data collected throughout the duration of the study will also be utilized to update the City’s stormwater asset inventory database.

1.2 Study Area Description

The Harrington Creek drainage basin is located north of the intersection of I-40 and I-240 along the northern City of Memphis limits and is split between the City of Memphis and the City of Bartlett. The drainage basin spans west to east from Scheibler Road and Austin Peay Highway to Summer Avenue and Kirby Witten Road and north to south from the Quail Ridge Golf Course and the intersection of Austin Peay Highway and New Covington Pike to John F. Kennedy Park and the intersection of Summer Avenue and Raleigh Lagrange Road. The total drainage area of the Harrington Creek drainage basin is approximately 7,595 acres. The City of Memphis limits divides the basin into western and eastern halves. The western City of Memphis portion of the basin is approximately 3,167 acres, and the eastern City of Bartlett portion of the basin is approximately 4,428 acres. Exhibit 1 in Appendix A shows the location of the drainage basin and the City's preliminary stormwater infrastructure and watershed GIS data.

The project data collection and modeling efforts focused on the western City of Memphis portion of the basin, but limited data was utilized throughout the project for the eastern City of Bartlett area because Harrington Creek meanders between both cities. The study area consists of a mixture of residential, commercial, industrial, parks/recreational, open natural, roadway, and water body land use classifications. The stormwater infrastructure components throughout the study area consist of underground pipe networks, culvert roadway crossings, open channel bridge crossings, man-made open channels, natural open channels, and stormwater detention ponds. All tributaries in the study area intersect Harrington Creek which ultimately converges with the Wolf River in John F. Kennedy Park at the southwest corner of the basin.

1.3 Public Involvement

The project included a public outreach component that focused on informing the public of the impending drainage project, providing a platform for citizens to inform the City of existing drainage-related issues, and aggregation of such data for use in modeling and planning purposes. A website was created for the project where people could report flooding in the basin and learn more about the basin and the goals of the project. Two virtual public meeting were held on June 3rd and 8th of 2021. These meetings were promoted via both physical and digital methods. Information was posted on the City's website in addition to the basin website. Approximately 4200 information cards were mailed to the residents of the areas prone to flooding, and flyers were hung and handed out to the community. Posts were also made on various social media platforms informing the community of the study and public meetings and encouraging participation and feedback. Approximately, 92,000 individuals were reached via the social media platforms. Those who attended the virtual public meetings were asked to provide their name and email address and report any drainage related issues within the basin. The various public involvement materials utilized throughout the project are located in Appendix B of the Volume I report document.

In addition to the flooding reports collected through the the public meeting the City of Memphis Public Works Drain Maintenance Department collected flooding and drainage-related maintenance reports and compiled a running list of the reports in the form a GIS point file. The collected reports within the Harrington Creek baisn were utilized in the study to check the model flooding results and aid in the identification of areas with repetitive flooding problems for proposed infrastructure improvements.

1.4 Team Members

- Smith Seckman Reid (SSR), Inc.
  2650 Thousand Oaks Boulevard, Suite 4200
  Memphis, TN 38118
  Office: (901) 683-3900
  Fax: (901) 683-3990

  Contacts:
  Ben Ledsinger, PE
  Joshua Bruce, EI

- Geodesy Professional Services, LLC
  1312 Dovecrest Road
  Memphis, TN 38134
  Office: (901) 388-2560
  Fax: (901) 388-2566

  Contacts:
  Josh Forsman, PLS

- Younger and Associates, LLC
  97 Directors Row
  Jackson, TN 38305
  Office: (731) 668-7367
  Fax: (731) 668-0042

  Contacts:
  Lana Suite
  Kayla Taylor

2.0 DATA COLLECTION

In an effort to better understand the unique characteristics and rainfall-runoff response of the Harrington Creek drainage basin, a variety of data was collected and analyzed for use in the model development process. This data included a combination of the City's storm water infrastructure GIS data, digital Light Detection and Ranging (LiDAR)-derived topographic data, land cover and soil GIS data, field survey data, and field collected precipitation and flow depth data. The sections below describe each of the data sets collected for incorporation into the drainage study Hydrologic and Hydraulic model.

2.1 City Stormwater Infrastructure GIS Data

The City of Memphis partnered with the University of Memphis Center for Applied Earth Science and Engineering Research (CEASER) to develop a geodatabase of the City's stormwater infrastructure. This data included the preliminary watershed boundary and all current geospatial and descriptive information for the stormwater and drainage elements in the basin. This data was digitized utilizing record drawings and aerial photography and had not been field verified for completeness or accuracy. It was utilized as a guide for the planning and implementation of both the field survey effort and H&H model development process. The dataset included drainage elements that were outside of the scope of the field survey and therefore aided in the comprehensive understanding of the drainage systems throughout the study area and the extent of the contributing urban drainage basin. Exhibit 1 in Appendix A shows the City's preliminary stormwater infrastructure and watershed GIS data.

2.1.1 Flooding Reports

The City of Memphis Public Works Drain Maintenance Department collected flooding and drainage-related maintenance reports and compiled a running list of the reports in the form a GIS database. These georeferenced reports were analyzed and utilized during the study to check the model flooding results and aid in the identification of areas with repetitive flooding problems for proposed infrastructure improvements. Section 6 of this report includes descriptions of the flooding and drainage-related maintenance reports in the vicinity of each identified improvement area. All of the reports utilized during the study are included in the model for reference.

2.2 Topography

GIS-based digital LiDAR-derived topographic data was used as the main source of elevation data for the study. A GIS Raster Digital Elevation Model (DEM) was created from 1-foot contour elevation data and was used to analyze the areas located beyond the top of bank of the open-channel conveyances within the study area. The DEM was updated throughout the data collection and model development efforts by incorporating the field survey data into the DEM. The DEM played a key role in the hydrologic analysis of the study as it aided in the delineation of subcatchments throughout the study area and was used to quantify model subcatchment parameters. It also served as the base layer for the flood-mapping efforts while analyzing the inundation extent of the simulated model results. Exhibit 2 in Appendix A displays the 5-foot contour elevation data for the overall basin. The DEM was constructed from 1-foot contours, but 5-contours were utilized for the exhibit so that the lines could actually be seen at the overall basin map scale.

2.3 Land Cover Data

GIS-based National Land Cover Database (NLCD) data was used at the main data source for surface cover classification of the drainage basin. This data was obtained from the United States Geological Survey (USGS) website and was reclassified to surface cover conditions corresponding to those utilized in the Natural Resources Conservation Service (NRCS) Urban Hydrology for Small Watersheds Technical Release 55 (TR-55) document. These ground cover classifications are also presented in Table 2-7, Table 2-8, and Table 2-9 of Volume 2 of the City of Memphis/ Shelby County Stormwater Management Manual. This reclassified land cover data was utilized as the base data set for the hydrologic analysis conducted during the study. Each of the land cover classifications were assigned a variety of base modeling parameters corresponding to the quantity of development and impervious area within each classification type. Table 1 below summarizes the various land cover classifications utilized in the study.

Table 1: Land Cover Classification Summary

Landuse ID	Description
1	Water
2	Urban Open Space, Fair
3	Urban Residential 1/4 acre average lot size
4	Urban Residential 1/8 acre average lot size
5	Urban Commercial, Business
6	Urban Newly Graded Areas
7	Woods, Fair
8	Brush, Fair
9	Meadow
10	Pasture/Grassland/Range, Fair
11	Row Crops - Straight Row (SR) + Crop Residue Cover (CR), Poor
12	Wetlands

Exhibit 3 in Appendix A displays the land cover data for the overall study area. Figure 1 in Appendix B summarizes, in table format, the percentage of each land use classification within each model subcatchment. Figure 2 in Appendix B summarizes, in table format, the NLCD land cover to NRCS TR-55 land cover reclassification process. Section 3.1 of this report includes more information about how the land use GIS data was utilized during the model development process.

2.4 Soils Data

GIS-based soils data was used as the main data source for soil infiltration classification of the basins in the drainage study. National Resource Conservation Service (NRCS) soil classification data was obtained from the Soil Survey Geographic (SSURGO) database for the Shelby County area and trimmed to the study area. The soils data was analyzed and grouped according to the respective soil texture attribute. Each of the soil texture classifications were assigned a variety of base modeling parameters corresponding to their respective infiltration properties. Table 2 below summarizes the various soil texture classifications utilized in the study.

Table 2: Soil Texture Classification Summary

Soil Texture ID
SILT LOAM
WATER

Exhibit 4 in Appendix A displays the soils texture data for the overall study area. Figure 3 in Appendix B summarizes, in table format, the percentage of each soil texture classification within each model subcatchment. Section 3.1 of this report includes more information about how the soil texture GIS data was utilized during the model development process.

2.5 Field Survey

In an effort to field verify the provided GIS stormwater infrastructure data and develop a model that included accurate existing surface and underground infrastructure elements a field survey was conducted in the study area. The field survey was completed by Geodesy Professional Services, LLC in 2020. Control points were established throughout the study area using Global Positioning System (GPS) technology, and traditional total station field survey efforts were conducted. The geospatial location and vertical elevation of the stormwater infrastructure in the study area were collected along with georeferenced photos of the structures. The surveyed infrastructure included open drainage channels, roadway culvert crossings, bridge crossings, storm sewer pipe networks, and catch basin/ manhole/ headwall structures. Exhibit 5 in Appendix A displays the extent collected survey data for the overall study area. The collected survey data and photos are included in the final set of electronic deliverables.

2.5.1 Open-Channel Cross-Sections and Road Crossings

Surveyed cross-sections for open-channels located in the study area were conducted from top of bank to top of bank and included a minimum of five elevation points. Each cross-section included two top of bank elevation points, two toe of slope elevation points, and a single flow line elevation point. These cross-sections were required at each significant change in channel geometry, material, or direction, at all points of concentrated stormwater discharge, and immediately upstream and downstream of structure crossings. Generally, the required spacing between cross-sections was 500 feet or less. At the location of roadway and/or bridge crossings additional elevation data was collected at the roadway/bridge deck surface. The constructed model consists of approximately 87,550 linear feet of open channel conveyances.

2.5.2 Stormwater Structures and Pipes

All storm sewer pipes greater than or equal to 24 inches were surveyed along with all connected drainage structures (catch basins, manholes, headwalls, etc.). Information was collected regarding the size, shape, and material of the surveyed storm sewer pipes. Each surveyed structure included elevation points at the flow line invert elevation, the surface opening rim elevation, and the top of structure elevation. Descriptive information was collected for each surveyed structure identifying what type of drainage structure was encountered (headwall, manhole, area inlet, 6-72 combination inlet, etc.). The constructed model consists of approximately 89,250 linear feet of underground pipe and box culvert conveyances.

2.5.3 Field Infrastructure Photos

All surveyed road crossings and outfalls were photographed and georeferenced. Structures crossing an open channel were photographed from both the upstream and downstream vantage points, and structures discharging into channels were photographed from the channel. Underground storm sewer drainage structures were photographed from the ground surface facing the front of the structure rim. The georeferenced location of the photographs was relatively close and did not have survey-grade accuracy. The collected photos were incorporated into a GIS data set that is included in the final set of electronic deliverables.

3.0 MODEL DEVELOPMENT

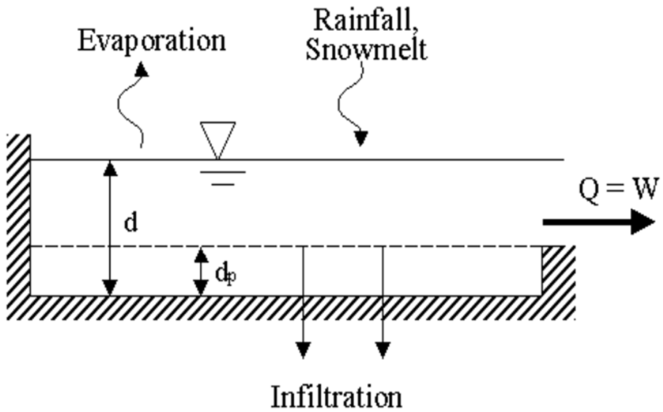
The hydrologic and hydraulic model developed during this study was created in the Innovyze modeling software package InfoSWMM Suite 15.0. The InfoSWMM software utilized an enhanced version of the US-EPA Storm Water Management Model (SWMM) analysis engine as developed and distributed by the Water Supply and Water Resources Division of the U.S. Environmental Protection Agency's National Risk Management Research Laboratory (SWMM Version EPA SWMM 5.1.015). The project was modeled using a combination 1D/1D hydrologic and hydraulic model: which relied on traditional model mechanics to characterize flow throughout the underground drainage network, channels, and overland flow areas. The model relied on the surveyed stormwater infrastructure data and a variety of hydrologic and hydraulic input parameters to perform the rainfall-runoff simulations. The parameter values input into the model were primarily based on engineering judgement, EPA SWMM and InfoSWMM software user manual guidance, and Volume 2 of the City of Memphis/ Shelby County Stormwater Management Manual.

Existing City stormwater infrastructure, topographic, and land cover/ soil classification GIS data were utilized in conjunction with field survey data and measured rainfall/flow depth data gathered during the project to construct a representative stormwater model of the drainage basin. A single existing conditions model was developed for the project that included both the City of Memphis and City of Bartlett portions of the drainage basin. The model was then calibrated to the measured rainfall and flow depth data gathered during the survey phase of the project and the rainfall for various NRCS Type II 24-hour storm events were simulated in the model. The model flooding results were then mapped and compared to City flooding complaints and target areas were identified for infrastructure improvements. The sections below summarize the hydrologic and hydraulic components of the model and their respective parameters.

3.1 Hydrologic Modeling Methodology

The surface runoff calculation methodology used in the model was the EPA SWMM Non-linear Reservoir method. This calculation method was established by coupling the continuity equation with Manning's equation. Each subcatchment is treated as a "reservoir" with a storage capacity equal to the maximum depression storage (d_p), which is the maximum surface storage provided by ponding, surface wetting, and rainfall interception. Inflow enters the "reservoir" from precipitation and the upstream subcatchments, and it exits the "reservoir" through infiltration, evaporation, and surface runoff (Q). Evaporation wasn't accounted for in our model, and soil infiltration was estimated using the Green-Ampt method. Surface runoff (Q) only occurs when the depth of water (d) in the subcatchment "reservoir" exceeds the maximum depression storage (d_p), in which case the outflow is given by Manning's Equation. The depth of water over the subcatchment "reservoir" (d) was continuously updated at each modeled time step through numerically solving a water balance equation over the subcatchment.

Equation (1) and the schematic diagram and below display how the EPA SWMM Non-linear Reservoir Runoff Model functions:



$$Q = W\ 1.49/n\ (d - d_p)^{(5/3)} \cdot S^{(1/2)} \qquad (1)$$

where:

Q is surface runoff flow rate
W is the subbasin width
n is Manning's roughness coefficient
d is the depth of water
d_p is the maximum depression storage depth, and
S is slope

The parameters accounted for in the hydrologic model include but are not limited to the following: sub-basin area, width, impervious area, slope, pervious and impervious overland flow Manning's n value, pervious and impervious depression storage, soil suction head, hydraulic conductivity, and initial moisture deficit. These parameters were utilized within subcatchment, soil, and rain gage model elements to calculate the amount of soil infiltration, surface storage, and surface runoff resulting from a single or series of rainfall events. The parameter values for each of these components were based on engineering judgement, EPA SWMM and InfoSWMM software user manual guidance, and Volume 2 of the City of Memphis/ Shelby County Stormwater Management Manual. The hydrologic parameters were incorporated into the model using the topographic DEM, land cover, and soil GIS data sets assembled during the data collection phase of the project. Figure 4 in Appendix B summarizes, in table format, the base hydrologic model parameters assigned to each land use and soil texture classification. The sections below summarize each of the parameters utilized in the hydrologic modeling efforts.

3.1.1 Subbasin Delineation

The preliminary watershed provided by CEASER was analyzed and fine-tuned to represent more accurately the basin's actual watershed boundary. The topographic DEM was utilized in conjunction with the GIS and field survey stormwater infrastructure data to refine the overall watershed boundary of the basin and subdivide it into urban subcatchments. The InfoSWMM Subcatchment Manager add-on software was used to perform the initial watershed refinement and subcatchment delineation efforts. The software could not be relied upon completely without consideration of the existing subsurface features within the basins, so the software ArcGIS Pro was used to fine-tune the subcatchment boundaries to the area contributing runoff to the entire pipe network within each subsurface storm sewer system.

The preliminary target subcatchment size range was 50-100 acres, but this range was altered based on the unique characteristics of the subcatchments within the drainage basin. Smaller subcatchments were incorporated into the model where necessary to ensure that all upstream pipe networks included in the model had contributing runoff into the analyzed system. Larger subcatchments were incorporated where necessary to reflect undeveloped areas where no underground storm sewer systems were present or where the size of the underground pipe networks were less than 24 inches and therefore fell outside of the scope of the modeling efforts. Larger subcatchments were also utilized within the eastern City of Bartlett portion of the basin due to limited data in the area and the fact that the project data collection and modeling efforts focused on the western City of Memphis portion of the basin.

The initial base and calibrated model included a total of 68 delineated subcatchments that varied in size from 21 acres to 872 acres with an average subcatchment size of 112 acres. After the initial review of the model, we were asked to further subdivide the basin into smaller subcatchments. The revised base and recalibrated final model included a total of 162 delineated subcatchments that varied in size from 3.6 acres to 872 acres with an average subcatchment size of 47 acres.

Exhibit 6 in Appendix A displays the refined overall watershed boundary of the drainage basin. Exhibit 7 in Appendix A displays the delineated sub-basins within the drainage basin. Figure 5 in Appendix B summarizes, in table format, the initial base pre-calibration subcatchment parameters for all initial model subcatchments. Figure 6 in Appendix B summarizes, in table format, the revised base pre-calibration subcatchment parameters for all revised model subcatchments.

3.1.2 Subbasin Width

The subcatchment width parameter is used along with the depth of flow to estimate the theoretical cross-sectional area applied to the Manning's equation while calculating the surface runoff from each subcatchment. It has a storage effect and shape effect on the routing hydrograph and is often used as a calibration parameter. The InfoSWMM Subcatchment Manager add-on software was used to calculate the base width values for the subcatchments in the model. A series of surface analysis raster files were created from the topographic DEM and used for the calculation procedures. These analysis raster files included the following: flow direction raster, flow accumulation raster, flow length raster, and slope raster. The flow direction raster was used to create the flow length raster, and the initial base width values for all subcatchments were calculated by dividing the subcatchment area by the longest flow length calculated from the flow length raster file. Figure 5 in Appendix B summarizes, in table format, the initial base pre-calibration subcatchment parameters for all initial model subcatchments. Figure 6 in Appendix B summarizes, in table format, the revised base pre-calibration subcatchment parameters for all revised model subcatchments.

3.1.3 Subbasin Slope

The subcatchment slope parameter is used in the Manning's equation to estimate the rate of flow leaving each subcatchment. The InfoSWMM Subcatchment Manager add-on software was used to calculate the base slope values for the subcatchments in the model. A series of surface analysis raster files were created from the topographic DEM and used for the calculation procedures. These analysis raster files included the following: flow direction raster, flow accumulation raster, flow length raster, and slope raster. The flow direction raster, flow accumulation raster, and slope raster were used to calculate the initial base slope values for all subcatchments by averaging the slope along the longest flow length. Figure 5 in Appendix B summarizes, in table format, the initial base pre-calibration subcatchment parameters for all initial model subcatchments. Figure 6 in Appendix B summarizes, in table format, the revised base pre-calibration subcatchment parameters for all revised model subcatchments.

3.1.4 Impervious Area and Sub-Area Routing

Each subcatchment modeled utilizing the EPA SWMM methodology is composed of two sub-areas, an impervious sub-area, and a pervious sub-area. These sub-areas differ in that almost all of the rainfall that falls on the impervious sub-area is converted into runoff while the rainfall that falls on the pervious sub-area has an opportunity to infiltrate prior to producing surface runoff. The EPA SWMM methodology also allows sub-area routing to occur within each subcatchment prior to runoff exiting through the outlet of the subcatchment. This capability allows the resulting runoff from either the impervious sub-area or pervious sub-area to be routed through the opposite sub-area prior to exiting the subcatchment. This concept enables Low Impact Development (LID) entities and connected/disconnected impervious areas to be modeled if desired or needed during the model calibration efforts. Initially, the subcatchments in this H&H model did not utilize the sub-area routing option and the resulting surface runoff for both the impervious and pervious sub-areas were routed directly to the outlet of each respective subcatchment. During the final calibration of the revised calibrated model this was altered so that all the subcatchments routed 30% of the resulting surface runoff from the impervious sub-areas across the pervious sub-area and then to the subcatchment outlet. This enabled a portion of the runoff from the impervious sub-area to be infiltrated and stored in the depression storage of the pervious sub-area prior to existing the subcatchment through the outlet.

Impervious area percentage values were assigned to each land cover classification based on the estimated quantity of anticipated development for each classification. The assigned percentages were based on aerial imagery, engineering judgement, NRCS TR-55 documentation, NLCD documentation, EPA SWMM and InfoSWMM software user manual guidance, and Volume 2 of the City of Memphis/ Shelby County Stormwater Management Manual. The reclassified NLCD GIS land cover data was then overlaid on the GIS subcatchment data and area-weighted impervious area percentages were calculated for each respective subcatchment. These area-weighted calculations were performed using the InfoSWMM Subcatchment Manager add-on software.

Exhibit 8 in Appendix A displays the impervious area percentages applied to the subcatchments throughout the drainage basin based on the land cover GIS data. Figure 5 in Appendix B summarizes, in table format, the initial base pre-calibration subcatchment parameters for all initial model subcatchments. Figure 6 in Appendix B summarizes, in table format, the revised base pre-calibration subcatchment parameters for all revised model subcatchments.

3.1.5 Depression Storage

Depression storage is the maximum surface storage provided by ponding, surface wetting, and rainfall interception in each subcatchment. The EPA SWMM methodology treats depression storage as a “reservoir” that must be filled and exceeded prior to surface runoff occurring. Separate depression storage values are utilized for the impervious and pervious sub-areas in each subcatchment. Impervious and pervious depression storage values were assigned to each land cover classification based on engineering judgement, EPA SWMM and InfoSWMM software user manual guidance, and Volume 2 of the City of Memphis/ Shelby County Stormwater Management Manual. The reclassified NLCD GIS land cover data was then overlaid on the GIS subcatchment data and area-weighted impervious and pervious depression storage values were calculated for each respective subcatchment. These area-weighted calculations were performed using the InfoSWMM Subcatchment Manager add-on software. Figure 5 in Appendix B summarizes, in table format, the initial base pre-calibration subcatchment parameters for all initial model subcatchments. Figure 6 in Appendix B summarizes, in table format, the revised base pre-calibration subcatchment parameters for all revised model subcatchments.

3.1.6 Sheet Flow Manning’s n

Sheet flow (overland flow) is shallow runoff, typically 1.2 inches or less in depth, flowing uniformly over a theoretical plane surface prior to reaching a concentrated flow state. Manning’s n coefficient for sheet flow represents the surface roughness and friction forces acting on the plane that the runoff is flowing across. It describes surface obstacles such as debris, vegetation, sediment, pavement, and rocks. The roughness coefficient is used in the Manning’s equation while calculating the resulting surface runoff from each subcatchment. Separate sheet flow Manning’s n values are utilized for the impervious and pervious sub-areas in each subcatchment.

Impervious and pervious sheet flow Manning’s n values were assigned to each land cover classification based on engineering judgement, EPA SWMM and InfoSWMM software user manual guidance, and Volume 2 of the City of Memphis/ Shelby County Stormwater Management Manual. The reclassified NLCD GIS land cover data was then overlaid on the GIS subcatchment data and area-weighted impervious and pervious sheet flow Manning’s n values were calculated for each respective subcatchment. These area-weighted calculations were performed using the InfoSWMM Subcatchment Manager add-on software. Figure 5 in Appendix B summarizes, in table format, the initial base pre-calibration subcatchment parameters for all initial model subcatchments. Figure 6 in Appendix B summarizes, in table format, the revised base pre-calibration subcatchment parameters for all revised model subcatchments.

3.1.7 Soil Infiltration

Soil infiltration is the process of rainfall penetrating the ground surface into the unsaturated soil zone of the pervious sub-area in each subcatchment. The rate of soil infiltration increases or decreases the depth of water over each subcatchment "reservoir" at each calculation time step and therefore directly impacts the quantity of surface runoff resulting from a given rainfall event. The five available calculation methods for modeling soil infiltration were the Horton Method, the Modified Horton Method, the Green-Ampt Method, the Modified Green-Ampt Method, and the NRCS (SCS) Curve Number Method.

The Green-Ampt Method was used to model soil infiltration in our H&H model. This method for modeling infiltration assumes that a sharp wetting front exists in the soil column, separating soil with some initial moisture content below from saturated soil above. The input parameters required are the initial moisture deficit of the soil, the soil's hydraulic conductivity, and the suction head at the wetting front. The recovery rate of moisture deficit during dry periods is empirically related to the hydraulic conductivity.

Initial moisture deficit, hydraulic conductivity, and suction head values were assigned to each soil texture classification based on engineering judgement, EPA SWMM and InfoSWMM software user manual guidance, and Volume 2 of the City of Memphis/ Shelby County Stormwater Management Manual. The GIS soil data was then overlaid on the GIS subcatchment data and the area percentage of each soil texture classification within each respective subcatchment was calculated. These calculations were performed using the InfoSWMM Subcatchment Manager add-on software. When performing the soil infiltration calculations, the model utilized the Green-Ampt parameter values assigned to each soil texture classification in conjunction with the calculated area percentage values of each classification within each subcatchment. Figure 4 in Appendix B summarizes, in table format, the base Green-Ampt soil infiltration parameters assigned to each soil texture classification.

3.2 Hydraulic Modeling Methodology

The hydraulic modeling efforts in the basin span west to east from Scheibler Road and Austin Peay Highway to Summer Avenue and Kirby Witten Road and north to south from the Quail Ridge Golf Course and the intersection of Austin Peay Highway and New Covington Pike to John F. Kennedy Park and the intersection of Summer Avenue and Raleigh Lagrange Road. The project data collection and modeling efforts focused on the western City of Memphis portion of the basin, but limited data was incorporated into the model for the eastern City of Bartlett area because Harrington Creek meanders between both cities. Hydraulic modeling efforts were performed for the stormwater infrastructure throughout the basin with a focus on the elements located with the City of Memphis limits. These stormwater infrastructure components consisted of underground pipe networks, culvert roadway crossings, open channel bridge crossings, man-made open channels, natural open channels, and stormwater detention ponds. All tributaries in the study area intersect Harrington Creek which ultimately converges with the Wolf River in John F. Kennedy Park at the southwest corner of the basin. Exhibit 9 in Appendix A displays the hydraulic model elements within the drainage basin.

The primary hydraulic components modeled in the Harrington Creek basin were one main open channel tributary (Harrington Creek) that reached from the southwest City of Memphis portion of the basin through the middle and eastern City of Bartlett portion of the basin and back into the northwest City of Memphis portion of the basin. There were three secondary open channel tributaries that intersected the main tributary in the southwest City of Memphis portion of the basin. There were two secondary open channel tributaries that intersected the main tributary in the lower middle and middle City of Memphis portion of the basin and the centermost secondary tributary broke into two tertiary tributaries. The main tributary broke into two secondary tributaries in Nesbit Park with one reaching back into the northwest City of Memphis portion of the basin and one continuing north through the City of Bartlett portion of the basin. There were a number of additional secondary tributaries that intersected the main tributary within the eastern middle City of Bartlett portion of the basin, but due to limited data and the modeling scope of the project they were not included in detail in the modeling efforts. Each of the modeled tributaries included a number of pipe network systems, culvert roadway crossings, and minor tertiary tributaries that intersected them throughout their reaches. There were over 45 pipes networks/ culvert roadway crossings that intersected Harrington Creek and its tributaries throughout the modeled western City of Memphis portion of the basin. Exhibit 10 in Appendix A displays the open channel and closed conduits throughout the modeled drainage basins.

The hydraulic parameters accounted for in the model include but are not limited to the following: conveyance geometry, pipe/channel/overbank Manning's n value, length, slope, entry/exit losses, open/closed flow type, available surface ponding area, detention pond storage and outlet structures, and Federal Highway Administration (FHWA) inlet/outlet control culvert codes and roadway overflow weir coefficients. These parameters are utilized in the flow routing portion of the simulation as the calculated surface runoff travels through the conveyance infrastructure. The surveyed stormwater infrastructure data was converted to a GIS format and merged with the appropriate hydraulic parameters to create conveyance elements in the model. These model conveyance elements consist of junctions, conduits, storage ponds, weirs, orifices, and outfalls that mimic real world stormwater infrastructure such as headwalls, manholes, catch basin inlets, pipe networks, pipe or box culvert crossing, roadway overflow weirs, man-made ditches and armored channels, natural creeks and streams, and detention ponds.

3.2.1 Open Channels

Open channel conveyances were modeled using the combination of conduit and junction model elements. Open channel junctions were located where one of the following conditions were present: change in invert elevation, change in direction, change in channel or overbank geometry, change in channel material, intersection with concentrated pipe network discharge, confluence of tributaries, and/or intersection with roadway or bridge crossings. These open channel junction elements relied on the following data inputs: invert elevation, rim elevation, ponded surface area, and external inflow data. The collected survey data and field photos were used along with engineering judgement to populate these open channel junction parameters. The rim elevations for the open channel junctions equal the top elevation of the defined cross-section transect data, and the ponded surface area values represent additional storage above the defined rim elevation.

Open channel conduits are the elements that connect the junction elements to one another and ultimately define model conveyance properties. These elements use the Manning equation to express the relationship between flow rate (Q), cross-sectional area (A), hydraulic radius (R), slope (S), and Manning's roughness coefficient (n). Equation (2) displays the Manning's equation in standard U.S. units.

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \tag{2}$$

where:

Q is open channel flow rate
n is Manning's roughness coefficient
A is the cross-sectional area
R is the hydraulic radius, and
S is slope

These open channel conduit elements relied on the following data inputs: length, Manning's n coefficient, upstream invert elevation, downstream invert elevation, entry loss coefficient, exit loss coefficient, and geometric shape definition. The collected survey data and field photos were used along with engineering judgement to populate these open-channel conduit parameters. Entry and exit losses were added to the open channel conduits on a case-by-case basis but were typically only added where a change in channel cross-section geometry or material occurred, at bridge crossings, or at the confluence of tributaries.

The field survey channel cross-section data was combined with the DEM elevation data to create representative transects for the open-channel conduits in the model. These transects refer to the geometric data that describe how the bottom elevation varies with horizontal distance over the cross-section of an irregular-shaped open channel. These transects utilized the survey data for the main channel geometry and elevation values and the DEM data for the overbank floodplain geometry and elevation values. Where applicable, different Manning's roughness coefficient values were utilized for the left overbank, right overbank, and main channel sections to reflect varying surface cover conditions throughout each cross-section. Exhibit 11 in Appendix A displays two typical open channel transects (cross-sections) utilized in the model and the location of their corresponding conduits. The model consists of approximately 87,550 linear feet of open channel conveyances split between approximately 215 open channel conduit elements.

This provided a more realistic estimate of channel conveyance under high flow conditions. Note that once the capacity of a conduit in the model is exceeded, the upstream junction surcharges and the excess water is allowed to pond atop the junction in the defined ponded surface area and subsequently drains back into the junction as capacity becomes available. The Manning's roughness coefficients and entry/exit losses (where applicable) assigned to the open channel elements of the model were based on the following: engineering judgement, EPA SWMM and InfoSWMM software user manual guidance, Volume 2 of the City of Memphis/ Shelby County Stormwater Management Manual, Minor Loss Coefficients for Storm Drain Modeling with SWMM (William H. Frost), and Chapter 3 of the HEC-RAS River Analysis System Hydraulic Reference Manual (Version 5).

3.2.2 Underground Pipe Networks

Closed system pipe networks and encapsulated channel conveyances were modeled using the combination of conduit and junction model elements. Closed system junctions were located at all connected drainage structures (catch basins, manholes, headwalls, etc.) and the intersection of pipe networks (blind connections), open channel conveyances, and roadway crossings. These closed system junction elements relied on the following data inputs: invert elevation, rim elevation, ponded surface area, and external inflow data. The collected survey data and field photos were used along with engineering judgement to populate these closed system junction parameters. The rim elevations for the closed system junctions equal the surface elevation of the drainage structures, and the ponded surface area values represent additional storage above the defined rim elevation. Just like open channel junctions, once the capacity of a closed conduit is exceeded, the upstream junction surcharges and the excess water is allowed to pond atop the junction in the defined ponded surface area and subsequently drains back into the junction as capacity becomes available.

Closed conduits are the elements that connect the junction elements to one another and ultimately define model conveyance properties. These closed conduit elements were modeled as gravity flow conveyances and therefore utilized the Manning's equation defined in Section 3.2.1 for their respective flow rate calculations. These closed conduit elements relied on the following data inputs: length, Manning's n coefficient, upstream invert elevation, downstream invert elevation, entry loss coefficient, exit loss coefficient, and geometric shape definition. The collected survey data and field photos were used along with engineering judgement to populate these closed system conduit parameters. Entry and exit losses were added to the closed system conduits on a case-by-case basis but were typically only added where a pipe network intersected an open channel conveyance or at roadway culvert crossings.

Closed conduits are typically pipes that utilize standardized shapes (circular, horizontal elliptical, arch, rectangular, etc.), sizes, and materials (reinforced concrete, corrugated metal, high-density polyethylene, ductile iron, vitrified clay, etc.). Exhibit 12 in Appendix A displays a typical closed pipe network in the model. The model consists of approximately 89,250 linear feet of underground pipe and box culvert conveyances split between approximately 580 closed conduit elements. Unlike open channel conduits, closed conduits use a single representative Manning's roughness coefficient value for their respective flow rate calculations. This roughness coefficient corresponds to the material that lines the inside diameter of the pipe. The Manning's roughness coefficients and entry/exit losses (where applicable) assigned to the closed conduit elements of the model were based on the following: engineering judgement, EPA SWMM and InfoSWMM software user manual guidance, Volume 2 of the City of Memphis/ Shelby County Stormwater Management Manual, and Minor Loss Coefficients for Storm Drain Modeling with SWMM (William H. Frost).

3.2.3 Roadway Culvert Crossings and Bridge Crossings

Roadway culvert crossings and bridge crossings were modeled utilizing closed conduit elements in parallel with overflow weir elements. These closed conduits used the same model parameters as defined in Section 3.2.2 with the exception that they all utilized entry and exit loss coefficients, and additional Federal Highway Administration (FHWA) flow control culvert codes. The culvert codes utilized in the model are based on Charts 1-59 of the FHWA Hydraulic Design Series Number 5 (HDS-5) publication and are used to model inlet control conditions for a variety of inlet configurations (projecting pipe, headwall, mitered to slope, grooved or square edge, etc.). Roadway culvert crossings were modeled utilizing standard pipe configurations (circular, elliptical, arch, etc.), or encapsulated channel configurations (rectangular, rectangular with triangular bottom, etc.). Open channel bridge crossings were modeled utilizing custom geometric shapes defined by shape curves input into the model similar to cross-section transects. These custom shape curves were used to define the cross-section of an open channel conveyance beneath a bridge with a capped top elevation representing the bottom of the bridge deck.

Weir elements were utilized to model potential roadway or bridge deck overtopping at the locations of roadway and bridge crossings. These weirs were defined as “Roadways” and additional inputs were provided to model the overtopping of the roadway/ bridge deck in accordance with the Chart 60 of the FHWA (HDS-5) publication. These additional inputs were the roadway surface material (paved, gravel), the roadway width, and the overtopping discharge coefficient. Modeling these overflow weirs were an important component of modeling the roadway and bridge crossings because they enabled the model to quantify the height of surface flooding above the roadway/ bridge deck. The parameter assigned to these model elements were based on engineering judgement, EPA SWMM and InfoSWMM software user manual guidance, Volume 2 of the City of Memphis/ Shelby County Stormwater Management Manual, and the FHWA (HDS-5) publication.

3.2.4 Boundary and Tailwater Conditions

The City’s drainage basins discharge into downstream stormwater systems, local rivers and streams, or directly into the Mississippi River. The Wolf River is the outfall of Harrington Creek and all of its tributaries. Boundary conditions in the model are the water surface elevations of the bodies of water downstream of the study area. This water surface elevation is effectively the tailwater elevation applied to the outfalls of the basin tributaries in the model during simulations. The most commonly used boundary conditions are the following types: free, normal, fixed, and time series. Under free outfall conditions, the tailwater elevation is determined by the minimum of the critical flow depth and normal flow depth. Under normal outfall conditions, the tailwater elevation is based on the normal flow depth of the receiving body of water. Under fixed outfall conditions, the tailwater elevation is determined by a fixed elevation input by the modeler. Under time series outfall conditions, the tailwater elevation is determined by a time series of elevations input by the modeler.

The original existing conditions model utilized a synthetic tailwater elevation time series created from the combination of historic U.S. Army Corps of Engineers (USACE) stage gage data and Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) elevation data. The USACE historic stage gage data was for the Wolf River at Raleigh, TN (Gage: WF111) for the dates 4/12/2019 through 4/16/2019, and the FEMA stage elevation data utilized was from the 2013 FIS document. After approval of the existing conditions model, the City directed us to utilize free outfall conditions for the proposed improvements modeling efforts and when comparing the impacts of the improvements to the existing conditions. All tailwater data is included in the model for reference.

3.3 Inundation Mapping

Inundation mapping consisted of comparing the resulting maximum hydraulic grade line (HGL) elevations simulated in the model to the surface elevations of the DEM and creating graphic representations of the results. This process was performed after the model was calibrated to the measured rainfall and flow depth data and standard design storms were simulated in the model. GIS inundation raster files that displayed the depth of surface flooding across the DEM were created for various selected design storm events. These inundation mapping procedures were performed in the InfoSWMM Risk Assessment Manager add-on software. These inundation raster files were used as the primary method of quantifying and visualizing the model results. The inundation raster files were converted into GIS polygon shapefiles that were then utilized to cross-reference GIS building footprint data throughout the basin in order to quantify the number of structures removed from the flooding footprint of the 10-year and 100-year storm events.

4.0 MODEL CALIBRATION

In order to develop a representative H&H model that accurately simulates the rainfall-runoff response of the drainage basin, the model had to be calibrated to the field measured rainfall and flow depth data obtained during the data collection process. Model calibration consists of fine tuning of the model parameters until the model simulates field measured conditions to an established degree of accuracy. Fine-tuning of the model entails making adjustments to the model parameters to obtain the desired output data. The degree of accuracy refers to the difference between simulated and actual values and is used to establish a level of confidence in the model. Calibration is important to establish model credibility and to increase knowledge and understanding of the system and how it responds to rainfall events. Successful model calibration is dependent upon the field measured rain gage and stage gage data and the storm events that occurred throughout the duration that the gages were installed and monitored.

4.1 Collected Rainfall and Stream Gage Data

The City of Memphis partnered with the University of Memphis Center for Applied Earth Science and Engineering Research (CEASER) to install and collect data from three rain gages and two stage gages (pressure transducers) placed throughout the basins in study area. These gages were monitored for a time span of approximately three and a half months and the data was furnished in two-week intervals. The data for all gages were taken in 5-minute intervals and all anomalies were purged prior to use in the calibration efforts. Table 3 below and Table 4 below summarize the locations of the gages and the duration they were installed. Exhibit 13 in Appendix A displays the location of the gages within the drainage basin.

Table 3: Rain Gage Summary

Model Rain Gage ID	Location	Duration of Gage Data Collection
BFD2	Bartlett Fire Dept. Station No. 2	11/25/2019 – 3/13/2020
BMP	Bartlett Municipal Park	
MFD48	Memphis Fire Dept. Station No. 48	

Table 4: Stage Gage Summary

Model Stage Gage Conduit ID	Location	Duration of Gage Data Collection
0105_FG01-1014	The pressure transducer was installed in the rectangular concrete open channel between parcels B0156G A00011 and B0156G A00010 southeast of the intersection of Stage Rd. and Alfaree St.	11/25/2019 – 3/13/2020
0105_FG02-1030	The pressure transducer was installed in the rectangular concrete open channel between parcels B0157S B00004 and B0157S B00005 south of the intersection of Memphis Arlington Rd. and Bartlett Blvd.	

Prior to utilizing the collected data in the model calibration efforts, the field measured rainfall and conduit stage data had to be analyzed and a series of calibration-worthy storm events selected. The analysis included reviewing the collected precipitation depth and flow depth data for various timeframes and quantifying the following parameters: cumulative precipitation depth, maximum incremental precipitation depth, date and time of precipitation peak, maximum incremental depth of flow, and date and time of flow depth peak. In order to qualify as a calibration-worthy storm event the precipitation and corresponding flow depth data needed measurable quantities and alignment from a time to peak perspective. Ideally, we were looking for storm events that had a cumulative precipitation depth greater than the 1-year recurrence interval NRCS Type II 24-hour storm event precipitation depth (3.35 inches) and maximum incremental flow depth values that peaked at or after the maximum incremental precipitation peak time.

The measured rainfall and stage gage conduit depth data collected during the following eighteen timeframes were utilized for the initial model calibration efforts:

- November 26 – 30, 2019
- November 30 – December 6, 2019
- December 6 - 9, 2019
- December 15 - 20, 2019
- December 28, 2019 – January 2, 2020
- January 2 - 7, 2020
- January 11 - 14, 2020
- January 14 – 18, 2020
- January 18 - 21, 2020
- January 23 - 26, 2020
- February 4 - 9, 2020
- February 9 - 12, 2020
- February 12 - 17, 2020
- February 18 - 22, 2020
- February 23 – March 1, 2020
- March 1 – 6, 2020
- March 9 – 11, 2020
- March 11 – 13, 2020

Out of the eighteen timeframes above, the collected data in the following six timeframes were utilized for the final model calibration and verification efforts:

- November 26 – 30, 2019 (Verification)
- December 15 - 20, 2019 (Verification)
- January 11 - 14, 2020 (Calibration)
- February 4 - 9, 2020 (Verification)
- February 12 - 17, 2020 (Verification)
- March 11 – 13, 2020 (Verification)

The field measured rainfall data for the selected storm events was utilized as hydrologic time series data and assigned to the model subcatchments for the calibration process. The collected rainfall data was assigned to groups of subcatchments based on their spatial location in relation to the locations of the rain gages. The BFD2 rain gage data was assigned to 50 subcatchments located in the upper third of the drainage basin. The BMP rain gauge data was assigned to 54 subcatchments located in the middle third of the drainage basin. The MFD48 rain gauge data was assigned to the remaining 58 subcatchments located in the lower third of the basin. This separation allowed the calibration parameters for the three groups of subcatchments to be modified independently from one another. Exhibit 14 through Exhibit 16 in Appendix A display the location of the subcatchment groups (in yellow) calibrated to each respective set of collected field data. The rain gage subcatchment groups are presented in the following order: BFD2 rain gage (Exhibit 14), BMP rain gage (Exhibit 15), and MFD48 (Exhibit 16).

4.2 Model Calibration and Verification Process

In an effort to accurately simulate the rainfall-runoff response of the specific basins in the study, the initial parameters input into the model had to be altered through a series of calibration iterations. These calibration iterations were performed utilizing the InfoSWMM SA Calibrator add-on software. Key calibration parameters were identified and grouped for adjustment during the calibration process and minimum and maximum adjustment ranges were established for each individual parameter based on published typical data ranges and the model's sensitivity to each parameter. Six subcatchment parameters were selected for modification during the initial calibration efforts. Three soil infiltration parameters and eight subcatchment parameters were selected for modification during the final calibration efforts. Table 5 and Table 6 summarize the selected model parameters that were modified during the initial and final calibration efforts.

Table 5: Initial Calibration Model Parameters

Subcatchment Parameters	Imperviousness (%)
	Width (ft)
	Slope (%)
	Manning's n for Pervious Portion
	Depression Storage for Impervious Portion (in)
	Depression Storage for Pervious Portion (in)

Table 6: Final Calibration Model Parameters

Green-Ampt Soil Infiltration Parameters	Suction Head (in)
	Hydraulic Conductivity (in/hr)
	Initial Moisture Deficit
Subcatchment Parameters	Imperviousness (%)
	Width (ft)
	Slope (%)
	Manning's n for Pervious Portion
	Depression Storage for Impervious Portion (in)
	Depression Storage for Pervious Portion (in)
	Runoff Routing Destination
	% of Runoff Routed to Destination

The soil infiltration parameters were calibrated globally for the entire model and the subcatchment parameters were calibrated within their respective rain gage groups discussed in Section 4.1.

The field measured rainfall obtained during the data collection phase of the project was used as the simulation input data for the calibration efforts and the field measured conduit flow depth data was used as the target output data. The Calibrator tool utilized genetic algorithms to optimally adjust all the specified parameters simultaneously to values that resulted in a simulation conduit flow depth curve that most closely matched the specified field measured flow depth curve. During the calibration procedures iterations were performed within the Calibrator tool utilizing three different mathematical convergence methods and a variety of evaluation trials. The three convergence methods utilized during the initial calibration process were the Difference in Total Volume (DTV) Method, the Mean Least Square Error (MLSE) Method, and the R^2 (R-Square) Method, and each of the convergence calibration iterations were ran for 10-200 evaluation trials. Table 7 below summarizes the equation and goal of each mathematical convergence method utilized in the initial calibration process.

Table 7: Calibration Mathematical Convergence Methods

Difference in Total Volume (DTV)	$Minimize \left(\sum_{i=1}^{N} Pobs_i - \sum_{i=1}^{N} Psim_i \right)$
	Can range from -∞ (poor performance) to ∞ (poor performance) with the ideal value being zero.
Mean Least Square Error (MLSE)	$Minimize \quad \frac{\sum_{i=1}^{N} (Pobs_i - Psim_i)^2}{N}$
	Can range from zero (best fit) to ∞ (poor fit).
R^2 (R-Square)	$Maximize \left(\frac{\sum_{i=1}^{N} (Pobs_i - \overline{P}obs)(Psim_i - \overline{P}sim)}{\left[\sum_{i=1}^{N} (Pobs_i - \overline{P}obs)^2 \right]^{0.5} \left[\sum_{i=1}^{N} (Psim_i - \overline{P}sim)^2 \right]^{0.5}} \right)^2$
	Can range from zero (poor fit) to one (best fit).

In the equations above, N designates the total number of measurements available, $Pobs_i$ represents the observed measurement values at time i; $Psim_i$ is the model simulated values at time i; $\underline{P}obs$ is mean of the measured values; $\underline{P}sim$ is mean of the simulated values. The Difference in Total Volume convergence method provided the best results and was therefore utilized the most.

During the initial model calibration efforts six subcatchment parameters were calibrated for each of the eighteen timeframes noted in Section 4.1. Those calibrated parameters were then averaged to create a single dataset of initial calibrated subcatchment parameters. After the initial calibration efforts were completed, three soil infiltration parameters and eight subcatchment parameters were further fine-tuned via the calibration tool and manual adjustment to finalize the model calibration process. Figure 7 in Appendix B summarizes, in table format, the eighteen data sets that were averaged during the initial calibration efforts to create the initial calibrated subcatchment parameters. Figure 8 in Appendix B summarizes, in table format, the initial calibrated subcatchment parameters. Figure 9 in Appendix B summarizes, in table format, the final revised calibrated subcatchment parameters. Figure 10 in Appendix B compares, in table format, the final revised calibrated subcatchment parameters and the revised base subcatchment parameters. Figure 11 in Appendix B compares, in table format, the final revised calibrated soil infiltration parameters and the revised based soil infiltration parameters.

The final simulated conduit flow depth curve for the January 11 - 14, 2020 calibration storm event matched the relative shape, peak timing, peak quantity, and fluctuations of the field measured flow depth curve for both sets of stage gage data. The simulated results for the FG01 stage gage matched the relative shape, timing, and quantity more closely than the simulated results for the FG02 stage gage, but the FG02 simulated results were still close to the measure data. The FG02 simulated results had a higher peak that hit slightly sooner than the first measured peak and a lower peak than the second measured peak.

Refer to Graph 1 through Graph 12 in Appendix C for comparison graphs of the field measured flow depth data overlaid with the simulated flow depth data for the calibration and verification storm events. The comparison graphs are presented in the following order: 1/11/2020 - 1/14/2020 calibration storm event (Graphs 1-2), 11/26/2019 - 11/30/2019 verification storm event (Graphs 3-4), 12/15/2019 - 12/20/2019 verification storm event (Graphs 5-6), 2/4/2020 - 2/9/2020 verification storm event (Graphs 7-8), 2/12/2020 - 2/17/2020 verification storm event (Graphs 9-10), 3/11/2020 - 3/13/2020 verification storm event (Graphs 11-12). The simulated results for these verification storm events matched the relative shape of the measured flow depth data for each storm event, but they did not align as closely with the measured peak values as the January calibration storm event.

5.0 EXISTING CONDITIONS MODEL RESULTS

Once the model parameters were calibrated so that the model simulation results fit the shape, timing, and peak of the observed data to the maximum extent practicable, frequency rainfall data was entered into the model. The NRCS (SCS) Type II 24-hour storm precipitation distribution was utilized to develop frequency rainfall data for use in the InfoSWMM model. The rainfall depths used in the model for the 2-year through 100-year return period storm events were taken directly from Table 2-2 of Volume 2 of the City of Memphis/ Shelby County Stormwater Management (SWM) Manual and are summarized in Table 13 below:

Table 13: NRCS (SCS) 24-Hour Rainfall Depths

Annual Recurrence Interval (ARI)	Duration	Rainfall Depth (in)
2-year	24-hours	4.01
5-year	24-hours	4.89
10-year	24-hours	5.58
25-year	24-hours	6.52
50-year	24-hours	7.27
100-year	24-hours	8.02

Hydrologic time series data was created from the NRCS Type II 24-hour storm precipitation distribution data for the storm events list above. The created time series data was assigned to model rain gage elements, which were then assigned to model subcatchments as rainfall input when simulating specific design storm events. Even though the 2-year through 100-year return period storm events were simulated in the calibrated model, the analysis of the existing conditions model focused primarily on the 10-year return period storm event and secondarily on the 100-year return period storm event.

One of the primary purposes for developing the H&H model for the Harrington Creek drainage basin was for the identification and verification of areas experiencing repetitive flooding problems. This process included mapping the maximum hydraulic grade line (HGL) elevations simulated in the model across the DEM for various storm events and analyzing the quantity and severity of the mapped flooding throughout the basins in the study area. Though this process provides an estimate of the quantity of surface flooding, it also should be noted that unrealistic flooding can occur in specific circumstances. One example of this is where the capacity of a conduit is exceeded and the excess water ponds atop the upstream junction rim elevation within the defined surface area. This defined surface area is a vertical cylinder of water, so the maximum HGL value simulated in areas where this occurs is dependent upon the ponded surface area defined by the modeler. Another example of this is where conduits and junctions are located adjacent to hillsides that extend to surface elevations lower than the elevations of the modeled conduit and junction elements. In these situations, unrealistic inundation can potentially occur in the areas lower than the elevations of the adjacent modeled conduits and junctions.

Note that the model quantifies the HGL surcharge elevations in more detail than the accuracy of the DEM that the inundation results are mapped across so that needs to be kept in mind when reviewing the inundation mapping results. The DEM was created from GIS 1-foot contour data, but the HGL elevations are calculated to the nearest 0.001 feet (0.012 inch). That said, the model has the potential to map large areas of inundation for areas surrounding junctions that have HGL surcharge elevations of 0.000 feet or 0.001 feet. Due to this and the fact that most buildings have a finish floor elevation of approximately 1 foot above the ground surface elevation, the inundation results between 0.001 feet and 0.999 feet were visually mapped but not utilized from the perspective of quantifying the flooded area or number of flooded buildings in the basin. When analyzing the inundation results, the flooding raster files were converted to GIS flooding depth polygons based on 1-foot flooding intervals. The GIS polygons representing a flooding depth of 1 foot of inundation or more were combined into a single polygon layer and utilized to cross-reference GIS building footprint data throughout the basin in order to quantify the flooding footprint area and the number of structures removed from the flooding footprint for the 10-year and 100-year storm events.

The mapped inundation was compared to any reports of known flooding collected during the public meeting in addition to flooding reports provided by the City of Memphis. The City of Memphis Public Works Drain Maintenance Department aided our efforts by collecting flooding and drainage-related maintenance reports and compiling a running list of the reports in the form of a GIS point file. Areas where observed flooding reports and modeled inundation overlapped were identified for further evaluation.

6.0 IMPROVEMENT ALTERNATIVES ANALYSIS & COST ESTIMATE

During the process of cross-referencing the known flooding reports and modeled inundation results eight areas were identified for proposed improvements. Exhibit 17 in Appendix A displays the locations of the identified improvement areas and the City of Memphis flooding and drainage-related maintenance reports. These identified areas were presented to the City of Memphis on 02/11/2022 and the decision was made to move forward with modeling improvement alternatives in all of the identified areas. Accounting for the sub-areas individually, fourteen proposed improvements options were modeled throughout the western City of Memphis portion of the basin. Exhibit 18 in Appendix A displays the locations of the identified improvement sub-areas and the City of Memphis flooding and drainage-related maintenance reports

The following eight areas were identified for proposed improvements:

- IA1: Improvement alternative one was bounded by Lynchburg Street to the west, Beckman Drive to the east, Twin Valley Lane to the north, and Timberdale Avenue to the south. Improvements in this area were split into two sub-areas, 1A and 1B. Improvement alternative 1A was located in the northern half of the Brownsville Road Elementary School property. Improvement alternative 1B was located at the intersection of Banbury Avenue and Sunnyside Street.

- IA2: Improvement alternative two was bounded by Lynchburg Street to the west, North Old Brownsville Road to the east, Banbury Avenue to the north, and Bruton Avenue to the south. Improvements in this area were split into two sub-areas, 2A and 2B. Improvement alternative 2A was located along Timberdale Avenue, Wythe Road, and Gloucester Avenue. Improvement alternative 2B was located north of the intersection of Bruton Avenue and North Old Brownsville Road and west of the intersection of Banbury Avenue and North Old Brownsville Road.

- IA3: Improvement alternative three was bounded by Covington Pike and New Covington Pike and was located northwest of the Northlake Apartments property and northeast of the intersection of New Covington Pike and Covington Pike.

- IA4: Improvement alternative four was bounded by Covington Pike to the west, Chowing Road to the east, Bruton Avenue to the north, and Fernleaf Avenue to the south. Improvements in this area were split into three sub-areas, 4A, 4B, and 4C. Improvement alternative 4A was located northeast of the intersection of Yale Road and Covington Pike through portions of three properties spanning from Yale Road to Chowing Road. Improvement alternative 4B was located southwest of the intersection of Yale Road and Chowing Road on one property. Improvement alternative 4C was located from the intersection Scrivener Drive and Chowing Road south to Fernleaf Avenue.

- IA5: Improvement alternative five was located along Keats Road from the intersection of Keats Road and Fieldcrest Avenue to an area north of the intersection of Keats Road and Craigmont Drive.

- IA6: Improvement alternative six was located at the intersection of Raleigh Lagrange Road and Battle Creek Drive and south of the Avery Park Apartments property. Improvements in this area were split into two sub-areas, 6A and 6B. Improvement alternative 6A was located in a property northeast of the intersection of Battle Creek Drive and Raleigh Lagrange Road and in the property located between the Avery Park Apartments property and the Memphis Light Gas and Water Property. Improvement alternative 6B was located along the north side of Raleigh Lagrange Road from the intersection of Battle Creek Drive to the intersection of West Barbara Circle and in the property located between the Avery Park Apartments property and the Memphis Light Gas and Water Property.

- IA7: Improvement alternative seven was bounded by Chriswood Street and Covington Pike to the west, Raleigh Lagrange Road to the east, Chriswood Street to the north, and Harrington Creek to the south. Improvements in this area were split into two sub-areas, 7A and 7B. Improvement alternative 7A was located along the northern side of the CSX Transportation railroad track, along Chriswood Street, and along Covington Cove. Improvement alternative 7B was located along Wilfong Road and Podesta Street south of the CSX Transportation railroad track.

- IA8: Improvement alternative eight was located south of the intersection of Kimbark Woods Drive and Kimbark Woods Cove and beneath the CSX Transportation railroad track.

The proposed improvements were modeled, and the results were presented to the City of Memphis on 07/01/2022. During this meeting, we were asked to break down the individual improvement results and exclude or add specific improvement options in order to establish the most cost-effective combination of improvements. During this analysis, eighteen improvement combinations were modeled and analyzed based on the reduction in flooded area and structures flooded. Nine of the fourteen proposed improvement alternatives proved to be the most cost-effective combination of improvements. These nine recommended improvements included a combination of pipe network and channel improvements to increase the overall conveyance capacity of the network, detention ponds to intercept and release the captured runoff at a reduced flow rate, and bypass pipe networks and/or channels to provide extra capacity and overflow relief to the network and route the runoff around target areas. The details of the proposed improvement alternatives are presented in the sections below.

6.1 Improvement Alternative Area 1:

Improvement alternative one was bounded by Lynchburg Street to the west, Beckman Drive to the east, Twin Valley Lane to the north, and Timberdale Avenue to the south. Improvements in this area were split into two sub-areas, 1A and 1B. There were 13 flooding and drainage-related maintenance reports in improvement alternative area one that support the existence of flooding issues at this location.

IA1A:

Improvement alternative 1A was located in the northern half of the Brownsville Road Elementary School property. The existing drainage elements in the area consisted of a natural creek running north to south through the wooded area covering the northwest side of the property (Open Channel Conduit 785-100062). The proposed improvements in the area include constructing an overflow detention pond adjacent to the top of bank of the creek to intercept and release the captured runoff at a reduced flow rate.

- Open Detention Pond DET1.3: The proposed pond was 7 feet deep, had 3:1 grass lined inner side slopes, grass lined 3:1 outer tie-in slopes, and a storage volume of 9.5 acre-feet. The pond berm adjacent to the creek had a 100-foot wide and 1.4-foot-tall trapezoidal weir with 3:1 side slopes to allow high water levels in the creek to inflow into the pond. The pond had a yard inlet outlet structure that utilized the combination of a bottom 12-inch orifice and the four overflow structure openings to regulate the outflow of the runoff exiting the pond. The outlet structure had a 36" RCP outlet pipe connected to the downstream southern end of the natural creek (Open Channel Conduit 785-100062).

Based on the mapped inundation results for the 10-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 2 acres and removed 1 structure located within the City of Memphis limits from the 10-year flooding footprint. When combined with the IA2B and IA1B improvements the flooded area in the basin was reduced by a total of approximately 1 acre and a total of approximately 8 structures were removed from the 10-year flooding footprint. Based on the mapped inundation results for the 100-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 6 acres and removed approximately 12 structures located within the City of Memphis limits from the 100-year flooding footprint. When combined with the IA2B and IA1B improvements the flooded area in the basin was reduced by a total of approximately 11 acres and a total of approximately 69 structures were removed from the 100-year flooding footprint.

Exhibit 19 in Appendix A outlines the proposed improvements within area IA1A. Exhibit 20 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 10-year recurrence interval storm event. Exhibit 21 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 100-year recurrence interval storm event.

The preliminary anticipated cost for these improvements is estimated to be approximately $2,115,000.00. Figure 12 in Appendix D summarizes, in table format, an itemized planning level cost estimate for improvement alternative 1A.

IA1B:

Improvement alternative 1B was located at the intersection of Banbury Avenue and Sunnyside Street. The existing system in the area consisted of one run of 15" RCP pipe, one run of 24" RCP pipe, three runs of 30" RCP pipe, and two runs of 36" RCP pipe. The proposed improvements in the area include a combination of pipe network improvements to increase the overall conveyance capacity of the network, and a new bypass pipe network to provide extra capacity and overflow relief to the network and route the runoff around target areas.

- Closed Conduit Segment 595-592: Remove existing 274 LF 30" RCP and connected drainage structures and replace with 274 LF of 42" RCP with larger drainage structures. Also, remove existing 43 LF 36" RCP and connected drainage structures and replace with 43 LF of 48" RCP with larger drainage structures.

- Closed Conduit Segment 591-592: Remove or abandon in place existing 147 LF 24" RCP and replace with newly aligned 163 LF of 2 parallel (42" RCP Eq.) 51" x 31" RCAP with larger drainage structures.

- Closed Conduit Segment 842-100059: Remove existing 278 LF 30" RCP and connected drainage structures and replace with 278 LF of 48" RCP with larger drainage structures.

- Closed Conduit Segment 592-100059_B: Construct new bypass pipe network consisting of 474 LF of 2 parallel 42" RCP and connected drainage structures. Also, remove existing 141 LF 15" RCP and connected drainage structures and replace with 141 LF of 2 parallel 48" RCP with larger drainage structures.

- Additional improvements outside of the project modeling scope were recommended for this area. These additional improvements include the following: replacing two adjacent existing 15" RCP pipe runs with two runs of 24" RCP pipe, and one adjacent existing 18" RCP pipe run with a run of 24" RCP pipe.

Based on the mapped inundation results for the 10-year storm simulation, when combined with the IA2B and IA1A improvements, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by a total of approximately 1 acre and removed a total of approximately 8 structures located within the City of Memphis limits from the 10-year flooding footprint. Based on the mapped inundation results for the 100-year storm simulation, when combined with the IA2B and IA1A improvements, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by a total of approximately 11 acres and removed a total of approximately 69 structures located within the City of Memphis limits from the 100-year flooding footprint.

Exhibit 22 in Appendix A outlines the proposed improvements within area IA1B. Exhibit 23 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 10-year recurrence interval storm event. Exhibit 24 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 100-year recurrence interval storm event.

The preliminary anticipated cost for these improvements is estimated to be approximately $2,164,230.00. Figure 13 in Appendix D summarizes, in table format, an itemized planning level cost estimate for improvement alternative 1B.

6.2 Improvement Alternative Area 2:

Improvement alternative two was bounded by Lynchburg Street to the west, North Old Brownsville Road to the east, Banbury Avenue to the north, and Bruton Avenue to the south. Improvements in this area were split into two sub-areas, 2A and 2B. There were 27 flooding and drainage-related maintenance reports in improvement alternative area one that support the existence of flooding issues at this location.

IA2A:

Improvement alternative 2A was located along Timberdale Avenue, Wythe Road, and Gloucester Avenue. The existing system in the area consisted of two runs of 24” RCP pipe, and one run of 27” RCP pipe. The proposed improvements in the area include a combination of pipe network improvements to increase the overall conveyance capacity of the network, and a new bypass pipe network to provide extra capacity and overflow relief to the network and route the runoff around target areas.

- Closed Conduit Segment 586-585: Remove existing 336 LF 24” RCP and connected drainage structures and replace with 336 LF of 36” RCP with larger drainage structures. Also, remove existing 38 LF 27” RCP and connected drainage structures and replace with 38 LF of 42” RCP with larger drainage structures.

- Closed Conduit Segment 589-767: Construct new bypass pipe network consisting of 674 LF of 48” RCP, 268 LF of 54” RCP, 300 LF of 24” RCP (lateral) and connected drainage structures.

- Additional improvements outside of the project modeling scope were recommended for this area. These additional improvements include the following: replacing three adjacent existing 18” RCP pipe runs with three runs of 24” RCP pipe, and one adjacent existing 15” RCP pipe run with a run of 24” RCP pipe.

Based on the mapped inundation results for the 10-year storm simulation, when combined with the IA2B, IA1A, and IA1B improvements, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by a total of approximately 1 acre and removed a total of approximately 8 structures located within the City of Memphis limits from the 10-year flooding footprint. Based on the mapped inundation results for the 100-year storm simulation, when combined with the IA2B, IA1A, and IA1B improvements, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by a total of approximately 18 acres and removed a total of approximately 75 structures located within the City of Memphis limits from the 100-year flooding footprint.

Exhibit 25 in Appendix A outlines the proposed improvements within area IA2A. Exhibit 26 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 10-year recurrence interval storm event. Exhibit 27 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 100-year recurrence interval storm event.

The preliminary anticipated cost for these improvements is estimated to be approximately $1,959,360.00. Figure 14 in Appendix D summarizes, in table format, an itemized planning level cost estimate for improvement alternative 2A.

IA2B:

Improvement alternative 2B was located north of the intersection of Bruton Avenue and North Old Brownsville Road and west of the intersection of Banbury Avenue and North Old Brownsville Road. The existing system in the area consisted of two runs of 42” RCP pipe, one run of 48” RCP pipe, one span of 7’ x 14’ rectangular-triangular bottom concrete open channel, one box culvert that transitions from 4.5’ x 15.5’ to 9.7’ x 12.2’, and a natural open channel downstream of the drainage system. The proposed improvements in the area include pipe network and channel improvements to increase the overall conveyance capacity of the network.

- Closed Conduit Segment 879-764: Remove existing 293 LF 48” RCP and connected drainage structures and replace with 293 LF of 60” RCP with larger drainage structures.

- Closed Conduit Segment 817-819: Remove existing 50 LF 42” RCP and connected drainage structures and replace with 50 LF of (60” RCP equivalent) 76” x 48” HERCP with larger drainage structures.

- Closed Conduit Segment 765-764: Remove existing 68 LF 4.5' x 15.5' box culvert and 39 LF 9.7' x 12.2' box culvert and replace with lowered 107 LF 7' x 15.5' box culvert. Also, reconstruct adjacent upstream and downstream open channels to ensure positive drainage.

Based on the mapped inundation results for the 10-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 1 acre and removed approximately 6 structures located within the City of Memphis limits from the 10-year flooding footprint. When combined with the IA2B, IA1A, and IA1B improvements the flooded area in the basin was reduced by a total of approximately 1 acre and a total of approximately 8 structures were removed from the 10-year flooding footprint. Based on the mapped inundation results for the 100-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 2 acres and removed approximately 41 structures located within the City of Memphis limits from the 100-year flooding footprint. When combined with the IA2B, IA1A, and IA1B improvements the flooded area in the basin was reduced by a total of approximately 18 acres and a total of approximately 75 structures were removed from the 100-year flooding footprint.

Exhibit 28 in Appendix A outlines the proposed improvements within area IA2B. Exhibit 29 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 10-year recurrence interval storm event. Exhibit 30 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 100-year recurrence interval storm event.

The preliminary anticipated cost for these improvements is estimated to be approximately $2,636,250.00. Figure 15 in Appendix D summarizes, in table format, an itemized planning level cost estimate for improvement alternative 2B.

Exhibit 31 and Exhibit 32 in Appendix A display the existing vs. post-improvement inundation comparisons for the 10-year and 100-year recurrence interval storm events for the overall area of proposed improvements IA1A, IA1B, IA2A, IA2B.

6.3 Improvement Alternative Area 3:

Improvement alternative three was bounded by Covington Pike and New Covington Pike and was located northwest of the Northlake Apartments property and northeast of the intersection of New Covington Pike and Covington Pike. There were 9 flooding and drainage-related maintenance reports in improvement alternative area one that support the existence of flooding issues at this location.

IA3:

This improvement alternative was not selected as a final recommended improvement, but below is a summary of the modeled improvements:

This modeled proposed improvement included constructing a detention pond on the property northeast of the intersection of New Covington Pike and Covington Pike. The proposed pond was 8.1 feet deep, had 3:1 grass lined inner side slopes, and a storage volume of 14.0 acre-feet. The pond had a yard inlet outlet structure that utilized the combination of a bottom 30-inch orifice and the four overflow structure openings to regulate the outflow of the runoff exiting the pond. The outlet structure had a 36” RCP outlet pipe connected to the existing storm sewer pipe crossing Covington Pike. The improvement also included removing one run of existing 24” RCP pipe, replacing one run of 24” RCP pipe with a run of 2 parallel 30” RCP pipes and replacing a run of 24” RCP pipe with a run of 2 parallel 42” RCP pipes. Exhibit 33 in Appendix A outlines the modeled improvements within area IA3.

6.4 Improvement Alternative Area 4:

Improvement alternative four was bounded by Covington Pike to the west, Chowing Road to the east, Bruton Avenue to the north, and Fernleaf Avenue to the south. Improvements in this area were split into three sub-areas, 4A, 4B, and 4C. There were 14 flooding and drainage-related maintenance reports in improvement alternative area one that support the existence of flooding issues at this location.

IA4A:

Improvement alternative 4A was located northeast of the intersection of Yale Road and Covington Pike through portions of three properties spanning from Yale Road to Chowing Road. The existing system in the area consisted of two runs of 36” RCP pipe separated by an open channel segment, one run of 24” RCP pipe, and one run of 27” RCP pipe. The proposed improvements in the area include constructing a detention pond to release the captured runoff at a reduced flow rate and a new bypass pipe network to route upstream runoff to the new pond.

- Open Detention Pond DET4.1: The proposed pond was 9 feet deep, had 2:1 rip rap lined inner side slopes, grass lined 2.5:1 outer tie-in slopes, and a storage volume of 14.0 acre-feet. The pond had a yard inlet outlet structure that utilized the combination of a bottom 18-inch orifice, a 1.25’ x 4’ (height x width) rectangular orifice, and four overflow structure openings to regulate the outflow of the runoff exiting the pond. The outlet structure connected to the downstream existing 36” RCP pipe remaining in place.

- Closed Conduit Segment 478-DET4.1: Construct new bypass pipe network consisting of 29 LF of (48” RCP Eq.) 60” x 38” HERCP, 198 LF of 48” RCP and connected drainage structures.

- Additional improvements outside of the project modeling scope were recommended for this area. These additional improvements include the following: adding two new drainage structures and three runs of 24” RCP pipe on Yale Road southwest of the proposed detention pond.

Based on the mapped inundation results for the 10-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 1 acre and removed approximately 3 structures located within the City of Memphis limits from the 10-year flooding footprint. Based on the mapped inundation results for the 100-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 12 acres and removed approximately 48 structures located within the City of Memphis limits from the 100-year flooding footprint.

Exhibit 34 in Appendix A outlines the proposed improvements within area IA4A. Exhibit 35 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 10-year recurrence interval storm event. Exhibit 36 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 100-year recurrence interval storm event. Additionally, Exhibit 37 and Exhibit 38 compare the existing vs. post-improvement inundation results for the 10-year and 100-year recurrence interval storm events at a zoomed out view.

The preliminary anticipated cost for these improvements is estimated to be approximately $2,518,747.50. Figure 16 in Appendix D summarizes, in table format, an itemized planning level cost estimate for improvement alternative 4A.

IA4B:

This improvement alternative was not selected as a final recommended improvement, but below is a summary of the modeled improvements:

This modeled proposed improvement included constructing an underground detention vault on the property southwest of the intersection of Yale Rd. and Chowing Rd. The proposed underground detention was 8 feet deep and a storage volume of 2.5 acre-feet. The pond had a 7-foot-tall v-notch weir with an opening angle of 40 degrees and a 1’ x 60’ overflow weir to regulate the outflow of the runoff exiting the vault. It also had a 60” RCP outlet pipe, and the improvement included replacing one run of 48” RCP pipe with a run of 2 parallel 48” RCP pipes. Exhibit 39 in Appendix A outlines the modeled improvements within area I4B.

IA4C:

This improvement alternative was not selected as a final recommended improvement, but below is a summary of the modeled improvements:

This modeled proposed improvement included constructing a bypass pipe line extending from Scrivener Dr. to Patrick Henry Drive. This bypass line consisted of five new runs of 36” RCP pipe. The improvement also included replacing three runs of 60” RCP pipe with three runs of 5.5’ x 6’ box culvert, replacing one span of 5.8’ x 6’ rectangular-triangular bottom concrete channel with a run of 5.5’ x 6’ box culvert, and installing three new runs of 24” RCP pipe. Exhibit 40 in Appendix A outlines the modeled improvements within area IA4C.

6.5 Improvement Alternative Area 5:

Improvement alternative five was located along Keats Road from the intersection of Keats Road and Fieldcrest Avenue to an area north of the intersection of Keats Road and Craigmont Drive. There were 5 flooding and drainage-related maintenance reports in improvement alternative area one that support the existence of flooding issues at this location.

IA5:

Improvement alternative five was located along Keats Road from the intersection of Keats Road and Fieldcrest Avenue to an area north of the intersection of Keats Road and Craigmont Drive. The existing system in the area consisted of five runs of 48” RCP pipe. The proposed improvements in the area include a combination of pipe network improvements to increase the overall conveyance capacity of the network, and a new bypass pipe network to provide extra capacity and overflow relief to the network and route the runoff around target areas.

- Closed Conduit Segment 435-446: Remove existing 713 LF 48” RCP and replace with 713 LF of 2 parallel 42” RCP with larger drainage structures.

- Closed Conduit Segment IA5_18-IA5_17: Construct new bypass pipe network consisting of 301 LF of 30” RCP, 725 LF of 42” RCP, 146 LF of 36” RCP (lateral), and connected drainage structures.

- Additional improvements outside of the project modeling scope were recommended for this area. These additional improvements include the following: replacing two adjacent existing 18” RCP pipe runs with two runs of 24” RCP pipe, and two adjacent existing 15” RCP pipe runs with two runs of 24” RCP pipe.

Based on the mapped inundation results for the 10-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 1 acre and removed approximately 5 structures located within the City of Memphis limits from the 10-year flooding footprint. Based on the mapped inundation results for the 100-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 3 acres and removed approximately 12 structures located within the City of Memphis limits from the 100-year flooding footprint.

Exhibit 41 in Appendix A outlines the proposed improvements within area IA5. Exhibit 42 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 10-year recurrence interval storm event. Exhibit 43 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 100-year recurrence interval storm event.

The preliminary anticipated cost for these improvements is estimated to be approximately $2,538,285.00. Figure 17 in Appendix D summarizes, in table format, an itemized planning level cost estimate for improvement alternative 5.

6.6 Improvement Alternative Area 6:

Improvement alternative six was located at the intersection of Raleigh Lagrange Road and Battle Creek Drive and south of the Avery Park Apartments property. Improvements in this area were split into two sub-areas, 6A and 6B. There were 19 flooding and drainage-related maintenance reports in improvement alternative area one that support the existence of flooding issues at this location.

IA6A:

This improvement alternative was not selected as a final recommended improvement, but below is a summary of the modeled improvements:

This modeled proposed improvement included constructing a detention pond on the property northeast of the intersection of Raleigh Lagrange Road and Battle Creek Drive. The proposed pond was 8 feet deep, had 3:1 grass lined inner side slopes, and a storage volume of 7.1 acre-feet. The pond had a yard inlet outlet structure that utilized the combination of a bottom 12-inch orifice and the four overflow structure openings to regulate the outflow of the runoff exiting the pond. The outlet structure had a 36" RCP outlet pipe connected to the adjacent open channel. The improvement also included one new run of 2 parallel 48" RCP pipes, replacing one run of 24" RCP pipe with a run of 36" RCP pipe, and constructing a new 20-foot bottom width, 3-foot-deep grass-lined trapezoidal bypass channel with 3:1 side slopes. Exhibit 44 in Appendix A outlines the modeled improvements within area IA6A.

IA6B:

Improvement alternative 6B was located along the north side of Raleigh Lagrange Road from the intersection of Battle Creek Drive to the intersection of West Barbara Circle, along Bay Pointe Circle East, and in the property located between the Avery Park Apartments property and the Memphis Light Gas and Water Property. The existing system in the area consisted of one run of 24" RCP pipe, and one open channel segment downstream of the 24" RCP pipe. The proposed improvements in the area include a combination of pipe network and channel improvements to increase the overall conveyance capacity of the network, and a new bypass pipe network and bypass open channel to provide extra capacity and overflow relief to the network and route the runoff around target areas.

- Closed and Open Conduit Segment 320-400031: Remove existing 124 LF 24" RCP and replace with 124 LF of 36" RCP with larger drainage structures. Also install a 52 LF rip rap trapezoidal channel with the following dimensions: 4' bottom width, 4' depth, 2.5:1 side slopes.

- Open Conduit Segment 400013-300010: Construct a new bypass open channel consisting of a 430 LF grass trapezoidal channel with the following dimensions: 20' bottom width, 3' depth, 3:1 side slopes.

- Closed Conduit Segment 755-734: Construct new bypass pipe network consisting of 979 LF of 60" RCP and connected drainage structures

- Additional improvements outside of the project modeling scope were recommended for this area. These additional improvements include the following: replacing one adjacent existing 18" RCP pipe run with one run of 36" RCP pipe.

Based on the mapped inundation results for the 10-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 5 acres and removed 0 structures located within the City of Memphis limits from the 10-year flooding footprint. Based on the mapped inundation results for the 100-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 17 acres and removed approximately 16 structures located within the City of Memphis limits from the 100-year flooding footprint.

Exhibit 45 in Appendix A outlines the proposed improvements within area IA6B. Exhibit 46 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 10-year recurrence interval storm event. Exhibit 47 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 100-year recurrence interval storm event.

The preliminary anticipated cost for these improvements is estimated to be approximately $2,032,875.00. Figure 18 in Appendix D summarizes, in table format, an itemized planning level cost estimate for improvement alternative 6B.

6.7 Improvement Alternative Area 7:

Improvement alternative seven was bounded by Chriswood Street and Covington Pike to the west, Raleigh Lagrange Road to the east, Chriswood Street to the north, and Harrington Creek to the south. Improvements in this area were split into two sub-areas, 7A and 7B. There were 3 flooding and drainage-related maintenance reports in improvement alternative area one that support the existence of flooding issues at this location.

IA7A:

Improvement alternative 7A was located at the intersection of the CSX Transportation railroad track and Covington Pike along the northern side of the railroad track, along Chriswood Street, and along Covington Cove. The existing system in the area consisted of two runs of 27” RCP pipe, two runs of 48” RCP pipe, one run of 60” RCP pipe, and two open channel segments located on either side of the 60” RCP pipe. The proposed improvements in the area include pipe network and channel improvements to increase the overall conveyance capacity of the network.

- Closed Conduit Segment 264-261: Remove existing 765 LF 27” RCP and connected drainage structures and replace with 765 LF of 2 parallel 30” RCP pipes with larger drainage structures.

- Closed Conduit Segment 254-500006: Remove existing 613 LF 48” RCP and connected drainage structures and replace with 613 LF of 2 parallel 48” RCP pipes with larger drainage structures.

- Closed Conduit Segment 262-261: Remove existing 355 LF 60” RCP and replace with 355 LF of 7’ x 5’ rectangular concrete channel. Also, regrade/reconstruct adjacent upstream and downstream open channels to ensure positive drainage.

Based on the mapped inundation results for the 10-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 14 acres and removed approximately 11 structures located within the City of Memphis limits from the 10-year flooding footprint. Based on the mapped inundation results for the 100-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 25 acres and removed approximately 19 structures located within the City of Memphis limits from the 100-year flooding footprint.

Exhibit 48 in Appendix A outlines the proposed improvements within area IA7A. Exhibit 49 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 10-year recurrence interval storm event. Exhibit 50 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 100-year recurrence interval storm event.

The preliminary anticipated cost for these improvements is estimated to be approximately $2,432,250.00. Figure 19 in Appendix D summarizes, in table format, an itemized planning level cost estimate for improvement alternative 7A.

IA7B:

Improvement alternative 7B was located at the intersection of the CSX Transportation railroad track and Covington Pike along the southern side of the railroad track, along Wilfong Road.

This improvement alternative was not selected as a final recommended improvement, but below is a summary of the modeled improvements:

This modeled proposed improvement included installing four new runs of 36” RCP pipe, replacing one run of 24” RCP pipe with a run of 2 parallel 42” RCP pipes, replacing two runs of 36” RCP pipe with two runs of 2 parallel 42” RCP pipes, replacing one run of 42” RCP pipe with a run of 2 parallel 42” RCP pipes, and replacing three runs of 48” RCP pipe with three runs of 2 parallel 42” RCP pipes. Exhibit 51 in Appendix A outlines the modeled improvements within area IA7B.

6.8 Improvement Alternative Area 8:

Improvement alternative eight was located south of the intersection of Kimbark Woods Drive and Kimbark Woods Cove and beneath the CSX Transportation railroad track. There were 19 flooding and drainage-related maintenance reports in improvement alternative area one that support the existence of flooding issues at this location.

IA8:

Improvement alternative eight was located south of the intersection of Kimbark Woods Drive and Kimbark Woods Cove and beneath the CSX Transportation railroad track. The existing system in the area consisted of one run of 48” RCP pipe, one run of 2 parallel 48” CMP pipes beneath the railroad track, and two open channel segments on either side of the CMP pipes. The proposed improvements in the area include pipe network and channel improvements to increase the overall conveyance capacity of the network.

- Closed Conduit Segment 2015-307: Remove existing 200 LF 48” RCP and connected drainage structures and replace with 200 LF of 2 parallel 48” RCP pipes with larger drainage structures.

- Closed Conduit Segment 309-311: Jack and bore replacement of existing 37 LF 2 parallel 48” CMP pipes with 37 LF of 2 parallel 48” DI pipes. Also, regrade/reconstruct adjacent upstream and downstream open channels to ensure positive drainage.

- Additional improvements outside of the project modeling scope were recommended for this area. These additional improvements include the following: replacing one adjacent existing 15” RCP pipe run with one run of 48” RCP pipe.

Based on the mapped inundation results for the 10-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 11 acres and removed approximately 8 structures located within the City of Memphis limits from the 10-year flooding footprint. Based on the mapped inundation results for the 100-year storm simulation, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 8 acres and removed approximately 20 structures located within the City of Memphis limits from the 100-year flooding footprint.

Exhibit 52 in Appendix A outlines the proposed improvements within area IA8. Exhibit 53 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 10-year recurrence interval storm event. Exhibit 54 in Appendix A compares the extent of the existing vs. post-improvement inundation results for the 100-year recurrence interval storm event.

The preliminary anticipated cost for these improvements is estimated to be approximately $1,075,470.00. Figure 20 in Appendix D summarizes, in table format, an itemized planning level cost estimate for improvement alternative 8.

6.9 Flood Reduction and Cost Benefit Analysis:

Preliminary planning costs were prepared for the nine proposed improvement alternatives. These costs were based on planning-level design information and are not meant to be considered an Engineer's Estimate of Probable Construction Costs. These estimates are purely a budget estimate for planning purposes. The costs were derived from Tennessee Department of Transportation (TDOT) bid tab unit costs. Figure 12 through Figure 20 in Appendix C summarize, in table format, an itemized planning level cost estimate for each recommended improvement alternative. The cost estimates are presented in the following order: IA1A (Figure 12), IA1B (Figure 13), IA2A (Figure 14), IA2B (Figure 15), IA4A (Figure 16), IA5 (Figure 17), IA6B (Figure 18), IA7A (Figure 19), and IA8 (Figure 20).

The preliminary costs were also compared to the flood reduction impacts of each alternative and a cost benefit analysis was performed for the 10-year and 100-year storm events. This analysis included evaluating the number of City of Memphis structures removed from the flooding footprint greater than or equal to one-foot depth for each of the storm events and calculating an approximate cost per structure removed for each recommended improvement alternative. This analysis also enabled us to group the improvements into a recommended construction sequence based on the target storm event. Figure 21 in Appendix C summarizes, in table format, the cost benefit analysis results for the 10-year storm event. Figure 22 in Appendix C summarizes, in table format, the cost benefit analysis results for the 100-year storm event.

Based on the mapped inundation results for the 10-year storm simulation, when all nine of the recommended improvements are combined, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 46 acres and removed approximately 45 structures located within the City of Memphis limits from the 10-year flooding footprint. Based on the mapped inundation results for the 100-year storm simulation, when all nine of the recommended improvements are combined, these improvements reduced the flooded area in the basin greater than or equal to 1 foot in depth by approximately 81 acres and removed approximately 192 structures located within the City of Memphis limits from the 100-year flooding footprint. Figure 23 in Appendix C summarizes, in table format, the flooded building and area reduction analysis for all eighteen modeled improvement alternative combinations for the 10-year and 100-year storm events.